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Development of an integrated project-level pavement management model using risk analysis

Jennifer A. Reigle
West Virginia University

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Development of an Integrated Project-Level Pavement Management Model Using Risk Analysis

Jennifer A. Reigle

Dissertation submitted to the
College of Engineering and Mineral Resources
at West Virginia University
in partial fulfillment of the requirements
for the degree of

Doctor of Philosophy
in
Civil Engineering

John P. Zaniewski, Ph.D., P.E., Chair
Darrell R. Dean, Ph.D., P.S.
Ronald W. Eck, Ph.D., P.E.
Gerald R. Hobbs, Ph.D.
David R. Martinelli, Ph.D.

Department of Civil and Environmental Engineering

Morgantown, West Virginia
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Pavement Rehabilitation and Preventive Maintenance

ABSTRACT

Development of an Integrated Project-Level Pavement Management Model Using Risk Analysis

Jennifer A. Reigle

Historically, federal highway funding focused on the construction of new pavements and the upgrading of existing pavements. Today, much of the infrastructure is in place. Therefore, the focus of federal funding is shifting toward pavement maintenance and preservation. With this in mind, highway agencies are directing attention toward pavement preservation strategies that yield the greatest value from existing pavements.

Life cycle cost analysis (LCCA) is a decision-making tool that highway agencies may use in selecting an optimal pavement preservation strategy. Traditionally, LCCA models for pavement management use discrete input values that represent a conservative “best guess” of each parameter. Thus, inherent uncertainty associated with each input parameter is not considered. There are situations, however, when this uncertainty may significantly influence the decision-making process.

The model developed for this research is a probabilistic model that derives flexible pavement designs, generates preservation strategies, and evaluates the life-cycle costs of each alternative. Risk analysis is incorporated into the LCCA model so that the inherent uncertainty of each input parameter is considered. Other features of the model include the incorporation of functional aspects (structural capacity and pavement condition) and safety (skid resistance) into the design, the inclusion of rehabilitation and preventive maintenance as preservation strategy alternatives, and the consideration of both agency and user cost in the present worth cost analysis.

The LCCA model output consists of probability distributions that describe the total present worth cost, the agency present worth cost, and the user present worth cost for each preservation strategy over a specified analysis period. The probabilistic nature of this LCCA model exposes areas of uncertainty that may be hidden in a deterministic LCCA model, and allows the decision-maker to assess the risk associated with each preservation strategy based on the probability of various costs that may be incurred.

Finally, a sensitivity analysis was performed to assess the effects of various input parameters on model output. The highway agency can enhance the model output by focusing more detailed data collection and parameter estimation on the model components that were identified as having a statistically significant effect on the model results.

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CHAPTER 1

INTRODUCTION

Historically, federal highway funding focused on the construction of new pavements and the upgrading of existing pavements. New pavement structures were constructed, opened to traffic, and left to deteriorate over time until either rehabilitation was performed or pavement reconstruction was necessary. Today, much of the pavement infrastructure is in place. Therefore, the focus of federal funding is shifting toward pavement maintenance and preservation.

With this in mind, highway agencies are directing attention toward pavement preservation strategies that yield the greatest value from existing pavements. For example, rather than allowing the pavement to deteriorate until it reaches an unacceptable level of serviceability and requires reconstruction, highway agencies may choose to perform rehabilitation, such as a structural overlay, on the pavement at a time when it has deteriorated below a specified serviceability level, but has not yet reached failure. Rehabilitation improves the capacity of the existing pavement, thereby extending the pavement life. An alternative maintenance strategy is to apply preventive maintenance treatments to the pavement while it is still in good structural condition. The objective of preventive maintenance is to extend the life of a pavement by applying relatively inexpensive treatments before the pavement deteriorates to a condition that requires rehabilitation or reconstruction.

It has been suggested that a proper preventive maintenance strategy can provide high-quality pavements over many years at minimal cost (Zaniewski and Mamlouk, 1996, Hicks, et al., 1997). One approach that may be used to investigate this theory is to perform a life-cycle cost analysis on various maintenance strategies, such as those illustrated in Figure 1.1.

Life-cycle cost analysis (LCCA) is a technique that builds on the well-founded principles of economic analysis to evaluate the overall long-term economic efficiency between competing alternative investment options (Walls and Smith, 1998). LCCA incorporates initial and discounted future agency and user costs over the life of the alternative investments to identify an optimal investment strategy. Typically, a sensitivity analysis is performed on the LCCA results to identify the significant input variables and to address the effects of variations in the input estimates on model results. Combining the LCCA output with sensitivity analysis results may provide valuable guidance to a decision-maker when faced with selecting an optimal investment strategy.

Figure 1.1 Example of Maintenance Alternatives for LCCA.

Traditionally, LCCA models use discrete input values that represent the "best guess" of each parameter. The inherent variability associated with each input parameter is not included in the model, and therefore is not reflected in the model results. There are situations, however, when this uncertainty may significantly influence the decision-making process. Therefore, it is desirable to incorporate inherent variability into the LCCA so that the decision-maker can weigh the probability of any particular outcome that may occur.

The research presented in this document involves the development of a unique model for evaluating the life-cycle costs of pavement preservation strategies. The LCCA model is developed using an analysis tool that incorporates the inherent variability associated with each input parameter. The research results provide highway agencies with insight regarding the most cost-effective preservation strategies, along with each strategy's most influential parameters and greatest cost components.

1.1 LIFE-CYCLE COST ANALYSIS (LCCA) LEGISLATIVE REQUIREMENTS

The National Highway System (NHS) Designation Act of 1995 required state highway agencies to conduct a life-cycle cost analysis (LCCA) of each NHS high cost usable project segment. Section 303 of the NHS Designation Act legislatively defined LCCA as “a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment.” The term “high cost” referred to useable project segments estimated to cost \$25 million or more, and a “useable project segment” was defined as “a portion of highway which a State proposed to construct, reconstruct or improve that when completed could be opened to traffic independent of some larger overall project” (Kane, 1996).

Although the Federal Highway Administration (FHWA) Division Offices did not prescribe the form of LCCA that a state highway agency should follow, it was expected that the state highway agencies should conduct their LCCA to reflect good/best practices. This means that the LCCA should:

- have sufficiently long analysis periods to reflect long term cost differences associated with reasonable investment alternatives,
- employ accepted discount rates, and
- address the inherent variability in input parameters.

Furthermore, the LCCA should be conducted at an early stage of the project development to prevent unnecessary delays at later stages (Kane, 1996).

The 1998 Transportation Equity Act for the 21st Century (TEA-21) removed the requirement for state highway agencies to conduct LCCA. However, FHWA policy recommends LCCA as a decision support tool, emphasizing that the results are not decisions in and of themselves. Often, the logical analytical framework this type of analysis promotes is as important as the LCCA results themselves (Walls and Smith, 1998). Therefore, state highway agencies are still encouraged to use LCCA when making investment decisions.

1.2 THE STRUCTURE OF A LCCA MODEL

The development and implementation of any model, whether deterministic or probabilistic, should be based on a logical simulation of the progression of activities involved in solving a problem. One method that is used to structure problem solving is known as the systems method. The systems method, described below, was the general structure used to develop the LCCA for this research.

1.2.1 The Systems Method

In general, the systems method is a logical, systematic process that may be used to solve problems efficiently. The comprehensive problem-solving process involves handling or managing a network of interrelated problems and/or tasks on a global basis to achieve the maximum utility or benefit (Haas, et al., 1994). With respect to pavement life-cycle cost analysis, this method provides a systematic procedure for the economic analysis of a large number of pavement strategies.

The following excerpt from Haas, et al. (1994) describes the major phases and components of the systems method shown in Figure 1.2:

The diagram illustrates that the recognition of a problem comes from some perceived inadequacy or need in the environment. It leads to a definition of the problem that involves a more in-depth understanding. This provides the basis for proposing alternative solutions. These alternatives are then analyzed in order to predict their probable outputs or consequences. Evaluation of the outputs is the next step in order that an optimal solution may be chosen. Implementation involves putting this solution into service, and its operation. Feedback for improving future solution, or checking on how well the system is fulfilling its function, is provided by periodic performance measurements.

1.2.2 Network Versus Project Level Analysis

Basically, the application of principles of engineering economy to pavement engineering occurs at two levels. At one level, management decisions are required to determine the feasibility and timing of projects. The second level is the requirement to achieve the maximum economy for a specific project (AASHTO, 1993).

The first level is referred to as the network level. This includes management type functions such as establishing priorities for various design or construction projects, determining the optimal use of funds in a limited budget, and selecting optimum maintenance policies for the entire network. The advantage of the network level is its ability to minimize total overall costs while maximizing utility (Seeds, 1980). However, the disadvantage of the network level is that the design models are simple, and thus do not adequately consider all factors associated with design at the project level.

Figure 1.2 Major Phases and Components of the Systems Method (Haas, et al., 1994).

Project level systems generally provide criteria for the selection of an optimum pavement design strategy for a specific section of road. Project level models are typically complex, dealing with technical concerns and requiring detailed information (Haas, et al., 1994).

Ideally, a systems model for pavement LCCA should include the entire management and decision-making process for design, construction, and maintenance at both the project and network levels. However, the model presented in this study was developed at the project level. At this level, the problem or task is to select a pavement design and preservation strategy that will provide an acceptable level of service to the user over a given period of time at a minimum overall cost.

It should be noted that network level and project level management systems are mutually dependent. For example, one of the many functions of a pavement management system (PMS) at the network level may be to identify pavement sections within a network of pavements that require immediate maintenance action. Subsequently, the project level may be responsible for determining the optimum maintenance strategy for each pavement section identified by the network level. After the maintenance strategy is selected and implemented for a particular pavement section, feedback is provided to both the network and project management levels. Therefore, although the LCCA model presented in this study was developed at the project level, the output of the model and results after implementation should be used to provide feedback to the network level.

1.3 TRADITIONAL LCCA MODELS AND IMPLEMENTATION LIMITATIONS

A general procedure for conducting a LCCA on a particular pavement section may include the following steps:

- determine alternative design strategies over the analysis period,
- determine treatment timings,
- estimate agency costs,
- estimate user costs,
- compute net present value, and
- analyze results.

Performing these steps over a sufficiently long analysis period using acceptable discount rates is fairly straightforward. However, there are several limitations in the development and implementation of many existing LCCA models.

One limitation of many existing LCCA models is the exclusion of user costs in the analysis. User costs are costs incurred by the user, and may include accident costs, the cost of

time associated with user delay, and vehicle operating costs (such as fuel, tires, engine oil, and vehicle maintenance). The exclusion of user costs in the LCCA may be due to the fact that user costs are difficult to quantify and the values associated with user costs are often disputed.

Another limitation in many existing LCCA models for pavements is the exclusion of preventive maintenance as a maintenance strategy alternative. Because preventive maintenance is a relatively new preservation strategy for pavements, data relating to the long-term benefits are still being collected. At this time, there are only a limited number of models that attempt to quantify the long-term effects of preventive maintenance treatments. Therefore, the incorporation of preventive maintenance in LCCA models is a challenge.

Finally, accounting for the uncertainty of input parameters in the LCCA is extremely complicated, and thus is often ignored. Traditionally, LCCA models treat input variables as discrete, fixed values where a conservative "best guess" of the value of each input parameter is used to compute a single deterministic result. A sensitivity analysis is often performed to assess the effects of various input parameters on the model results. However, the sensitivity analysis does not necessarily reveal areas of uncertainty that may be a critical part of the decision making process. This shortcoming of deterministic LCCA models can lead to endless debates over the validity of the results. In this situation, it is difficult to ascertain which alternative "truly" has the lowest life-cycle cost (Walls and Smith, 1998). To avoid unproductive debate, an analysis tool that exposes areas of uncertainty that the decision-maker may not be aware of is needed. Risk analysis is one technique that includes uncertainty in the analysis, allowing the decision-maker to weigh the probability of any particular outcome that may occur.

1.4 INTRODUCTION TO RISK ANALYSIS

Risk analysis is a general term used to describe any qualitative and/or quantitative method for assessing the impacts of risk on decision situations. Risk analysis addresses three basic questions about risk:

- What are the possible outcomes?
- What is the probability of each outcome?
- What are the consequences of decisions based on the knowledge of the probability of each outcome? (@Risk, 1997)

Risk analysis combines probabilistic descriptions of uncertain input parameters with computer simulation to characterize the risk associated with possible outcomes (Walls and Smith, 1998). Therefore, risk analysis exposes areas of uncertainty that may be hidden in a deterministic approach to LCCA, and allows the user to predict the probability of a specific outcome.

1.5 RESEARCH OBJECTIVE

The objective of this research was to develop a risk-based model for evaluating the life-cycle costs of various flexible pavement designs and maintenance alternatives. The systems model developed for this research is a project-level, probabilistic model that considers the inherent uncertainty associated with input variables. Other features of the model include the incorporation of functional aspects (structural capacity and pavement condition) and safety (skid resistance) into the design, the inclusion of rehabilitation and preventive maintenance as preservation strategy alternatives, and the consideration of user costs in the present worth cost analysis.

The LCCA model allows the user to input a mean and coefficient of variation for variables ranging from traffic factors, design conditions, and material properties to the discount rate needed for present worth cost calculations. Furthermore, the model includes an “advanced user” option that allows the user to change default values for parameters such as accident rates, percent wet time, skid resistance prediction model parameters, unit costs of construction materials, productivity estimates, work zone schedules, accident costs, user delay costs, and vehicle operating costs.

The LCCA model output consists of probability distributions describing the total present worth cost, present worth agency costs, and present worth user costs for each preservation strategy over a specified analysis period. The probability distributions describing the present worth costs were obtained through the use of risk analysis, which enables the model to take into account the inherent uncertainty associated with each input parameter. This unique aspect of the LCCA model allows the decision-maker to assess the risk associated with each preservation strategy based on the probability of various costs that may be incurred.

Finally, a sensitivity analysis was performed to identify the input variables that have a statistically significant effect on the model output. The results of the sensitivity analysis provide important information to the pavement management system at both the network level and the project level. For example, model developers can enhance the model output by focusing on adjusting and improving the model components that were identified as having the greatest effect on the model output. In addition, since many components within the LCCA model were developed based on limited data, the sensitivity analysis provides insight to highway agencies regarding various types of data that need to be collected in greater detail. After more detailed data are collected, better, more accurate models can be developed, which will in turn enhance the LCCA model output.

1.6 SCOPE

The pavement design and preservation strategies in this research were developed for flexible pavements only. The initial pavement design approach was based on the AASHTO flexible pavement design procedure, and considers factors related to the surface, base, subbase, and subgrade layers (AASHTO, 1993). Design thicknesses for the pavement surface, base and subbase were established from the AASHTO design models. In addition, the structural overlay design approach for this research was based on the overlay design procedure presented in the AASHTO Design Guide (AASHTO, 1993).

Preservation strategies considered in this LCCA model include a rehabilitation strategy and a preventive maintenance strategy. For this research, the rehabilitation strategy includes rehabilitation treatments, i.e. structural overlays, and routine maintenance activities. A thin surface treatment may be included in the rehabilitation strategy if either the pavement condition or the skid resistance deteriorates below a user specified threshold value. The preventive maintenance strategy for this LCCA model includes preventive maintenance treatments (thin surface treatments applied to a pavement in relatively good condition at any point in time during the analysis period that the pavement condition or the skid resistance drops below its corresponding threshold value) as well as routine maintenance activities. It should be noted that the allowable pavement condition threshold value for the rehabilitation strategy is significantly less than the allowable pavement condition threshold value for the preventive maintenance strategy, in order to maintain a distinct difference between the two preservation strategy alternatives. Also, for the purposes of this research, the term “maintenance activities” is used as a generic term referring to a structural overlay or surface treatments for the rehabilitation strategy or preventive maintenance treatments for the preventive maintenance strategy.

The input variables for this LCCA are defined by probability distributions. Therefore, the inherent uncertainty associated with each input variable is considered in the analysis. However, the model does not include the uncertainty of the design models used in the analysis.

The economic evaluation is performed on a present worth basis. Therefore, all costs incurred over a specified period of time, including both agency and user costs, are discounted to the present year so that the two preservation strategy alternatives may be compared. Agency costs include the initial cost of construction, cost of treatments, cost of routine maintenance, and the salvage value (considered as a negative cost). User costs are costs incurred by the highway user and include costs associated with accidents, user delay, and vehicle operation in both normal and work zone conditions.

The agency and user costs for a particular preservation strategy are calculated for a one-mile section of a two-lane, rural road. When maintenance activities are scheduled to occur, work zones are established so that one lane is closed at a time, and a flagger is used for traffic control. In order to simplify user delay calculations, a 50/50 directional split was assumed, which is typical for low-volume, two-lane rural roads. Geometric design elements such as horizontal and vertical curves and pavement shoulders may affect accident rates as well as costs associated with pavement construction and maintenance. Therefore, the model was developed with a generic structure that allows the user to input values for the parameters affected by these geometric conditions.

A distinct feature of this LCCA model is the inclusion of skid resistance. It is recognized that certain surface aggregate types possess characteristics that directly influence skid resistance. Several state highway agencies, including the West Virginia Division of Highways (WVDOH), limit the use of certain aggregate types in the surface layer to reduce some of the detrimental effects of aggregate characteristics to skid resistance. For example, limestone is the most commonly used and readily available aggregate in West Virginia. However, due to the susceptibility of limestone to polishing and, consequently, high wet weather accident experience, WVDOH established a threshold value of 3,000 ADT (average daily traffic, in vehicles per day) for the use of skid resistant wearing courses.

The use of skid resistant aggregate (SRA) in the surface layer reduces the total life-cycle cost of a pavement if the decrease in user costs associated with the reduction of wet weather accidents is greater than the increase in agency costs associated with the SRA. The inclusion of skid resistance into the LCCA provides additional insight to the significance of the SRA requirement, including its cost effectiveness.

1.7 RESEARCH APPROACH

The research began with an extensive literature search for existing project-level pavement performance and skid resistance models, in addition to models describing the effects and costs associated with routine maintenance, preventive maintenance treatments, accidents, user delay, and vehicle operating costs. The models were summarized in a literature review presented in Chapter 2.

The approach used for developing the LCCA model for this research is given in Figure 1.3. The main components that were required for the model development are:

Figure 1.3 Flowchart of General Components in Research Approach.

- data input,
- initial pavement design,
- generation of preservation strategies,
- calculation of present worth costs over the analysis period,
- analysis of model output, and
- summary and conclusions based on the results.

Following the structure outlined in Figure 1.3, the first step of the model development was to identify the input variables and constraints. Inputs include those variables that are essential for pavement design and LCCA, such as traffic factors, design parameters, material properties, and performance criteria. Additional inputs required for the model include accident rates, productivity estimates (for estimating work zone durations) and unit costs of various materials and surface treatments. Unit costs for accidents, user delay, and vehicle operating costs are also necessary. Finally, model constraints are included, such as the pavement condition threshold values for the rehabilitation and preventive maintenance strategies, as well as the skid resistance threshold value.

The initial pavement design is established based on a general form of the AASHTO design equation for flexible pavements (AASHTO, 1993). For this research, the reliability term was omitted from the AASHTO design equation. Uncertainty was accounted for by using a risk-based approach to incorporate the variability associated with each input variable into the model. The initial pavement design model component is discussed in detail in Chapter 3.

Once the initial pavement design is established, various preservation strategies are generated, as described in Chapter 4. Preservation strategies include the rehabilitation strategy and the preventive maintenance strategy. For the rehabilitation strategy, the structural overlay design is based on the AASHTO overlay design procedure. The user selects an overlay thickness of 1.5, 2.0, or 2.5 inches, and the structural overlay is performed when the pavement deteriorates beyond a predetermined level of serviceability (AASHTO, 1993). The timing of the structural overlay is dependent on the effective structural number and the expected traffic load that the pavement must carry over the remainder of the analysis period. On the other hand, the preventive maintenance strategy includes thin surface treatments that are applied to a pavement that is in relatively good condition. The timings for the preventive maintenance treatments are derived using Monte Carlo simulation techniques and are dependent on the existing condition of the pavement and the pavement condition and skid resistance threshold values.

Chapter 5 discusses the present worth cost calculations for each preservation strategy over the analysis period. The total present worth cost of a particular strategy includes both agency costs and user costs. Agency costs consist of the initial cost of construction, the cost of treatments, and annual routine maintenance costs. User costs include costs due to accidents, delay, and vehicle operating costs. Since the variability associated with each input variable is included in the model, every cost incurred over the analysis period is characterized by a probability distribution. Thus, the total present worth costs determined by the model are also characterized by probability distributions.

After the present worth cost distribution for each preservation strategy is determined, the model output is analyzed. A comparison of the performance and cost of each strategy emphasizes desirable versus undesirable strategies. By analyzing the model output probability distributions, the decision-maker can assess the risk associated with each preservation strategy, and make logical maintenance decisions accordingly.

Chapter 6 describes the LCCA model execution in detail, beginning with initiating model execution using Microsoft® Excel, then describing the user input screens and default values, and finally illustrating the output screens and interpreting the results. The LCCA model developed in this research was coded in Visual Basic for Applications and is run as a macro using Microsoft® Excel 97 (Microsoft, 1997). Input and output screens were designed to facilitate the user's interaction with the program. Default values are provided for all input parameters, based on typical values experienced in West Virginia. The user has the option to change any or all of the values to accommodate a wide range of conditions.

Chapter 7 presents a discussion on the LCCA model validation. Since no existing LCCA models were found that contained all of the same features of the probabilistic LCCA model developed for this research, the model components were validated individually. Chapter 7 also presents conclusions from the sensitivity analysis that was performed on the model results. The sensitivity analysis identifies input variables that significantly effect the model output. This analysis provides valuable insight to highway agencies regarding the LCCA model components that should be developed to the best of the highway agency's ability in order to enhance future model results.

The final step of the research was to present conclusions based on the model output as well as the sensitivity analysis results, and to provide recommendations for model enhancement and future research. The conclusions and recommendations are presented in Chapter 8.

CHAPTER 2

LITERATURE REVIEW

An extensive literature search was conducted for project-level models related to all factors of the LCCA proposed in Chapter 1. Chapter 2 consists of a literature review, focusing on design, performance and skid resistance prediction, and cost estimation models. Section 2.1 highlights the American Association of State Highway and Transportation Officials (AASHTO) procedure for flexible pavement design. Included in this section is a detailed description of the reliability concept that was accepted by AASHTO and incorporated into the design equation. Section 2.2 summarizes several pavement performance prediction models. Skid resistance is the focus of Section 2.3, including factors that affect skid resistance as well as models for predicting skid resistance based on traffic loading and/or surface aggregate type. Section 2.4 provides performance and cost estimation models related to routine maintenance, preventive maintenance treatments, and structural overlays. Section 2.5 focuses on user cost models, such as accident prediction, user delay estimation, and vehicle operating cost (VOC) estimation. Section 2.6 provides an overview of risk analysis and Monte Carlo simulation, and then presents the risk-based model that served as a basis for the detailed LCCA developed for this research. Finally, conclusions of the literature review are documented in Section 2.7.

2.1 PAVEMENT DESIGN

The AASHTO design equations for flexible and rigid pavements were developed from data collected at the AASHO Road Test, which was conducted in Ottawa, Illinois, in the late 1950s and early 1960s. The original design equations have been modified based on theoretical considerations into the current format (AASHTO, 1993). This section outlines the AASHTO flexible pavement design procedure and provides a detailed discussion on the reliability concept that has been incorporated into the design equation.

2.1.1 The AASHTO Flexible Pavement Design Procedure

The AASHTO Guide for Design of Pavement Structures (1993), also referred to as the AASHTO Design Guide, describes a design procedure for flexible pavements. The procedure is based on identifying a flexible pavement structural number (SN) that will withstand the projected level of axle load traffic. The AASHTO design equation for flexible pavements is:

$$\log W_{18} = Z_R * S_0 + 9.36 * \log(SN + 1) - 0.20 + \frac{\log \left[\frac{\Delta PSI}{4.2 - 1.5} \right]}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 * \log M_R - 8.07 \quad (2.1)$$

where: W_{18} = predicted number of 18-kip equivalent single axle load (ESAL)

applications,

Z_R = standard normal deviate for a given reliability R,

S_0 = combined standard error of the traffic prediction and performance prediction,

SN = structural number,

ΔPSI = difference between the initial design serviceability index, p_0 , and the design terminal serviceability index, p_t , and

M_R = effective roadbed soil resilient modulus (psi).

Equation 2.1 is solved in an iterative manner to find the structural number that provides the required level of traffic prediction, W_{18} . Once the required structural number is known, the layer thicknesses are determined to satisfy the equation:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad (2.2)$$

where: a_1, a_2, a_3 = layer coefficient for surface, base, and subbase courses, respectively,

D_1, D_2, D_3 = layer thickness (in inches) of surface, base, and subbase courses, respectively, and

m_2, m_3 = drainage coefficient for base and subbase layers, respectively.

The SN equation does not have a unique solution: many combinations of layer thicknesses may yield satisfactory solutions. Therefore, cost effectiveness and construction and maintenance constraints must be considered when selecting layer thicknesses in order to avoid the possibility of creating an impractical design (AASHTO, 1993). Since it is generally impractical and uneconomical to place surface, base, or subbase courses of less than some minimum thickness, the AASHTO Design Guide (1993) provides minimum practical thicknesses of asphalt

surface and aggregate base for an expected traffic load. The minimum thicknesses recommended by AASHTO are shown in Table 2.1.

Table 2.1 Minimum Thickness of Pavement Layers (in inches) (AASHTO, 1993).

Traffic, ESALs	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0 (or surface treatment)	4.0
50,001 – 150,000	2.0	4.0
150,001 – 500,000	2.5	4.0
500,001 – 2,000,000	3.0	6.0
2,000,001 – 7,000,000	3.5	6.0
Greater than 7,000,000	4.0	6.0

The procedure for thickness design follows the structure of Figure 2.1, where E_i , a_i , and D_i correspond to the resilient modulus, layer coefficient, and thickness of layer i (Huang, 1993). First, E_2 is substituted for M_R and the structural number SN_1 required to protect the base is determined using Equation 2.1. Then the thickness of layer 1 is computed by Equation 2.3:

$$D_1 \geq \frac{SN_1}{a_1} \quad (2.3)$$

Next, the structural number SN_2 required to protect the subbase is determined in the same manner as above, except that E_3 is substituted for M_R . The thickness of layer 2 is computed using:

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2} \quad (2.4)$$

Finally, the total structural number (SN_3) of the pavement is determined using M_R , and the thickness of layer 3 is computed by:

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 D_2 m_2}{a_3 m_3} \quad (2.5)$$

The general procedure for determining an initial pavement design, as outlined above, is relatively simple. The reliability factor that is included in Equation 2.1 (the term $Z_R * S_o$, where Z_R is the standard normal deviate for a given reliability R , and S_o is the overall standard deviation) introduces probability theory into the procedure, and thus requires further explanation.

Figure 2.1 Procedure for Determining Layer Thicknesses Using a Layered Analysis Approach.

2.1.2 Reliability

In 1971, Lemer and Moavenzadeh contemplated the uncertainty involved in all aspects of the pavement design process, from planning and design to construction, operation, and maintenance. The authors discussed the significance of including reliability as a design parameter and recognized that this inclusion into the design process had the potential to produce economically efficient pavements. Reliability was incorporated into the AASHTO Guide for Design of Pavement Structures (1986) using the concepts developed by Irick, Hudson, and McCullough (Irick, et al. 1987). The remainder of this section describes the reliability concept as presented in the AASHTO Design Guide (1993) and Irick, et al. (1987).

Pavement design methods can be either deterministic or probabilistic. In a deterministic design method, the designer typically assigns a factor of safety to those parameters that are

uncertain or have a significant effect on the final design. However, this traditional design approach may result in over-design or under-design, depending on the magnitudes of the safety factors applied and the sensitivity of the design procedures (Huang, 1993). In a probabilistic pavement design method, each design parameter is described by a probability distribution, and the reliability of the design can then be evaluated.

Huang (1993) summarized several standard deviations or coefficients of variation that have been used in the past to define probability distributions for various traffic and design parameters. As a note, the coefficient of variation is a percent that is equal to the standard deviation divided by the mean, and multiplied by 100 percent. The estimated standard deviations of layer thicknesses for four different paving materials are shown in Table 2.2. The estimated coefficients of variation for design period traffic prediction and for performance prediction of flexible pavements are presented in Tables 2.3 and 2.4, respectively.

Table 2.2 Standard Deviations of Layer Thickness for Flexible Pavements (Huang, 1993, referenced from Darter, et al., 1973).

Material	Standard Deviation (inches)
Hot mix asphalt	0.41
Cement-treated base	0.68
Aggregate base	0.79
Aggregate subbase	1.25

Table 2.3 Coefficients of Variation for Design Period Traffic Prediction (Huang, 1993, referenced from AASHTO, 1985).

Description	Symbol	Coefficient of Variation (percent)
Summation of EALF over % axle distribution	$\Sigma p_i F_i$	35
Initial average daily traffic	ADT_0	15
Traffic growth factor	G	10
Percentage of trucks	T	10
Average number of axles per truck	A	10
Overall traffic prediction		42

Table 2.4 Coefficients of Variation for Performance Prediction of Flexible Pavements (Huang, 1993, referenced from AASHTO, 1985).

Description	Symbol	Coefficient of Variation (percent)
Initial serviceability index	p_0	6.7
Surface strength factor	a_1	10.0
Surface thickness	D_1	10.0
Base strength factor	a_2	14.3
Base drainage factor	m_2	10.0
Base thickness	D_2	10.0
Subbase strength factor	a_3	18.2
Subbase drainage factor	m_3	10.0
Subbase thickness	D_3	10.0
Subgrade resilient modulus	M_R	15.0

The reliability of a pavement design is defined in general terms as “the probability that the design will perform its intended function over its design life (or time) and under the conditions (or environment) encountered during operation” (AASHTO, 1993). One general method for evaluating the reliability of a pavement design is to use traffic, or the number of load repetitions, as the design objective. This method is described in the following section.

2.1.3 Evaluating Reliability Based on Traffic Loading As a Failure Criterion

The method for evaluating reliability based on traffic loading as a failure criterion requires the prediction of two types of load repetitions in the design process. Traffic prediction refers to the number of load repetitions that will be applied to the pavement over the design period (w_T). Performance prediction refers to the estimated number of load repetitions that the pavement structure can carry over the design period (W_i). Note that W_i is equivalent to the dependent variable W_{18} of the AASHTO equation before reliability was added to the equation. In a deterministic design method, both w_T and W_i are unique values. In a probabilistic design method, w_T and W_i are represented by distributions.

Traffic prediction during the design period may be determined from the following equation:

$$w_T = (ADT)(\%T)(TF)(365)(DD)(LD)\left(\frac{(1+G)^Y - 1}{G}\right) \quad (2.6)$$

where: w_T = predicted 18-kip ESALs during the design period,

ADT = average daily traffic at the start of the design period,

$\%T$ = percentage of trucks in ADT (percent),

TF = truck factor,

DD = percentage of ADT in the design direction (percent),

LD = percentage of ADT in the design lane (percent),

G = growth rate per year (percent), and

Y = design period for initial pavement (years).

$\text{Log}(w_T)$ is calculated by applying the logarithmic function to both sides of Equation 2.6.

Performance prediction may be calculated from a general form of the AASHTO flexible pavement design equation:

$$\log W_t = 9.36 \cdot \log(SN+1) - 0.20 + \frac{\log\left[\frac{p_0 - p_t}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \cdot \log M_R - 8.07 \quad (2.7)$$

where: W_t = allowable ESALs during the design period,

SN = structural number,

p_0 = initial serviceability index,

p_t = terminal serviceability index, and

M_R = effective roadbed soil resilient modulus (psi).

In addition to the two values predicted during design, the method for evaluating reliability based on traffic loading as a failure criterion also requires the measured values of the two types of load repetitions during the pavement's service life. The actual number of ESALs applied to the pavement over the design period is designated as N_T . The actual number of ESALs carried by the pavement structure before reaching terminal serviceability is designated as N_r . Figure 2.2 illustrates the predicted and measured values.

Figure 2.2 Predicted and Observed Traffic Loading in the Pavement Design-Performance Process (Irick, et al., 1987).

2.1.3.1 The Pavement Design-Performance Process

The reliability of a pavement design is based on a repetition of events that are referred to as the pavement design-performance process. The following assumption was made by Irick, et al. (1987):

It is assumed that the design-performance process begins with a well-specified design equation (or algorithm) for the design of a highway project, proceeds into specified construction and quality control procedures, moves on to treatments by environmental and traffic factors, includes normal maintenance procedures, implies the observation of a performance criterion (d), and finally moves into the rehabilitation stage when $d = d_t$.

In the above assumption, d represents the pavement condition at any point during its performance period, and d_t represents the terminal distress, or the pavement serviceability at which the current phase of the pavement's life cycle is to be terminated.

Each iteration of the design-performance process yields predictions for traffic and performance (w_T and W_t) at the design stage. At the end of the design and performance periods, corresponding values for actual traffic and performance (N_T and N_t) can be obtained.

At this point, Figure 2.3 may be used as an aid in describing the concept of reliability. The notation $\delta(N_T, N_t)$ is used to represent the distance from N_T to N_t . Based on this figure, reliability is defined as (Irick, et al., 1987):

$$R(d) = \text{Prob}(\log N_t \geq \log N_T) = \text{Prob}[(\log N_t - \log N_T) \geq 0] \quad (2.8)$$

In essence, Equation 2.8 states that the reliability of a design is equal to the probability that the number of axle repetitions a pavement carries before failure is greater than the number of axle repetitions applied to the pavement during the design period.

2.1.3.2 Traffic and Performance Prediction Errors

The traffic prediction and performance prediction errors are determined using Equations 2.9 and 2.10, respectively.

$$\delta(N_T, w_T) = \log(w_T) - \log(N_T) \quad (2.9)$$

$$\delta(W_t, N_t) = \log(W_t) - \log(N_t) \quad (2.10)$$

Figure 2.3 Basic Points and Deviations for Design-Performance Reliability (AASHTO, 1993).

These equations define the differences between traffic predictions during design, w_T and W_t , and observations of actual pavement performance, N_T and N_t . For any iteration of the design-performance process, Equations 2.9 and 2.10 are equally likely to be both positive, both negative, or one positive and one negative. Therefore, the distributions describing Equations 2.9 and 2.10 have means of zero (positive errors for some iterations are balanced out by negative errors from other iterations), and thus are completely defined by their variances (or standard deviations). The variance of the traffic prediction error distribution is denoted by S_w^2 and the variance of the performance prediction error distribution is denoted by S_N^2 .

The reliability factor, F_R , is defined as the ratio of predicted design applications to predicted traffic:

$$F_R = \frac{W_t}{w_T} \quad \text{or} \quad \log F_R = \log W_t - \log w_T \quad (2.11)$$

Figure 2.3 illustrates how $\log F_R$, designated as $\delta(w_T, W_t)$, acts as a “spacer” between the error distributions. If the performance error distribution is forced a positive distance from the traffic error distribution, the designer can be assured that N_t will be at least as large as N_T , which is the goal of pavement design.

2.1.3.3 The Overall Process Deviation

The overall process deviation is defined as the total deviation from $\log N_T$ to $\log N_t$ for each design-performance process iteration (Irick, et al., 1987). The overall process deviation, δ_0 , is shown in Figure 2.3 and is defined by:

$$\begin{aligned} \delta_0 &= \log N_t - \log N_T \\ &= (\log w_T - \log N_T) + (\log W_t - \log w_T) + (\log N_t - \log W_t) \end{aligned} \quad (2.12)$$

As described previously, the first and third terms of Equation 2.12 are defined by normal probability distributions with mean values of zero and variances of S_w^2 and S_N^2 , respectively. The middle term, $\log F_R$, is a constant with no variance. Therefore, δ_0 has a mean equal to $\log F_R$ and a variance equal to the summation of S_w^2 and S_N^2 .

Reliability can then defined in terms of δ_0 as:

$$R(d) = \text{Prob}(\delta_0 \geq 0) \quad (2.13)$$

Therefore, the probability that a design will perform its intended function over its design life is equal to the area under the $\delta(N_i, N_T)$ curve where a specific value, δ_0 , is greater than zero. Figure 2.4 illustrates this concept. For example, if the design period traffic prediction (w_T) is equal to W_i , then the reliability factor, F_R , is one and $\log F_R$ equals zero. Thus, the $\delta(N_i, N_T)$ probability distribution has a mean of zero, and the reliability, defined in Equation 2.13 as the probability of obtaining a specific value δ_0 in the $\delta(N_i, N_T)$ distribution that is greater than zero, is 50 percent. Therefore, the probability of failure is 50 percent.

In order to assure a higher reliability for the design-performance process, the designer must design the pavement structure so that W_i is greater than the predicted number of design period applications (w_T). The difference between $\log w_T$ and $\log W_i$ is shown in Figure 2.3 as $\log F_R$. The value of $\log F_R$ can be determined by transforming δ_0 to a standard normal deviate, using Equation 2.14. This transformation is shown in Figure 2.4.

$$z = \frac{\delta_0 - \bar{\delta}_0}{S_0} = \frac{\delta_0 - \log F_R}{S_0} \quad (2.14)$$

At the point where δ_0 is zero, z is defined as Z_R , where:

$$Z_R = \frac{0 - \log F_R}{S_0} \quad (2.15)$$

For a given reliability level, Z_R can be found in the standard normal curve area tables. For example, for a reliability of 95 percent, the corresponding standard normal deviate, Z_R , is -1.645 . Finally, Equation 2.16 is regarded as an algebraic definition for the reliability design factor.

$$\log F_R = -Z_R * S_0 \quad \text{or} \quad F_R = 10^{-Z_R * S_0} \quad (2.16)$$

Figure 2.4 Definition of Reliability (AASHTO, 1993).

2.1.4 Applying Reliability to Pavement Design

The theory behind the reliability concept is based on the four basic points defined in Section 2.1.3; the traffic prediction, performance prediction, actual traffic, and actual performance. However, at the design stage, the actual traffic and performance are not known. Therefore, Irick, et al. (1987) provided the following six steps for applying the above reliability concept to pavement design.

- Select a performance criterion (such as d = serviceability loss) and a corresponding performance prediction equation (such as Equation 2.7).
- Select values for environmental factors, soil factors, and traffic load factors and substitute the values into the design equation.
- Select a design period and a design period traffic prediction algorithm (such as Equation 2.6). Derive a design period traffic prediction (w_T).
- Select a reliability level, R , assume a process standard deviation, S_o (AASHTO recommends assuming a value of $S_o = 0.45$ for flexible pavement design), and look up the reliability factor, F_R , in a standard normal curve area table.
- Calculate the design applications, W_i , using Equation 2.11 and then substitute W_i in the design equation (Equation 2.7).
- Calculate alternative designs and select the optimum design.

2.2 PAVEMENT PERFORMANCE

Pavement performance models are used in pavement design as well as pavement management systems to forecast future pavement performance. Chen, et al. (1995) reported that pavement performance prediction is “the most technologically difficult portion of pavement management.” The authors noted several factors that contribute to the complexity of pavement performance prediction, such as:

- the uncertainty of pavement behavior under variable traffic loads, environments, etc.,
- the difficulty of quantifying many factors affecting pavement performance,
- the error associated with using discrete testing points to represent the total pavement area when estimating pavement condition, and
- the nature of the subjective condition survey.

The literature search revealed numerous forms of performance prediction models, including both deterministic models and probabilistic models. However, the literature showed that the performance of many of these models has been poor, regardless of their simplicity or complexity.

As mentioned in the previous section, the AASHTO flexible pavement design equation (Equation 2.1) is typically used for thickness design (AASHTO, 1993). However, this equation can also be used to predict the performance of a particular pavement design for a given traffic loading over a specified period of time t . For example, the PSI at time t , denoted as PSI_t , can be estimated by re-writing ΔPSI as $(PSI_{initial} - PSI_t)$, and solving the equation for PSI_t .

Lee, et al. (1993) used nonlinear regression analysis to develop models for predicting the present serviceability rating¹ (PSR) of a pavement for various pavement types. Lee, et al. defined the PSR as the average of subjective user evaluations describing the serviceability of a pavement. The model for predicting the PSR of a new or existing flexible pavement is:

$$PSR = PSR_i - 14.2889 * STR^{-1.8720} * AGE^{0.3499} * CESAL^{0.3385} \quad (2.17)$$

where: PSR_i = initial value of PSR at construction,

STR = structural number (of existing pavement structure),

AGE = age of pavement since construction or major rehabilitation (years), and

$CESAL$ = cumulative 18-kip equivalent single-axle loads applied to pavement in the heaviest traffic lane (millions).

The above model was developed using the serviceability records of flexible pavements from the original AASHO Road Test. The correlation coefficient (R^2) was 0.52, and the standard error of the estimate (SEE) was 0.45.

Adjustment factors were then introduced to adjust the rate of deterioration of PSR for different climatic zones (wet, intermediate, or dry) and functional groups (interstate highways and other principal arterials or minor arterials and collectors). For example, for a flexible pavement in group j (a particular climatic zone and functional group), the proposed model with the adjustment factor added in becomes:

¹ According to the terminology used at the AASHO Road Test, Lee, et al. (1993) actually developed a model for predicting the Present Serviceability Index (PSI) rather than the PSR .

$$PSR_j = PSR_I - AF_j * \left(14.2889 * STR^{-1.8720} * (C_{1j} + \Delta YEAR)^{0.3499} * (C_{2j} + \Delta ESAL)^{0.3385} \right) \quad (2.18)$$

$$C_{1j} = \frac{PSR_I - PSR_j}{AF_j * \left(14.2889 * STR^{-1.8720} * ESALPYR^{0.3385} \right)} \quad (2.19)$$

$$C_{2j} = C_{1j} * ESALPYR \quad (2.20)$$

where: PSR_j = PSR for a new or existing pavement in pavement group j ,

AF_j = adjustment factor for pavement group j ,

$\Delta YEAR$ = change in age of pavement (years),

$\Delta ESAL$ = change in cumulative ESALs (millions),

$ESALPYR$ = current yearly ESALs (millions), and

PSR_I = current year pavement condition.

As a reference, for a flexible pavement in an intermediate climatic zone with freeze-thaw and a functional class of minor arterials and collectors, the recommended adjustment factor AF_j for the proposed model is 0.71. Therefore, the actual rate of PSR loss in this pavement group is less than the rate predicted by the original model based on AASHO Road Test conditions.

Al-Mansour and Sinha (1994) developed a relationship between pavement serviceability index (PSI) and pavement age. The best-fit model is in the following form:

$$PSI = a + (b * Age) \quad (2.21)$$

where: PSI = present serviceability index,

Age = pavement age (in years) since construction or last resurfacing, and

a, b = estimated regression parameters.

The estimated regression parameters vary based upon the following factors; north/south climate region, no maintenance/basic routine maintenance, and high/low traffic volume. The estimated regression parameters provided by the authors, as well as the number of observations and correlation coefficients (R^2), are shown in Table 2.5.

Table 2.5 Estimated Regression Parameters of Pavement Condition Prediction Model (Al-Mansour and Sinha, 1994).

Climate Region	Maintenance Category	No. of Observ	R ²	Estimated Parameters	
				<i>a</i>	<i>b</i>
North	No Maintenance	13	0.4797	3.8816	-0.1051
	Basic Routine Maintenance				
	• High Traffic (AADT > 2000)	33	0.4127	3.9732	-0.0885
	• Low Traffic (AADT ≤ 2000)	43	0.5403	4.1523	-0.0817
South	No Maintenance	45	0.5407	4.0135	-0.0978
	Basic Routine Maintenance				
	• High Traffic (AADT > 2000)	48	0.5822	4.2315	-0.0915
	• Low Traffic (AADT ≤ 2000)	102	0.4081	4.0736	-0.0773

2.3 SKID RESISTANCE

Skid resistance is a concern for highway agencies and pavement designers when considering pavement safety. Although the primary interest of the designer is the strength and durability of the pavement, frictional characteristics may deteriorate faster than other pavement characteristics, posing a potential safety hazard (Smith and Fager, 1991).

2.3.1 General Definition and Factors that Affect Skid Resistance

Skid resistance is defined as the force that resists the sliding of tires on a pavement when the tires are prevented from rotating (Irick, 1972). Mathematically, the skid number (SN) is often used to describe the skid resistance of a pavement and is defined as:

$$SN = 100 * \frac{F}{L} \quad (2.22)$$

where: *SN* = skid number,

F = frictional resistance to motion at the pavement surface, and

L = load normal to the pavement surface.

Skid resistance depends on vehicle and operational parameters in addition to pavement condition. ASTM E-274 fixes the non-pavement parameters so the results of the test can be associated with pavement characteristics. In the following discussion it is assumed that standard test methods are followed so skid number (SN) values are attributed to pavement characteristics.

The skid number, SN, is typically measured using a locked-wheel skid trailer operated in accordance with ASTM E-274. When the skid testing is conducted at 40 miles per hour, the result is referred to as SN_{40} . Some agencies and authors use friction number, FN, in lieu of SN. Threshold values for minimum acceptable skid number varies between state highway agencies. Table 2.6 summarizes a recently conducted survey of agency practices (Ksaibati, et al., 1996).

Table 2.6 Skid Number Thresholds for Rehabilitation (Ksaibati, et al., 1996).

SKID NUMBER THRESHOLDS ¹	STATE HIGHWAY AGENCY
≤ 43	AZ ²
≤ 40	ID, NC ³
≤ 39	NE
≤ 37	NV, OR
≤ 35	MD, MS, OH, UT, WY
≤ 30	IN
≤ 25	WA

¹ SN_{40} unless noted.

² Measured with Mu Meter and calibrated to same standard as locked wheel skid trailer.

³ SN measured at 45 mph.

Based on a study of skid resistance and wet weather accidents in the United States, Smith (1976) recommended a minimum skid number (SN_{40}) of 37 and 41 for mean traffic speeds of 50 and 60 mph, respectively. In Virginia, Runkle and Mahone (1977) proposed a friction number (FN) of 30 as a minimum guideline value for interstate and other divided highways, and an FN of 40 as a minimum guideline value for two-lane highway sites. Burchett and Rizenbergs (1982) documented that new pavements in Kentucky should be designed so that the SN_{40} , at -2.5 standard deviations (99.4 percent assurance), remains above 32. In Ontario, Kamal and Gartshore

(1982) reported that for 2-lane roads with a speed limit of 50 mph, SN_{50} (SN measured at 50 mph) values of 32 or more are good, SN values ranging from 27 to 31 are borderline, and SN values less than 27 are low.

Several factors affect the skid resistance of a pavement, including environmental conditions, traffic loading, vehicle factors, and pavement factors. These factors are described in the following sections.

2.3.1.1 Environmental Conditions

Environmental conditions have both long- and short-term effects on the skid resistance of a pavement. Seasonal variations are an example of the long-term effects of climate on skid resistance. Rain, snow, and ice are factors that contribute to short-term variations of skid resistance. For example, the skid resistance of a flexible pavement is relatively high on a hot, dry, summer afternoon in West Virginia. However, during this time, oil from motor vehicles and the asphalt binder may accumulate on the pavement surface. Therefore, when rainfall begins, skid resistance diminishes almost instantaneously until the oil is washed off of the pavement surface. Once the pavement is washed clean, the level of skid resistance may slightly increase until it reaches a steady state of reduced friction during rainfall. After the pavement dries, the skid resistance returns to a high level. The significance of wet pavements in accident experience is discussed in Section 2.5.1.2.

2.3.1.2 Traffic Loading

Traffic loading is another significant factor that causes a change in skid resistance over time. As vehicles pass over the pavement, tires wear down the surface, resulting in rutting and polishing, dislocation, or reorientation of aggregates. All of these conditions have been recognized as possible contributors to a reduction in skid resistance.

2.3.1.3 Vehicle Factors

Vehicle factors affecting skid resistance include speed, tire pressure, wheel load, and tire treads. In general, friction decreases as speed, tire pressure, and/or wheel loads increase, particularly on wet surfaces. The combination of high speeds and wet pavements can lead to hydroplaning, which occurs when water cannot escape fast enough from the tire-pavement interface. Thus, the tire and the pavement are separated by a thin film of water. At this time, pavement surface factors lose all of their skid resistant qualities. The chances of hydroplaning can be reduced by improving drainage capabilities of the pavement structure and/or reducing vehicle speed. Tire treads are another important factor because they continuously change as they wear. After the tread is worn away, tires develop more friction on dry pavements because more rubber comes into contact with the pavement. However, when the pavement becomes wet, the

friction diminishes with tread wear because the tire cannot expel water from the contact area through the treads (Irick, 1972).

2.3.1.4 Pavement Factors

Pavement factors affecting skid resistance are drainage and surface characteristics. A good drainage system with appropriate cross slopes is also necessary in order to provide rapid removal of water from the pavement surface, thereby reducing the risk of hydroplaning. The most common surface characteristics that affect skid resistance are texture, polishing of surface aggregates, bleeding, rutting, and contamination.

Surface texture is most significantly influenced by aggregate size and shape, and is defined in terms of microtexture and macrotexture. Surface microtexture is attained from the fine, hard grains in the surface of coarse or fine aggregate. Microtexture provides a gritty surface to penetrate thin water films and produce skid resistance through good friction between the tire-pavement interface. Microtexture contributes to frictional resistance at all speeds, but is a dominating influence at speeds less than 30 mph (Roberts, et al., 1996). Microtexture depends largely on the mineral composition and roughness of the aggregates, and thus is achieved by selecting good quality, polish-resistant aggregates. Surface macrotexture provides drainage channels for the removal of water between the tire and the roadway, which allows better tire contact with the pavement to improve frictional resistance and prevent hydroplaning. It is less important at lower speeds, but is essential at high speeds in wet conditions (Shahin, 1994). Initial macrotexture depends on the specific type of mix, the aggregate gradation, and the stability of the mix (Jayawickrama, et al., 1996).

Polishing is the loss of surface microtexture, which results in the smoothing and rounding of exposed aggregates. Thus, skid resistance diminishes as a result of polishing. When surface aggregates become smooth, friction between the pavement and vehicle tires is considerably reduced during wet weather, and the pavement may become dangerously slippery.

Some aggregate types tend to polish more readily than others, thereby decreasing surface texture more rapidly than other aggregate types. Carbonate rocks such as limestone, primarily composed of calcite, and dolomite, primarily composed of calcium and magnesium, tend to have a high susceptibility to polishing. On the other hand, gravel and slag are two aggregate types that have a low tendency to polish. For this research, the term “skid resistant aggregate,” or SRA, includes all aggregates with acceptable polishing characteristics.

Bleeding occurs on bituminous pavements when the asphalt binder fills the air voids of the mix and expands to form a thin film on the pavement surface. Bleeding is a result of excessive binder content, excess application of a bituminous sealant, or low air void content. It

occurs at high temperatures, and is not reversible in cold weather. Therefore, the film of asphalt accumulates on the pavement surface, obscuring the effectiveness of the skid resistant qualities of the aggregate and resulting in a significant loss of skid resistance when the pavement becomes wet.

Rutting is a term used to describe permanent deformation of a pavement in the wheel paths. The permanent deformation can occur in any of the pavement layers or subgrades, and is typically the result of consolidation or lateral movement of materials due to repeated traffic loading. Studded tires can wear away the pavement material in the wheel paths producing the functional equivalent of a rut. Rutting is most noticeable to the driver after rainfall ends, when the ruts remain filled with water while the pavement surface begins to dry. The excess water in the wheel paths can lead to hydroplaning at lower speeds, as well as increase splash and spray, all of which are potential hazards to drivers.

While contamination is not a permanent surface characteristic such as the factors previously mentioned, it is nonetheless an important pavement factor that may significantly affect skid resistance. Rubber, oil, and water are some of the more common contaminants that are found on roadways. It has been reported that when contamination is present, such as a thin film of oil, the tire-pavement interface will be lubricated, thus reducing tire-pavement friction whether or not the pavement is wet. However, when water is added to such a surface, the skid resistance diminishes significantly (Irick, 1972).

2.3.2 Skid Resistance Models

When a polished, bleeding, rutted, or contaminated pavement surface becomes wet, there is a significant loss of friction between the vehicle tire and the pavement surface. Such a situation may result in a high number of accidents, which is not only undesirable, but also unacceptable to the pavement designer who is responsible for building as much safety as possible into the pavement. Further information regarding wet weather accidents is found in Section 2.5.1.2.

The importance of a safe design is reflected in the abundance of literature in the area of skid resistant pavement surfaces. Many researchers have developed models for predicting the skid resistance of a pavement over time for a given aggregate type or surface type. The remainder of this section describes several of these models.

In 1975, Quinn investigated skid resistant characteristics of carbonate rock aggregates. The results of the research led to a recommendation that carbonate rock be banned from use in

surface course bituminous concrete pavements in New Jersey, due to its susceptibility to polishing under repeated traffic loading.

One factor that was considered before making the recommendation was the number of vehicle passes which various aggregates could sustain without polishing to an unacceptable level of skid resistance. Regression analysis was performed on 80 locations consisting of trap rock aggregate, 50 locations consisting of carbonate rock, and 33 sites consisting of gneiss as the coarse aggregate. Skid number (SN) was the dependent variable and traffic was the independent variable. The regression line for carbonate rock showed a significant deterioration of SN with polishing under traffic. This relationship was not seen at all with trap rock and was seen to a much lesser extent on the gneiss sections.

The regression line presented in the report for carbonate aggregates has the following linear approximation:

$$SN_{40} = (-0.3 * v) + 40.5 \quad (2.23)$$

where: SN_{40} = skid number at 40 mph, and

v = number of cumulative vehicle passes (in millions).

The report states that the variability of data, along with the limited number of sites, does not allow the use of this regression curve as a predictor, but that it does illustrate the general polishing trend. Quinn (1975) concluded that the use of carbonate rock aggregate in the surface course does imply a rapid decay of SN with cumulative vehicle passes.

In Illinois, Dierstein (1977) studied pavements associated with frequent wet weather accidents. Eighty-five percent of the 226 sites selected for this study had a friction number (FN) at 40 mph of less than 33. This finding emphasized the need for providing skid resistant pavements. Another phase of this research involved the development of wear curves for pavement surfaces consisting of various aggregate types. The wear curves were based on cumulative axles in millions, and friction number at 40 mph. The wear curve for gravel shows that the friction number should remain above 40, regardless of traffic volumes. However, the wear curves for dolomite and limestone show a loss in pavement friction as axle applications increase. After 14 million cumulative axle applications, the minimum FN for dolomite is expected to be around 40, while the minimum FN for limestone is expected to be slightly less than 30. No equations or correlation coefficients were provided by the authors.

Emery, et al. (1982) developed models for predicting anticipated skid numbers at the design stage, based on aggregate and mix properties and projected traffic. Models were developed for dense graded and open graded surface course mixes with high traffic volumes and low traffic volumes. For low traffic volume roads, a total of 17 pavement sections were constructed and observed for use in developing the predictive models. The number of dense graded and open graded sections were not reported. Both low traffic volume models were developed based on Spring and Fall test data combined together. The prediction models for dense graded friction course mixes with low traffic volumes were:

$$SN_{80} = 2.155(MS) + 0.192(FLOW) + 4.418(VOID) - 8.57 \quad (2.24)$$

$$SN_{50} = 1.024(SN_{80}) + 6.239 \quad (2.25)$$

where: SN_{80} = skid number at 80 km/h (50 mph),

SN_{50} = skid number at 50 km/h (30 mph),

MS = Marshall stability (kN),

$FLOW$ = Marshall flow (0.25 mm), and

$VOID$ = percent air voids in the mix.

The multiple R coefficient (R) for SN_{80} was 0.876, and the R for SN_{50} was 0.822.

The prediction models for open graded friction course mixes were:

$$SN_{80} = 0.196(MS) + 5.472(VOID) + 37.320(EQT(4))^{0.016} - 40.32 \quad (2.26)$$

$$EQT(4) = 3 \times 10^5 \left(1 + \frac{4-1}{100}(COMM) \right) (AGE)(AADT) \quad (2.27)$$

$$SN_{50} = 0.953(SN_{80}) + 7.338 \quad (2.28)$$

where: $EQT(4)$ = equivalent traffic with a commercial vehicle equivalence factor of 4,

$COMM$ = percent commercial vehicles,

AGE = service life of the pavement (months), and

$AADT$ = annual average daily traffic.

The EQT(4) variable in this model is not universally known, and the term “commercial vehicle equivalence factor of 4” was not clearly defined in the literature. However, Emery, et al. (1982) reported that the R value for the open graded friction course SN_{80} model was 0.865, and the R for SN_{50} was 0.772. The authors note that while there is “fair confidence” in the dense graded, high traffic volume models for various aggregate types, the models for open graded surface course mixes for high and low traffic volumes are considered to be preliminary due to limited data.

Burchett and Rizenbergs (1982) studied the effects of frictional performance of pavements in Kentucky. The authors used regression analysis to develop logarithmic relationships between skid resistance and cumulative traffic. Best-fit equations were derived for various types of pavements based on effective AADT, which is the average number of vehicles per day that traverse the pavement. Best-fit curves representing a lower limit of –2.5 standard deviations were also plotted for various pavement types and effective AADT ranges. These curves represent SN levels that should be exceeded by 99.4 percent of the measured SNs, and thus provide an indication of worst-case performance. For Class I bituminous US and KY rural roads with effective AADT of 1,000 to 2,499 and 2,499 to 34,000, the best-fit equations are:

$$SN_{40} = 42.3 + (-4.3 * \log(CT)) \text{ for AADT range of 1,000 to 2,499} \quad (2.29)$$

$$SN_{40} = 40.2 + (-2.0 * \log(CT)) \text{ for AADT range of 2,500 to 34,000} \quad (2.30)$$

where: SN_{40} = skid number at 40 mph, and

CT = cumulative traffic in millions of vehicle passes.

Ninety-nine data points were used in the development of the equations for the lower AADT range, and 132 data points were used to develop the equations for the higher AADT range. The corresponding equations for the lower limits at –2.5 standard deviations are:

$$SN_{40} = 28.2 + (+1.0 * \log(CT)) \text{ for AADT range of 1,000 to 2,499} \quad (2.31)$$

$$SN_{40} = 25.8 + (-3.1 * \log(CT)) \text{ for AADT range of 2,500 to 34,000} \quad (2.32)$$

where: SN_{40} = skid number at 40 mph, and

CT = cumulative traffic in millions of vehicle passes.

Skerrit (1993) analyzed pavement friction data to determine the adequacy of New York's high-friction aggregate (HFA) specification established in 1970. All friction testing was performed using the skid trailer at 40 mph. A friction number (FN) of 32 or higher was considered adequate. This number was chosen as a minimum design target value because it corresponds to the friction assumed by AASHTO in calculating stopping distances. Pavement sections that were tested consisted of homogeneous aggregates, sandy rock types, and aggregate blends. Results of this analysis were presented in the form of graphs with the data points and fitted curves. No equations or correlation coefficients were provided.

Homogeneous aggregates included traprock, granite, limestone, and dolomite. Since the NYDOT HFA specification did not allow the use of limestone alone as the coarse aggregate, no such sites were available for testing. For traprock and granite, a linear regression line was fitted to lane annual average daily traffic (LAADT) versus friction number data, showing that these aggregate types were expected to provide adequate friction ($FN \geq 32$) up to 33,000 LAADT. Seventeen data points (10 sites with traprock and 7 sites with granite) were used in the analysis. The linear regression model for Wappinger dolomite, based on data from 9 sites, showed this aggregate could provide adequate friction for pavement lanes carrying less than 4,000 vehicles daily.

Sandy rock types included sandstone, siltstone, quartzite, siliceous limestone, and siliceous dolomite. Results showed that for sandstone, siltstone, and quartzite, 90 percent of all data (based on 15 data points) remained above an average FN of 50, regardless of traffic volume. For siliceous limestone and siliceous dolomite, 90 percent of all data (based on 40 data points) remained above an average FN of 40, regardless of traffic volume.

Aggregate blends consisted of two or more rock types, both carbonate and noncarbonate. The NYDOT HFA specification requires that aggregate blends must consist of a minimum of 20 percent noncarbonate particles, and at least 20 percent of the plus $\frac{1}{4}$ inch particles must be noncarbonate. Noncarbonate particles must have an acid insoluble residue content of 80 percent or higher. The regression analysis was performed using 82 data points, including two sites that contained less than 20 percent noncarbonate particles. Results showed that aggregate blends meeting the HFA specification provided adequate friction for pavements with traffic volumes up to 10,000 LAADT.

2.4 PRESERVATION STRATEGIES

Since the highway system in the United States is largely in place today, the selection of cost effective preservation strategies is crucial to highway agencies. A preservation strategy is a combination of treatments selected to optimize pavement cost effectiveness. Treatments include routine maintenance, preventive maintenance, rehabilitation, and reconstruction. The types of treatments are well defined in the literature. However, the effect of these treatments on pavement performance is difficult to quantify. The purpose of this section is to identify existing models that are used to describe the effects of preservation treatments on the performance of pavements, as well as to identify existing models related to the cost or duration of routine maintenance, preventive maintenance, and rehabilitation.

2.4.1 Routine Maintenance

As mentioned previously, routine maintenance is performed on pavements to correct deficiencies. Typical routine maintenance activities include crack sealing, patching, basic shoulder maintenance, and maintaining drainage structures. The extent of annual routine maintenance activities performed and their corresponding costs vary with respect to pavement condition, pavement age, traffic loads, and availability of funds. Experience shows that routine maintenance activities for a pavement in poor condition require more materials and man-hours than routine maintenance activities for a pavement in good condition.

Al-Mansour and Sinha (1994) developed routine pavement maintenance cost models based on pavement condition, climate, and traffic volumes. The logarithmic cost equation is:

$$\log(AMC) = a + (b * PSI) \quad (2.33)$$

where: AMC = annual roadway or shoulder maintenance expenditure (\$/lane-mile),
 PSI = PSI at time of maintenance, and
 a, b = estimated regression parameters.

The estimated regression parameters provided by the authors are shown in Table 2.7. This table also shows the corresponding number of observations and correlation coefficients (R^2) for each model given the estimated parameters.

Table 2.7 Estimated Regression Parameters of Annual Basic Routine Maintenance Cost Model (Al-Mansour and Sinha, 1994).

Type of Maintenance	Traffic Level	No. of Observ	R ²	Estimated Parameters	
				a	b
Roadway Maintenance	High Traffic (AADT > 2000)	55	0.5193	4.0283	-0.4621
	Low Traffic (AADT ≤ 2000)	67	0.5887	3.7781	-0.4252
Shoulder Maintenance	High Traffic (AADT > 2000)	14	0.4099	3.3221	-0.3547
	Low Traffic (AADT ≤ 2000)	27	0.5693	3.5323	-0.4573

2.4.2 Preventive Maintenance Treatments

Preventive maintenance is applied to pavements in good structural condition to extend the life of the pavement in a cost-effective manner (Zaniewski and Mamouk, 1996). The objective of such a strategy is to extend the functional life of a pavement by applying relatively inexpensive treatments before the pavement deteriorates to a condition that requires corrective maintenance. It is hypothesized that preventive maintenance is a successful strategy for providing high-quality pavements over many years at minimal cost. However, there is a lack of data to quantify pavement performance effects. As more long-term data are collected, the effects of preventive maintenance will become more apparent. The remainder of this section consists of performance and cost data for several preventive maintenance treatment alternatives, including chip seals, slurry seals, microsurfacing, and thin overlays.

One of the difficulties in evaluating the performance of preventive maintenance is distinguishing between the life of the treatment versus the effect of the treatment on the life of the pavement. The majority of the literature identifies the time between treatments as the treatment life. There is little information on the effect of the treatment on pavement life. The effect of preventive maintenance on the pavement life is extremely important for LCCA.

A framework for selecting effective preventive maintenance treatments for flexible pavements was presented by Hicks, et al. (1997). The types of preventive maintenance treatments included in this report were crack sealing, fog seals, slurry seals, microsurfacing, chip seals, and thin hot-mix overlays. The authors recommended that, when selecting the most

appropriate maintenance treatment, the cause of distress, level of distress, and traffic volume should all be considered. Furthermore, it was reported that not all forms of distress can be corrected with the preventive maintenance treatments considered in this research. The authors concluded that the use of a maintenance treatment for an uncorrectable distress type is not an effective use of funds.

Table 2.8 summarizes the preventive maintenance strategies considered by the authors to be most appropriate for various types of distress. In addition, the expected life and average unit cost of these typical preventive maintenance treatments are presented in Table 2.9.

Cost estimates for chip seals and hot-mix asphalt overlays were provided by Collura, et al. (1993). The estimates were based on 24 chip seal projects and 47 overlay projects in the New England region. The average cost of a chip seal was \$0.80/yd² with a standard deviation of \$0.32. The average cost of an overlay was \$30.36/ton with a standard deviation of \$3.88. The hot-mix asphalt overlays consisted of two categories including 0.5-inch overlays as well as 1- to 1.5-inch overlays.

Ksaibati, et al. (1996) performed an evaluation of surface treatment practices in the United States. Forty-seven states, including West Virginia, replied to a survey. A review of the replies showed that the average service life of a typical surface treatment in all states was approximately 6 years. West Virginia, along with 18 other states, reported an expected service life of a typical surface treatment less than the average of 6 years.

Table 2.8 Appropriate Maintenance Strategies for Various Types of Distress (Hicks, et al., 1997).

TYPE OF DISTRESS	Crack Sealing	Fog Seal	Micro-surfacing	Slurry Seal	Chip Seal	Thin Overlay
Roughness (nonstability related)			X			X
Roughness (stability related)						X
Rutting			X	X		X
Fatigue Cracking						
Longitudinal and Transverse Cracking	X	X	X	X	X	X
Bleeding			X			
Raveling		X	X	X	X	

Key: X = appropriate strategy

Table 2.9 Average Unit Costs and Expected Life of Typical Preventive Maintenance Treatments (modified from Hicks, et al., 1997).

TREATMENT	Cost / m ²	Cost / yd ²	Cost / lane-mile ⁸	Expected Life
Crack Treatment	\$3.29 ⁶	\$1.00 ⁷	\$7,040	2 to 3 years
Fog Seals ¹	\$0.54	\$0.45	\$3,168	3 to 4 years
Slurry Seals ²	\$1.08	\$0.90	\$6,336	4 to 6 years
Microsurfacing ³	\$1.50	\$1.25	\$8,880	5 to 7 years
Chip Seals ⁴	\$1.02	\$0.85	\$5,984	4 to 6 years
Thin HMA Overlay ⁵	\$2.09	\$1.75	\$12,320	2 to 10 years

¹ 0.2 l/m² (0.05 g/yd²) of a 1:1 dilution of CSS emulsion and water

² 7 kg/m² of ISSA Type II slurry

³ 14 kg/m² of ISSA Type II microsurfacing

⁴ 15 kg/m²

⁵ 30 to 44 mm/m²

⁶ Cost per meter

⁷ Cost per lineal foot

⁸ 12-foot lane

A review of existing literature and current practices related to preventive pavement maintenance was collected by Geoffroy (1996) and presented in *NCHRP Synthesis 223*. A survey was sent to each state, in addition to the District of Columbia, Puerto Rico, 13 Canadian agencies, and 37 local transportation agencies in the U.S. West Virginia was one of five states that did not reply to the questionnaire. Each highway agency was asked to reply to several questions regarding preventive maintenance practices as well as performance and cost data. Performance and cost data was accounted for by the highway agency circling the most appropriate range of values. Table 2.10 is an overall summary of the survey responses for selected preventive maintenance treatments. Additional performance and cost information for each treatment type is provided in the corresponding sections that follow.

2.4.2.1 Chip Seals

A chip seal is a sprayed application of asphalt binder that is immediately covered by a single layer of aggregate of uniform size and immediately rolled to seat the aggregate into the binder. Chip seals are typically used on roads with low to moderate traffic volumes. Historic use of chip seals on high volume roads was limited due to the possibility of loose chips damaging vehicles. While the performance of chip seals is mixed, it has been reported that a properly

applied chip seal can provide good performance on roads with 5,000 vehicles per day for about 4 to 7 years (Zaniewski and Mamlouk, 1996). Raza (1994) reported that chip seal performance has been mixed: some chip seals perform for many years, whereas other chip seals fail fairly early. However, generally speaking, the performance life of a chip seal varies from 3 to 6 or more years (Raza, 1994). Selected single application chip seal performance and cost data are shown in Table 2.11.

Table 2.10 Summary of the Performance of Selected Preventive Maintenance Treatments for Asphalt Concrete Pavements (Geoffroy, 1996).

TREATMENT	Pavement Age at Time of First Application (yrs)	Frequency of Application (yrs)	Observed Increase in Pavement Life (yrs)
Single Application Chip Seal	Min <2	2-4	2-4
	Mode 7-8	5-6	5-6
	Max 15-20	9-10	7-8
Slurry Seal	Min 4-5	2-4	2-4
	Mode 5-6,7-8,9-10	5-6	5-6
	Max 9-10	7-8	7-8
Micro-Surfacing	Min 5-6	5-6	2-4
	Mode 9-10	5-6	5-6
	Max 10-15	9-10	7-8
Thin HMA Overlay	Min 5-6	2-4	>2
	Mode 9-10	9-10	7-8
	Max 15+	11-12	9-10

Table 2.11 Single Application Chip Seal Performance and Cost Data (Goeffroy, 1996).

STATE	Pavement Age at Time of First Application (yrs)	Frequency of Application (yrs)	Observed Increase in Pavement Life (yrs)	Cost per Lane Mile (dollars)
AL	7-8	7-8	2-4	5,000-6,999
AZ	7-8	7-8	2-4	7,000-9,999
IN	7-8	5-6	5-6	2,000-3,999
MD	9-10	5-6	5-6	4,000-4,999
NY	7-8	2-4	2-4	7,000-9,999
NC	7-8	5-6	5-6	5,000-6,999
PA	5-6	5-6	5-6	4,000-4,999
TN	>10	Varies	2-4	10,000-14,999

Al-Mansour and Sinha (1994) used regression analysis to determine a functional relationship between the immediate gain in PSI and the PSI at the time of application of a chip seal. The authors note that the immediate gain in PSI represents the change in PSI estimated within one year of undertaking a chip seal activity. The equation describing the relationship is:

$$\Delta PSI = 0.3325 * (PSI - 1.433) \quad (2.34)$$

where: ΔPSI = gain in pavement serviceability owing to chip seal activity, and
 PSI = PSI at time of chip seal application.

Thirty-four observations were used in the development of this equation, and the correlation coefficient (R^2) was 0.5453.

Al-Mansour and Sinha (1994) also developed a model for the cost (in \$ per lane-mile) of performing a chip seal. The cost model is based on the pavement condition at the time the chip seal is performed. The logarithmic equation shown below is based on 34 observations and has a correlation coefficient (R^2) of 0.3079.

$$\log(SC) = 3.6101 + (-0.1034 * PSI) \quad (2.35)$$

where: SC = cost of performing chip seal (\$ per lane-mile), and
 PSI = pavement serviceability index at time of chip seal.

A life cycle cost analysis was also performed in this study. The results showed that for optimal cost savings when considering total costs (agency costs and vehicle operating costs), chip seal applications should be applied before the PSI value drops below 3.0.

In Nevada, Sebaaly, et al. (1995) developed performance models for chip seals using actual pavement performance data. The models were developed based on linear regression techniques, and include such parameters as pavement age, materials properties, traffic loadings, and environmental conditions. The state of Nevada was divided into three districts: District 1 included the southern portion of the state; District 2 included the northwestern portion of the state; and District 3 included the northeastern portion of the state. Performance models for chip seal applications were developed for each district. The District 2 chip seal performance model is

shown as Equation 2.36. The model was developed using 234 observations and has a correlation coefficient (R^2) of 0.87. Variables and ranges are shown in Table 2.12.

$$PSI = 2.86 + C1 + C2 + C3 + C4 - 1.02 * e^{(-4*ESALS)} - 0.015(AGGR) + 0.075(TMAX) - 2.98 * e^{(-3*FT)} - 0.125(SN) - 0.33(YEAR) + 0.005(YEAR)^2 \quad (2.36)$$

where: *ESALS* = cumulative value of 80-kN equivalent single axle loads,
AGGR = aggregate spread rate for chip seal project (lbs/yd²),
TMAX = maximum average yearly temperature that pavement may experience,
FT = total number of freeze-thaw cycles that pavement may experience over course of one year,
SN = structural number prior to application of chip seal,
YEAR = service year of the project (year of construction is year 0),
C1 = constant for specific binder type,
C2 = constant for binder type used in first structural layer below chip seal,
C3 = constant for maximum nominal aggregate size, and
C4 = constant for combination of binder type used in chip seal and binder type used in first structural layer below chip seal.

Table 2.12 Ranges for Variables in Chip Seal Model (Sebaaly, et al., 1995).

VARIABLES	Range
ESALS	365 – 1,647,245
AGGR	20 – 38
TMAX	58 – 73
FT	100 – 183
SN	1.68 – 6.17
YEAR	1 – 4
Binder Type	Constant (C1)
CRS-2/CRS-2H	1.281414527
LMCRS-2	1.475765738
AR-2000	0.00
AC	Constant (C2)
85 – 100	1.166532005
120 – 150	-0.098528394
SC-800	0.869102804
AR-2000	0.143673193
AR-4000	0.00
AGGS	Constant (C3)
3/8"	0.579529646
1/2"	0.00
Binder – AC Combination	Constant (C4)
CRS-2/CRS-2H & 120 – 150	0.554234128
CRS-2/CRS-2H & AR-4000	0.283288225
All other combinations	0.00

2.4.2.2 Slurry Seals

A slurry seal is a mixture of emulsified asphalt, water, well-graded fine aggregate, and mineral filler that is spread onto the pavement. Slurry seals are typically used on county and city streets. Slurry seals should not be applied to pavements exhibiting excessive cracking, rutting, or shoving. After application, traffic must be detoured to allow the slurry to cure. Slurry seals typically provide good performance for 3 to 5 years with an ADT (average daily traffic) of 5,000 vehicles per lane (Zaniewski and Mamlouk, 1996 and Raza, 1994). Table 2.13 shows performance and costs data for slurry seals from selected states (Geoffroy, 1996).

Table 2.13 Slurry Seal Performance and Cost Data (Geoffroy, 1996).

STATE	Pavement Age at Time of First Application (yrs)	Frequency of Application (yrs)	Observed Increase in Pavement Life (yrs)	Cost per Lane Mile (dollars)
CA	5-6	2-4	2-4	10,000-14,999
GA	11-12	-	2-4	1,500-1,999
MD	9-10	5-6	5-6	4,000-4,999
NC	7-8	5-6	5-6	4,000-4,999
TN	9-10	-	2-4	4,000-4,999
VA	Varies	5-6	2-4	2,000-3,999

2.4.2.3 Microsurfacing

Micro-surfacing is a thin surface layer (10-15 mm) consisting of polymer-modified asphalt emulsion, crushed aggregate, mineral filler, water, and field control additives as needed. Micro-surfacing is most often used for texturing, sealing, and filling ruts on existing asphalt pavements, and is suitable for use on roads with moderate to high traffic volumes. When properly designed and constructed, microsurfacing improves skid resistance and provides resistance to rutting for 4 to 7 years (Zaniewski and Mamlouk, 1996 and Raza, 1994). Table 2.14 shows microsurfacing performance and cost data from selected nearby states (Geoffroy, 1996).

Table 2.14 Microsurfacing Performance and Cost Data (Geoffroy, 1996).

STATE	Pavement Age at Time of First Application (yrs)	Frequency of Application (yrs)	Observed Increase in Pavement Life (yrs)	Cost per Lane Mile (dollars)
IN	9-10	-	2-6	10,000-14,999
KS	9-10	7-8	2-4	15,000-24,999
NY	7-8	5-6	-	25,000-49,999
NC	7-8	5-6	5-6	7,000-9,999
OH	9-10	-	5-6	7,000-9,999
TN	9-10	-	2-4	5,000-6,999
TX	10+	-	5-6	10,000-14,999
VA	7-10	5-6	5-6	15,000-24,999

Microsurfacing was applied to a 5.7-mile section of I-285 in Atlanta, Georgia in an attempt to repair pavement distress in preparation for the 1996 summer Olympics (Watson and Jared, 1998). The pavement contained raveling in the wheel paths and cracking in various areas. The estimated traffic load on this pavement section was 4.2 million equivalent single axle loads in each direction. Construction was complete in exactly one month.

After the microsurfacing was in place for one year, observations were made regarding pavement friction and smoothness, noise level, and appearance. The average friction number immediately after construction was 50, and the average friction number after one year was 46. It was documented that this difference was insignificant within the margin of testing variability. The method of friction testing used in this study was not provided. In addition, the friction number prior to applying the microsurfacing was not provided. The annual evaluation showed that the microsurfacing after one year was “good.” Very little raveling occurred and very few faint cracks were noticed since the microsurfacing was placed. Furthermore, no broken windshield claims were reported throughout the year, which may occur frequently with other surface treatments.

A cost estimate for the southeastern region of the United States indicated that microsurfacing mix costs \$1.07 to \$1.20/m² (\$0.90 to \$1.00/yd²), in 1998 dollars, at a spread rate of 11 to 16 kg/m² (20 to 30 lb/yd²). The actual cost of the I-285 project was \$1.48/m² (\$1.24/yd²), in 1998 dollars. High traffic volumes on the pavement section required a continuous paving operation, which resulted in higher placement costs. The microsurfacing treatment was anticipated to have a service life of approximately 5 to 7 years (Watson and Jared, 1998).

2.4.2.4 Thin Overlays

Thin hot-mix asphalt overlays are another alternative for correcting minor surface defects and restoring skid resistance. Such overlays are classified by their aggregate gradation as dense, open, or gap. Conventional asphalt concrete mixes use dense-graded aggregate, which contains appropriate amounts of various sizes to form a high-density mixture with a very small amount of air voids between aggregate particles. Dense-graded aggregates provide stability in the mix and minimize the need for binder. Performance of dense-graded thin overlays varies anywhere from 2 to 10 years, but typically they perform well for 5 to 8 years (Zaniewski and Mamlouk, 1996 and Raza, 1994). Thin overlay thicknesses may range from 15 to 40 mm (0.5 to 1.5 inches). Performance and cost data for thin overlays are shown in Table 2.15 (Geoffroy, 1996).

Table 2.15 Thin Overlay Performance and Cost Data (Geoffroy, 1996).

STATE	Pavement Age at Time of First Application (yrs)	Frequency of Application (yrs)	Observed Increase in Pavement Life (yrs)	Cost per Lane Mile (dollars)
AL	9-10	7-8	7-8	25,000-49,999
GA	11-12	11-12	9-10	15,000-24,999
KS	7-8	2-4	>2	10,000-14,999
MD	9-10	7-8	7-8	10,000-14,999
MI	>10	-	9-10	15,000-24,999
NY	7-8	9-10	9-10	25,000-49,999
NC	7-8	7-8	7-8	7,000-9,999
OH	9-10	9-10	7-8	25,000-49,999
PA	7-8	9-10	7-8	15,000-24,999
TN	Varies	-	2-4	10,000-14,999

2.4.3 Rehabilitation: Structural Overlays

A structural overlay is performed at the time when the initial (or existing) pavement reaches its terminal serviceability level. A structural overlay consists of an application of a layer of hot mix asphalt concrete with adequate thickness to an existing pavement structure. The purpose of a structural overlay is to improve the load carrying capacity of the existing pavement structure over the analysis period. Several models of overlay design and performance are reviewed in this section, beginning with the overlay design model presented in the AASHTO Design Guide (1993).

2.4.3.1 AASHTO Overlay Design Procedure

The AASHTO Guide for Design of Pavement Structures (1993) identifies eight design steps that are used for determining the required overlay thickness for an existing pavement. Although the design approach recommends testing the pavement to obtain valid design inputs, an approximate overlay design may be obtained by estimating inputs. The eight steps are described briefly in the following section. Costs can be estimated based on the overlay thickness and construction materials.

Step 1: Existing pavement design and construction.

This step includes determining the thickness and material type of each pavement layer as well as subgrade soil information.

Step 2: Traffic analysis.

Traffic analysis includes the past cumulative 18-kip ESALs in the design lane (for use in the remaining life method in determining SN_{eff}), and the predicted future 18-kip ESALs in the design lane over the design period. Equation 2.6 is used for estimating future traffic.

Step 3: Condition survey.

The condition survey requires measuring and recording the following information from the heaviest trafficked lane; percent of area with alligator cracking, number of transverse cracks per mile, mean rut depth, and evidence of pumping at cracks and at pavement edges (for use in the determination of the structural coefficients).

Step 4: Deflection testing (strongly recommended).

A heavy-load deflection device, such as a Falling Weight Deflectometer (FWD), and a load magnitude of approximately 9,000 pounds are recommended for measuring deflections in the outer wheel path of the pavement at intervals of 100 to 1,000 feet. Deflections are measured at the center of the load and at least one other distance from the load. The subgrade resilient modulus (M_R) can be back-calculated from deflection measurements made at sufficiently large distances from the load. The temperature of the AC mix during deflection testing must be either measured directly or estimated from surface or air temperatures. Finally, the effective modulus of all pavement layers above the subgrade (E_p) may be determined from the deflection measured at the center of the load plate.

Step 5: Coring and materials testing (strongly recommended).

It is recommended that coring and materials testing be performed. These processes are used to determine the resilient modulus of the subgrade, to visually assess asphalt stripping, degradation, and erosion of the AC layers and stabilized base, to assess degradation and contamination by fines of the granular base and subbase, and to measure the thickness of all layers.

Step 6: Determination of required structural number for future traffic (SN_f).

SN_f for future traffic is computed using the flexible pavement design equation (Equation 2.1) or the nomograph provided in the AASHTO Guide for Design of Pavement Structures (1993). The required inputs include the effective design subgrade resilient modulus, the design PSI loss (PSI immediately after overlay minus PSI at time of next rehabilitation), the overlay design reliability, and the overall standard deviation for flexible pavements. The effective M_R is determined by laboratory testing, back-calculation from deflection data, or an estimation based on available soil information.

Step 7: Determination of effective structural number (SN_{eff}) of the existing pavement.

Three methods are presented for determining the structural number of the existing pavement; a non-destructive testing (NDT) method, a condition survey method, and a remaining fatigue life method. It is recommended that the designer use all three methods, and then select a value for SN_{eff} based on the results, using engineering judgment and the past experience of the agency.

(1) SN_{eff} from NDT for AC Pavements

This method follows an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The effective modulus of pavement layers above the subgrade (E_p) is back-calculated from NDT deflection data as described in Step 4. The equation for this method is:

$$SN_{eff} = 0.0045D \sqrt[3]{E_p} \quad (2.37)$$

where: D = total thickness of all pavement layers above the subgrade (inches), and
 E_p = effective modulus of pavement layers above the subgrade (psi).

(2) SN_{eff} from Condition Survey for AC Pavements

The condition survey method uses the structural number equation:

$$SN_{eff} = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \quad (2.38)$$

where: D_1, D_2, D_3 = thicknesses of existing pavement surface, base, and subbase layers,
 a_1, a_2, a_3 = corresponding structural layer coefficients, and
 m_2, m_3 = drainage coefficients for granular base and subbase.

Drainage coefficients are determined in the same manner used in the initial pavement design. However, depending on the types and amounts of deterioration present, the layer coefficients assigned to materials in the existing pavement should, in most cases, be less than the values that would be assigned to the same materials for new construction.

Guidance for selecting layer coefficients for in-service pavement materials is provided in the AASHTO Guide for Design of Pavement Structures (1993).

(3) SN_{eff} from Remaining Life for AC Pavements

The remaining life of an existing pavement is described by the following equation:

$$RL = 100 \left[1 - \frac{N_p}{N_{1.5}} \right] \quad (2.39)$$

where: RL = remaining life (percent),

N_p = total traffic to date (ESALs), and

$N_{1.5}$ = total traffic to pavement “failure” (ESALs).

$N_{1.5}$ can be estimated from the flexible pavement design equation or from the nomograph provided in the AASHTO Design Guide (1993), where a “failure” PSI of 1.5 and a reliability of 50 percent are recommended. The effective structural number of the existing pavement is then determined using:

$$SN_{eff} = CF * SN_0 \quad (2.40)$$

where: CF = condition factor, where $CF = RL^{0.165}$ (AASHTO, 1993), and

SN_0 = structural number of the pavement if it were newly constructed.

Without modification, this method is not applicable to pavements that have already received one or more overlays. Furthermore, since this method does not reflect any benefit for pre-overlay repair, the estimate of SN_{eff} should be considered a lower limit value.

Step 8: Determination of overlay thickness.

The following equation is used to calculate the overlay thickness:

$$D_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{(SN_f - SN_{eff})}{a_{ol}} \quad (2.41)$$

where: SN_{ol} = required overlay structural number,

a_{ol} = structural coefficient for the AC overlay,

D_{ol} = required overlay thickness (inches),

SN_f = structural number determined in Step 6,

SN_{eff} = effective structural number of the existing pavement, from Step 7.

2.4.3.2 Texas Overlay Design Study

In Texas, DeSolminihaç and Hudson (1995) attempted to define estimates of the Serviceability Index (SI) at key points in the pavement's life, including the SI immediately after construction of a new pavement (the initial SI), just before rehabilitation, and just after rehabilitation. The SI used in this study is equivalent to the Present Serviceability Index (PSI) used in the AASHTO design method.

Experimental design was based on the factorial approach, where a three-factor experiment was developed. The three main factors selected were climatic zone, type of pavement, and category of use. The "climatic zones" experienced in Texas and included in this study are: (a) Climatic Zone I, which is wet but does not freeze; (b) Climatic Zone II, which is wet but has freeze-thaw cycling; (c) Climatic Zone IV, which is dry but does not freeze; and (d) Climatic Zone V, which is dry but has freeze-thaw cycling. The "type of pavement" included both flexible pavements and rigid pavements. Finally, the "category of use" consisted of four levels: (a) serviceability immediately after construction (new pavements); (b) serviceability before scheduled overlay projects (terminal pavements); (c) serviceability immediately after rehabilitation (resurfaced pavements); and (d) serviceability after reconstruction (reconstructed pavements). There were 145 pavement sections around the state of Texas selected and profiled for this study, of which 109 were flexible pavements. A summary of the data collected is shown in Figure 2.5.

Based on the summary shown in Figure 2.5, DeSolminihaç and Hudson (1995) concluded that the category "reconstruction" did not contain sufficient sections to allow a good statistical analysis. Therefore, this category was not considered for further analysis.

A statistical analysis (ANOVA) was performed on the data shown in Figure 2.5. The authors defined the inference space, or the space where the results of the study may be applied, as the highway system in Texas. The statistical analysis showed that the climatic region does not influence the variation of the SI in Texas. The study also showed that for flexible pavements, the initial SI currently used by TxDOT for pavement design is 5 percent higher than the average initial SI observed in the field. Similarly, the SI after rehabilitation used by TxDOT for flexible pavement design is 5 percent higher than the SI after rehabilitation observed in the field. Finally,

the terminal SI currently used by TxDOT for flexible pavement design is 7 percent lower than the average terminal SI observed in the field.

Table 2.16 summarizes the recommended values for new SI, resurfaced SI (just after overlay), and terminal SI for flexible pavements based on the results of this study. Also shown in Table 2.16 are the corresponding values for SI recommended by AASHTO and the state of Texas.

Climatic Zones		Region I		Region II		Region IV		Region V	
Pavement Types		Rigid	Flexible	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
Category of Use	New	5 ¹ 3.71 ² 0.62 ³	---	4 3.67 0.28	3 3.47 1.14	1 4.04 ---	1 3.84 ---	3 3.94 0.08	4 3.64 0.58
	Reconst.	---	---	---	---	---	1 3.73 ---	---	1 4.13 ---
	Resurfaced (after the overlay)	2 4.02 0.45	7 4.06 0.15	6 3.71 0.41	4 3.52 0.21	1 4.16 ---	14 3.95 0.20	2 4.15 0.10	18 3.81 0.59
	Terminal	4 3.25 0.34	12 2.85 0.72	4 3.67 0.12	13 3.01 0.65	2 3.26 0.89	17 3.24 0.55	2 4.12 0.04	14 3.07 0.77

¹ Sample size of all studied sections in specific condition
² Average SI value for all studied sections in specific condition
³ Standard deviation for all studied sections in specific condition

Figure 2.5 Summary of Data Collection (DeSolminihaç and Hudson, 1995).

Table 2.16 Comparison of New, Resurfaced, and Terminal SI Values for Flexible Pavements.

AGENCY	NEW SI (asphalt concrete)	RESURFACED SI (AC overlay)	TERMINAL SI	
			Primary	Secondary
AASHTO	4.2	-	2.5	2.0
Texas	4.2	4.2	3.0	2.5
This Study	4.0	4.0	3.2	2.8

2.4.3.3 Ontario Overlay Design Study

Hajek, et al. (1987) developed performance prediction models for asphalt concrete overlays of flexible pavements in Ontario. The models estimate the duration of overlay life cycle as a function of overlay thickness, traffic, maintenance patching, and life cycle duration of the initial pavement. Long-term pavement data were collected for 20 pavements. The data included the pavement condition rating (PCR), which is a measure of overall pavement condition on a scale of 0 to 100 (new pavements typically have a PCR around 95). Since rehabilitation is typically performed when the PCR is between 40 and 60, the threshold serviceability in this analysis was set a PCR level of 55. Overlay thicknesses ranged from 40 mm to 175 mm (1.5 – 7 inches), and the average was 70 mm.

The authors reported that the increase in PCR immediately following an overlay placement ranged from 32 to 66 PCR units, with an average of 53 PCR units. Furthermore, there was no correlation between this increase in PCR and overlay thickness. There was a correlation between the increase in PCR and the PCR prior to the overlay, although no useful prediction model for the increase in PCR could be constructed. However, a model for predicting the overlay life cycle was developed:

$$AFT55 = 1.32 * BEF55^{0.33} * THOV^{0.47} * ESAL^{-0.097} * 1.14^{PATCH} \quad (2.42)$$

where: *AFT55* = duration of overlay life cycle corresponding to the terminal PCR level of 55 (years),

BEF55 = duration of initial pavement structure life cycle corresponding to the terminal PCR level of 55 (years),

THOV = thickness of overlay (mm),

ESAL = number of equivalent single axle loads per day, and

PATCH = represents the extent of patching before the overlay (= 0 for no or limited amount of patching, or = 1 for all other cases).

The model above (Equation 2.42) was based on 20 observation sections and has a correlation coefficient (R^2) of 0.72.

A prediction model for the change in PCR during the first 5 years of overlay life was also developed (Equation 2.43). The authors report that determining the PCR change over 5 years helps to define the pavement performance curve and can be used in economic analysis. Although the correlation coefficient for this equation was relatively low ($R^2 = 0.26$), the output of this model showed that deterioration in the overlay occurred at an accelerated rate when there was a delay in placing the overlay (Hajek, et al., 1987).

$$\Delta PCR5 = 36.31 * THOV^{-0.65} * ESAL^{0.16} * DMI^{0.21} \quad (2.43)$$

where: $\Delta PCR5$ = change in PCR during the first five years of overlay life,

$THOV$ = thickness of overlay (mm),

$ESAL$ = number of equivalent single axle loads per day, and

DMI = distress manifestations index at time of overlay.

DMI was calculated as the sum of 25 individual pavement distresses, characterized by their severity and extent, and weighted according to their contribution to the PCR. A pavement with no distresses has a DMI of 0, while a pavement with many distresses, such as severe surface deformations and cracking, may have a DMI exceeding 100.

2.5 USER COSTS

User costs are the costs incurred by the user of a facility. User costs consist of accident costs, delay costs, and vehicle operating costs, all of which are significant factors that should be included in the LCCA. This section provides existing models and techniques for predicting each factor, and also includes typical cost rates assigned to each factor.

2.5.1 Accidents

Accident costs are determined based on the number and severity of accidents. The number of accidents on a facility may increase due to a number of factors such as a decrease in skid resistance, an increase in traffic demand, or the establishment of a work zone. Accident severity is typically classified in one of three ways: fatal, injury, or property damage only² (PDO).

² In West Virginia, a property damage only (PDO) accident is defined as an accident in which no one is killed or injured, but there is property damage of \$500 or more (West Virginia Accident Data, 1998).

West Virginia Accident Data is a report compiled by the West Virginia Division of Highways each year. The report consists of yearly accident totals, severity summaries, contributing factors in accidents, and 3-year statewide average accident rates for different highway types, among many other accident statistics. Based on West Virginia Accident Data averaged over the years 1990 through 1998, fatal accidents accounted for approximately 0.72 percent of all accidents, injury accidents accounted for 33.65 percent of all accidents, and PDO accidents accounted for the remaining 65.63 percent of all accidents. Furthermore, on the average, each fatal accident resulted in approximately 1.10 fatalities, while each injury accident resulted in approximately 1.54 injured persons.

As the severity of an accident increases, the accident-related user cost increases significantly. This is reflected in the Federal Highway Administration's (FHWA) unit cost estimates for accidents based on severity. For example, the FHWA estimates for crash cost rates in 1998 were:

Fatality	\$2,600,000
Injuries	
A. Incapacitating	\$180,000
B. Evident	\$36,000
C. Possible	\$19,000
Property Damage Only	\$2,000

To emphasize the significance of accident-related user costs, West Virginia Accident Data (1998) reported a total economic loss of \$2,851,383,000, due to accidents that occurred during 1998 (based on the number of accidents, severity, and FHWA cost per accident estimates above).

In order to perform a LCCA for evaluating alternative maintenance strategies, it is necessary to estimate the user costs related to accidents. This estimate can be determined by predicting the number and severity of accidents expected to occur on the facility over a given period of time. Since accident rates vary with traffic, skid resistance, and work zones, each situation must be considered carefully.

2.5.1.1 Accidents During Normal Operation

Accident data are used at the federal, state, and local levels for many purposes. At the federal level, accident data are used in rule making, legislative decision-making, and design and policy decision-making. At the state and local levels, accident data may be used for maintenance decision-making or for determining where to place a stronger enforcement effort. Furthermore, at

the state and local levels, a typical accident rate for a general classification of road can be compared to the actual accident rate of a particular road to determine high-accident locations.

Traffic Safety Facts (1999), published by the National Highway Traffic Safety Administration (NHTSA), documented that 6,334,000 accidents were reported in the United States throughout 1998. These accidents resulted in 41,471 fatalities and 3,192,000 injuries. Also, 4,269,000 accidents involved property damage only. West Virginia Accident Data (1998) documented 47,460 reported accidents during 1998, including 372 fatalities and 24,173 injuries. 31,439 of the 47,460 accidents in West Virginia involved property damage only. The trend of West Virginia crash data over the years 1990 through 1998 is shown in Figure 2.6.

Several statewide average accident rates for 1996-1998 are shown in Table 2.17. Accident rates such as these can be used to locate high-accident sites or to estimate the number of expected accidents for a particular road and traffic volume. However, the use of typical accident rates should be used with caution, as accidents may vary with differences in geometric design or surface conditions of the roadway.

Figure 2.6 Trends in West Virginia Accident Data.

Table 2.17 Selected West Virginia Average Accident Rates (1996-1998).

HIGHWAY TYPE	3-YEAR ACCIDENT RATE PER HUNDRED MILLION VEHICLE MILES
Rural Primary – 2 lanes	199
Rural Primary – 2 lanes 16-17'	280
Rural Primary – 2 lanes 18-22'	198
Rural Primary – 2 lanes over 22'	176
County Routes over 500 ADT	241
County Routes less than 500 ADT	599

2.5.1.2 Wet Weather Accidents

When pavements become wet, there is a significant loss of friction at the tire-pavement interface. This loss of friction may lead to an increased number of skidding accidents during wet weather conditions. The significance of wet weather accidents is shown in Figure 2.7, which is a summary of the contribution of wet weather accidents to the total number of accidents.

West Virginia Accident Data (1998) documents that wet roads were a contributing factor in 24.7 percent of all reported accidents in 1998. This percentage became even more significant after the West Virginia Division of Highways (WVDOH) reported that pavements in West Virginia are exposed to wet weather approximately 15 percent of the time.

The high percentage of wet weather accidents in relation to the low percentage of wet weather exposure shows the importance of studying wet weather accident experience, including contributing factors and various alternatives taken by highway agencies to reduce the number of wet weather accidents. It is necessary that highway agencies consider factors that may contribute to wet weather accidents during the design stage, maintain an adequate skid resistant surface on pavements throughout their service life, as well as identify and correct pavements with high wet weather accident experience.

Figure 2.7 Contribution of Wet Weather Accidents to Total Accidents in West Virginia.

Harwood, et al. (1976) developed relationships between wet-pavement accident rate and skid number at 40 mph (SN_{40}) for various types of roads. The equation developed for a rural, two lane road with an ADT of less than 10,000 vehicles per day is:

$$AR = 2.95 + [-0.046 * (SN_{40} - 48.36)] \quad (2.44)$$

where: AR = wet-pavement accident rate (accidents per 10^6 vehicle miles), and
 SN_{40} = skid number at 40 mph.

The authors note that while this equation provides an accurate estimate of the long-term expected value of accident rate, the usefulness of this relationship for predicting the effect of skid number on the accident rate is limited by the low correlation coefficient, R , of 0.23.

A study was conducted in Kentucky to determine a relationship between accident experience and pavement friction for principal, two-lane rural roads (Rizenbergs, et al., 1977). Based on 230 test sections, the authors found that modeling skid number versus the ratio of wet-to-dry pavement accidents yielded a higher correlation than modeling skid number (SN_{40}) versus wet-pavement accidents. However, the correlation coefficients were low (less than 0.430). The authors reported that the pavements selected for this study were wet approximately 11 percent of the time.

A 10-point moving average with volume stratification methodology was used to reduce variability in the SN – ratio of wet-to-dry pavement accidents relationship. The relationships are shown in Figure 2.8. Lines were drawn to approximate trends, and “reasonably” distinct break points were evident. The SN value corresponding to the break point on the trend line was referred to as the critical SN . The authors reported that improving the skid resistance of those pavements that exhibit SN s above the critical value has no meaningful reduction in wet-pavement accidents. The relationships developed using the 10-point moving average with volume stratification showed critical skid numbers that were higher for low-volume roads than for high-volume roads. More specifically, the critical SN s were 45 and 39 for roads with 650 to 2700 vehicles per day (vpd) and 2701 to 8400 vpd, respectively.

Ten-point moving averages, ADT less than or equal to 2700, 110 test sections.

Ten-point moving averages, ADT greater than 2700, 120 test sections.

Figure 2.8 Ratio of Wet- to Dry-Pavement Accidents versus SN, With Volume Stratification (Burchett and Rizenbergs, 1982).

Burchett and Rizenbergs (1982) developed models for predicting the number of wet pavement accidents per mile per year based on SN_{40} for various AADT ranges. Precipitation data were collected on a monthly basis from seven weather stations in and around Kentucky. Yearly averages of the percent of time of precipitation (including rainfall and snow and ice) were determined and averaged. The results showed that Kentucky experienced rainfall an average of 12.1 percent of the time. The following best-fit equations were adjusted to 12 percent wet time:

$$Y = \frac{0.14 + 10^{(0.478 - 0.0383SN)}}{0.62} \quad \text{for AADT range of 750 to 2,499} \quad (2.45)$$

$$Y = \frac{0.27 + 10^{(1.097 - 0.0518SN)}}{0.62} \quad \text{for AADT range of 2,500 to 4,999} \quad (2.46)$$

$$Y = \frac{0.66 + 10^{(1.294 - 0.0491SN)}}{0.62} \quad \text{for AADT range of 5,000 to 14,000} \quad (2.47)$$

where: Y = wet pavement accidents per mile per year, and

SN = skid number at 40 mph.

Such relationships were developed to show the benefits of improving the skid resistance of a pavement. For example, this model can be used to estimate the reduction in the number of wet-pavement accidents as a result of improving skid resistance.

Burchett and Rizenbergs (1982) also provided a model for estimating wet pavement accidents as a percentage of total accidents, based on skid number of the pavement. The authors adjusted the model for 12 percent wet time. The regression model was based on 1200 sections (approximately 5000 miles) of two-lane roads in Kentucky. The model is shown below:

$$Y = 16.0 + 10^{1.92 - (0.027 * SN)} \quad (2.48)$$

where: Y = wet-pavement accidents as percent of total (wet + dry), and

SN = skid number at 40 mph.

2.5.1.3 Work Zone-Related Accidents

Accident rates for work zones are not well established in the literature, due to several factors. The primary reason for this inadequacy is that there are currently no nationally recognized definitions of work zones or work-zone accidents. At this time, each state highway agency has their own unique definitions of these terms, although the Federal Highway Administration (FHWA) is currently involved in an effort to develop standardized definitions (Turner, 1999).

The current lack of standardized definitions inhibits research involving work zone safety. Work zone accident history data are limited, and available data are ambiguous (Walls and Smith, 1998). For example, an accident that occurs in a work zone-generated queue may or may not be classified as a work zone accident. In addition, it is difficult to accurately quantify the work zone exposure rate, including the length of the work zone, number of hours per day, and the number of days the work zone and resultant queues are in place. Furthermore, the accident rate, “while significantly higher in work zones than non-work zones, is sometimes still low enough that there aren’t any crashes in a given work zone because the exposure period is just too short to allow for statistically valid results” (Walls and Smith, 1998). Finally, the problem becomes more complex since traffic is handled in significantly different manners. For example, some work zones use permanent barriers while others use cones or drums, and some work zones use narrow lanes while others maintain the lane width and shoulders.

In November 1998, representatives from AASHTO, the American National Standards Organization (ANSI), and other interested groups gathered in Washington, D.C. to discuss the proposed work-zone definitions. The latest drafts of the definitions are (Turner, 1999):

A work zone is an area of a trafficway with highway construction, maintenance, or utility-work activities. A work zone is typically marked by signs, channeling devices, barriers, pavement markings, and/or work vehicles. It extends from the first warning sign or flashing lights on a vehicle to the “End of Road Work” sign or the last traffic control device. A work zone may be for short or long durations and may include stationary or moving activities.

A work-zone accident is a traffic accident in which the first harmful event occurs within the boundaries of a work zone or on an exit from a work zone, resulting from an activity, behavior, or control related to the movement of the traffic units through the work zone.

Despite the lack of standardized definitions, Walls and Smith (1998) documented a general rule of thumb indicating that accident rates in work zones are about three times the normal rate for the facility. However, the authors also note that “there does not appear to be much statistically significant research data to support this rule of thumb.” No references for this generalization were provided.

The West Virginia Division of Highways reported that in 1998, there were 231 work zone accidents, resulting in four fatalities and 133 injured persons (West Virginia Accident Data, 1998). However, work zone accident rates cannot be determined from this data, as the number and type of work zones, length of work zones, and traffic volumes in the work zones were not provided.

Additional work zone accident statistics were documented by Turner (1999), who reported that approximately 55 percent of work-zone fatalities occur in rural areas. Furthermore, it was reported that in 1994, work-zone fatalities in the US rose to an all-time high of 833. Although work-zone fatalities declined to 771 and 719 in years 1995 and 1996 respectively, Turner suggests that there is still a need for continuous emphasis on work-zone safety.

An FHWA report by McGee, et al. (1982) documented a synthesis of work zone related research, including accident experience, effectiveness of information and guidance systems, effectiveness of barrier systems, and effectiveness of various traffic management techniques. The section on work zone accident experience highlighted a number of accident based research studies. Wang and Abrams (1981) performed a regression analysis that indicated that accident rates during construction are strongly related to accident rates before construction. For rural road work zones, the authors recommended the following equation for estimating the “during construction” accident rate:

$$AR = 0.50 + (0.95 * RATEB) \quad (2.49)$$

where: AR = work zone accident rate, and

$RATEB$ = accident rate before construction (accidents per million vehicle miles).

This regression was based on various locations within seven states. The average accident rate for rural two-lane roads was 335 accidents per hundred million vehicle miles (hmvm). Using Equation 2.49, Wang and Abrams (1981) found an average work zone accident rate for these roads to be 368 accidents per hmvm.

Another work zone accident study by Graham, et al. (1977) showed that the degradation of a road type affects accident rates. A “before-during” comparison of accident rates showed that, in general, the more the capacity is reduced, the higher the increase in accident rates. Seven “two-lane reduced to one lane” sites were studied. The average accident rate before construction for these seven locations was 363.99 accidents per hmvm, while the average accident rate for the same locations during construction was 475.73 accidents per hmvm, an increase of 30.7 percent.

Highway accidents in construction and maintenance work zones were studied by Pigman and Agent (1990). Twenty case study locations in Kentucky were analyzed over a three year period from 1983 through 1986. Six of the locations were US or state (KY) routes, consisting of rural, two-lane roads. A 3-year accident rate for each pavement section prior to the work zone was determined. Then, an accident rate for each section during the work zone was determined. The average accident rate for the six US and KY routes prior to the work zones was 243 accidents per hundred million vehicle miles (hmvm). The average accident rate during the work zones was 410 accidents per hmvm. Based on these data, the average work zone accident rates for 2-lane, rural US and KY routes was approximately 1.7 times the average normal accident rate for these roads.

Pigman and Agent (1990) also noted the following observations. First, work zone accidents involving injury or fatality were more severe than statewide accidents. Second, the percentage of work zone accidents occurring in rural areas was much higher and the percentage in business and residential areas much lower than for all accidents. Also, the percentage of work zone accidents during wet, snow, or ice roadway conditions was low, which was related to less activity during such conditions. Finally, work zone accidents were compared to statewide accidents with regard to roadway geometry. It was found that a higher percentage of work zone accidents (than the percentage of statewide accidents) occurred on a curve, which shows the importance of providing adequate sight distance at the work zone.

Walls and Smith (1998) suggested a method for estimating the number of crashes and the additional crash costs associated with a work zone-generated detour. The technique involved using the Federal Highway Administration’s (FHWA) most current annual *Highway Statistics* report to determine crash rates by crash type for various roadway functional classes. Once crash rates were established for the regular route and the detour, the vehicle miles traveled (VMT) was calculated by multiplying together the expected number of vehicles per day, the number of days, and the number of miles for both the original route and the detour. The expected number of

crashes for each route was estimated by multiplying together the crash rate (crashes per 100 million VMT) and the VMT. The expected number of crashes for each route was multiplied by a constant crash cost rate (in \$/crash), yielding the total crash cost in dollars for each route. Finally, the total crash cost of the original route was subtracted from the total crash cost of the detour to determine the additional crash costs associated with the work zone-generated detour. Walls and Smith (1998) use the term *crash* rather than *accident* because the term accident implies that they are unavoidable, while in reality, highway crashes, to a large extent, are avoidable.

2.5.2 User Delay

User costs associated with delays are difficult to quantify because the cost rate describing the value of user time is very controversial. Some models have described the value of time in terms of vehicle type, such as passenger cars and trucks, while other models have based the value of time on a travel category, such as personal, business, and truck drivers. Once a cost rate for the value of time is established, the number of delay hours for a particular facility must be determined. Delays occur when vehicle demand exceeds the capacity of the facility, such as congestion due to accident sites or work zones, or when traffic is required to travel at slower speeds than normal, such as reduced speed limits in work zones.

NCHRP Report 133 (Curry and Anderson, 1972) provided estimated costs for the value of time based on vehicle type. The estimated costs, in 1970 dollar values, are \$3 per hour for passenger vehicles and \$5 per hour for all trucks. To convert these 1970 dollars to baseline study year dollars (1999), an escalation factor must be used (Walls and Smith, 1998). The escalation factor for the dollar value of time is determined by using changes to the *All Items Component* of the Consumer Price Index (CPI) for the base year (1970) and the baseline study year (1999). In 1970, the *All Items Component* of the CPI was 38.8, and in 1999, the *All Items Component* of the CPI was 166.6 (Bureau of Labor Statistics, 2000). The escalation factor is determined by dividing the baseline study year CPI by the base year CPI. For this example, the escalation factor is equal to $166.6/38.8$, or 4.294. The updated NCHRP Report 133 estimated costs for the value of time are \$12.88/vehicle-hour and \$21.47/vehicle-hour for passenger cars and trucks, respectively.

The U.S. Department of Transportation Office of the Secretary of Transportation (OST) provided estimated costs for the value of time based on travel category, including personal, business, or truck drivers (Walls and Smith, 1998). The estimated costs are based on the assumption that business and truck travel are valued more highly than personal travel, and inter-

city personal travel is valued more highly than local personal travel. Table 2.18 shows updated estimated cost ranges for the value of time developed by the OST. The values associated with *Mixed* are the ranges that should be used when the distribution between auto business and personal trips is not known.

Table 2.18 Estimated Cost Ranges for the Value of Time (\$/person-hr, updated to 1999 \$), (Walls and Smith, 1998, from U.S. DOT, The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations).

Travel Category	Local	Inter-city
Personal	\$6.56 to \$11.15	\$11.15 to \$16.72
Business	\$16.40 to \$24.70	\$16.40 to \$24.70
Mixed	\$7.00 to \$11.70	\$11.37 to \$17.16
Truck Drivers	\$18.03	\$18.03

Walls and Smith (1998) outlined a rational, step-by-step procedure for calculating work zone-related user delay costs. The procedure involves projecting future traffic demand, calculating work zone directional hourly demand and roadway capacity, identifying user delay cost components and quantifying traffic affected by each component, selecting and assigning delay cost rates, computing individual user cost components by vehicle class, and summing the total work zone user delay cost. The authors noted several types of delay that should be included in the determination of user delay costs due to work zones, including speed change delay, reduced speed delay, stopping delay, and queue delay.

Average work zone capacities (in vehicles per hour per lane) were provided for freeways with two or more directional lanes. Based on the hourly traffic demand and work zone capacity, the number of queued vehicles was determined, as well as the number of vehicles that traversed the work zone, traversed the queue, stopped (55-0-55 mph), and slowed down (55-40-55 mph). The delay time through the work zone and through the queue was determined by subtracting the time it takes to traverse either the work zone or queue length when they are present from the time it takes to travel the same distance when they are not present. Both calculations depend on the length to be traversed and the appropriate travel speeds when a work zone and/or a queue are present and when they are not.

Once the delay time per vehicle was calculated for the work zone and queue, delay cost rates were assigned. Vehicle classification was divided into three categories, including passenger cars, single-unit trucks, and combination trucks. The recommended values (in 1999 dollars) of travel time (in \$ per vehicle hour) for each vehicle class are \$10.93 to \$14.21 per vehicle-hour for passenger cars, \$18.58 to \$21.86 per vehicle-hour for single-unit trucks, and \$22.95 to \$26.23 per vehicle-hour for combination trucks.

Delay costs for speed change, reduced speed, stopping, and queuing were then calculated for each vehicle class. For example, the user delay cost for speed change was calculated by multiplying the number of affected vehicles by the delay time (in hours per 1,000 vehicles). This product was then multiplied by the delay cost rate (in \$ per vehicle-hour) to determine the delay cost per day. Multiplying the cost per day by the number of days the work zone is present yields the total speed change delay cost. Finally, the total costs for each delay type were summed together, resulting in a total cost for work-zone delay.

2.5.3 Vehicle Operating Costs (VOC)

Vehicle operating costs (VOC) are user costs associated with operating a vehicle. These costs include fuel, oil, tires, maintenance and repair, and depreciation. Under normal operating conditions, it is disputed whether or not there are significant differences in VOC for various pavement surfaces and conditions in the United States. Some researchers have argued that as surface roughness increases, VOC also increases, while other researchers argue that the increase in VOC is insignificant. However, it is agreed upon that there are some situations that result in additional VOC. For example, accidents or work zone conditions may result in an increase VOC due to additional speed changes, stops, miles, and idling time that the user does not usually experience under normal operating conditions.

NCHRP Report 133 (Curry and Anderson, 1972) provided estimates of additional VOC rates for stopping/speed changes and idling. The estimated costs shown in Table 2.19 reflect 1970 dollars. To make these factors applicable to current analysis, the values shown must be escalated to reflect more current year dollars (Walls and Smith, 1998). The escalation factor for VOC is determined by using changes to the *Transportation Component* of the Consumer Price Index (CPI) for the base year (1970) and the baseline study year (1999). In 1970, the *Transportation Component* of the CPI was 37.5, and in 1999, the *Transportation Component* of the CPI was 144.4 (Bureau of Labor Statistics, 2000). The escalation factor is determined by dividing the current year CPI by the base year CPI. For this example, the escalation factor is

equal to $144.4/37.5$, or 3.851. The estimated additional VOC for stopping and idling can be determined by multiplying the 1970 dollar values shown in Table 2.19 by the escalation factor.

Table 2.19 Additional Vehicle Operating Costs Associated with Stopping and Idling (\$/1,000 stops, in 1999 dollars), (from Curry and Anderson, 1972).

Initial Speed (mph)	Added Cost (\$/1,000 Stops) (Excludes Idling Time)		
	Passenger Cars	Single-Unit Trucks	Combination Trucks
5	2.73	9.36	34.00
10	8.93	20.95	78.37
15	15.33	34.27	131.43
20	21.99	48.95	192.20
25	29.00	64.70	259.44
30	36.51	81.14	331.92
35	44.56	97.97	408.40
40	53.30	115.26	487.65
45	62.77	131.55	568.48
50	73.13	147.61	649.66
55	84.41	162.70	729.92
60	96.78	181.00	808.02
65	110.25	198.06	NA*
70	125.00	NA*	NA*
75	141.10	NA*	NA*
80	158.62	NA*	NA*
Idling Cost (\$/vehicle-hr)	0.70	0.78	0.83

* Original data did not provide values for trucks at higher speed. Analysts will need to extrapolate these values when truck calculations are needed at these higher speeds.

Zoltan, et al. (1992) derived three equations for determining user costs for an average representative vehicle (the term average representative vehicle was not defined). The user cost functions were developed for use in maintenance treatment decision-making based on distress, structural capacity, and roughness. The three user cost equations are:

$$Y_1 = 895 - 508(PSR) + 234(PSR)^2 - 50.6(PSR)^3 + 4.1(PSR)^4 \quad (2.50)$$

$$Y_2 = 668 - 355(PSR) + 158(PSR)^2 - 33.8(PSR)^3 + 2.7(PSR)^4 \quad (2.51)$$

$$Y_3 = 398 - 267(PSR) + 118(PSR)^2 - 25.1(PSR)^3 + 2.0(PSR)^4 \quad (2.52)$$

where: Y_1 = total economic vehicle operation costs including passenger travel time per 1000 vehicle-km (in U.S. dollars),

Y_2 = total economic vehicle operation costs excluding passenger travel time per 1000 vehicle-km (in U.S. dollars),

Y_3 = total variable economic vehicle operation costs (gasoline, oil, tire, and vehicle maintenance) per 1000 vehicle-km (in U.S. dollars), and

PSR = present serviceability rating, based on a scale of 0 to 5.

Walls and Smith (1998) described a procedure for calculating vehicle operating costs (VOC) associated with work zones. The step-by-step procedure is the same as that described by Walls and Smith (1998) for user delay costs in Section 2.5.2. The additional VOC rates for vehicle running cost per 1,000 stops and idling costs used in this analysis are based on vehicle classification and initial speed (in mph), as documented in NCHRP Report 133 (Curry and Anderson, 1972). The VOC rates were adjusted to August 1996 dollars. The transportation component of the CPI was 37.5 in 1970 and 142.8 in August 1996. The VOC rates in 1970 dollars were multiplied by an escalation factor of 3.808 to obtain the VOC rates in August 1996 dollars. Work zone related excess VOC including costs due to speed change, stopping, and idling were calculated for each vehicle class. Finally, the total cost of each VOC type was calculated for the duration of the work zone, and summed together to yield a total cost for the work-zone VOC.

2.6 RISK ANALYSIS

Risk analysis describes any quantitative and/or qualitative method for assessing the impacts of risk on decision situations (@Risk, 1997). The term “risk” implies that a given action has more than one possible outcome. The risk associated with a particular event may be quantified by determining all possible outcomes and the relative likelihood of each value.

However, the decision as to whether or not a particular situation is risky involves personal judgment. The decision-maker must first determine the amount of risk he or she is willing to take, and then make a decision based on the risk associated with each alternative.

There are four general steps in performing a risk-based analysis. First, the user must develop a model for the analytical situation. Then, the user must identify the uncertainty associated with each variable in the model by defining each input variable by a probability distribution. Next, the model is exercised numerous times by selecting sets of input data from the user-defined input parameters. The output is recorded and the exercising of the model is terminated when the output distributions are stable. Finally, the user must make a decision based on the simulation results and personal preferences. One type of simulation process that is commonly used is known as the Monte Carlo simulation process, which is presented in the following section.

2.6.1 Monte Carlo Simulation

Monte Carlo simulation is an analysis method whereby random sampling procedures are used for treating deterministic mathematical situations. The simulation process allows the user to include the inherent uncertainty associated with each input parameter into the analysis. The output of a Monte Carlo simulation is a probability distribution describing the probability associated with each possible outcome.

The general procedure for a Monte Carlo simulation is shown in Figure 2.9. First, a deterministic model is developed where multiple input variables are used to estimate a single value outcome. The user must be certain that all input parameters used in the analysis are independent of each other. Then, each independent parameter is defined by a probability distribution describing the variability associated with that particular parameter. A random trial process is then initiated to establish a probability distribution function for the deterministic situation being modeled. During each iteration of the process, a value for each parameter is randomly selected from the probability distribution defining that parameter. The random values are entered into the calculation and an output value is obtained. Numerous solutions are obtained by making multiple iterations through the program and obtaining a solution for each iteration. The appropriate number of iterations for an analysis is a function of the number of input parameters, the complexity of the modeled situation, and the desired precision of the output. The final result of a Monte Carlo simulation is a probability distribution describing the output parameter.

Figure 2.9 General Monte Carlo Simulation Approach (Hutchinson and Bandalos, 1997).

2.6.2 Risk-Based LCCA Models

Only one risk-based LCCA model for pavements was found in the literature. The report by Walls and Smith (1998) recommends procedures for conducting LCCA of pavements, provides detailed procedures for determining work zone user costs, and introduces a risk-based approach to LCCA.

The LCCA procedure presented by Walls and Smith (1998) includes eight steps. First, the user must derive alternative pavement design strategies for the analysis period. Next, pavement performance periods and activity timings are determined based on state highway agency experience. Using these inputs, agency and user costs are estimated. Agency costs are

determined based on unit prices from previously bid jobs of comparable size. The procedures used for estimating various user cost components were presented in great detail by Walls and Smith (1998). The procedures for estimating accident costs, user delay costs, and excess vehicle operating costs were described previously in Sections 2.5.1, 2.5.2, and 2.5.3, respectively.

The next step of the LCCA procedure is to develop expenditure stream diagrams (also known as cash flow diagrams) for each pavement design strategy, which help the user to visualize the extent and timing of expenditures. The net present value for each strategy is then computed by discounting all future costs to the base year and adding these costs to the initial cost. The next step is to analyze the results, which is accomplished by performing a sensitivity analysis on the LCCA results. Finally, the analyst uses the results from the LCCA and the sensitivity analysis to re-evaluate design strategies.

The detailed procedures for estimating work zone user costs described by Walls and Smith (1998) were presented in Section 2.5 of this chapter. The final part of the Walls and Smith report (1998) introduced a risk-based approach for pavement LCCA. The general approach consisted of the following steps. First, the user must identify the structure and layout of the problem. Next, the uncertainty associated with each input variable is quantified using probability distributions. The simulation is performed, and the results are interpreted and analyzed. A sensitivity analysis may be performed to determine the effects of various input variables on the model output. Finally, the user must make an overt decision based on a combination of the level of risk he or she is willing to tolerate and the probabilistic model output.

2.7 CONCLUSIONS

No integrated models using risk analysis were found for deriving various pavement preservation strategies and analyzing their performance and life-cycle costs. However, the general concept of the risk-based LCCA developed by Walls and Smith (1998) was used as a basis for this research. Components needed for an integrated model are available in the literature at varying degrees of sophistication. The AASHTO model for structural design of new pavements and rehabilitation is adequate for inclusion into the integrated model. However, this model only considers serviceability (PSI). Additional models are needed for skid resistance (skid number) to supplement the AASHTO model. Furthermore, only limited serviceability and skid resistance models are available for describing the effects of preventive maintenance treatments.

The emphasis of this research is the development of an integrated model using risk analysis. Therefore, component models from the literature will be incorporated, even when they are not as robust as would be desired. This allows for the development of the model and

subsequent sensitivity analysis. The results of the sensitivity analysis will be evaluated to identify and prioritize a research program to refine the integrated model.

CHAPTER 3

INITIAL PAVEMENT DESIGN MODEL

The initial pavement design for this research was derived in a unique manner combining a general form of the AASHTO flexible pavement design equation with a risk analysis-based approach for defining the design reliability. The AASHTO pavement design procedure was discussed previously in Chapter 2. Section 3.1 identifies a drawback to the reliability concept that is incorporated into the AASHTO design equation. Section 3.2 describes the risk analysis-based approach to pavement design that was developed for this research. A flowchart and detailed description of the model are provided in Section 3.3. Section 3.4 describes the application of the initial pavement design model, including the required inputs as well as the model output. Finally, Section 3.5 summarizes the initial pavement design procedure and highlights the features of this unique approach.

3.1 A DRAWBACK TO RELIABILITY IN THE AASHTO DESIGN EQUATION

The AASHTO pavement design procedure described in Chapter 2 is relatively straightforward. Since variability exists in each of the design parameters, resulting in variability in the overall pavement performance, probability theory should be included in the design equation.

The theory behind the reliability concept is sound. However, the two measured values (actual traffic and actual performance) cannot be measured at the design stage. Therefore, the reliability concept is incorporated into the AASHTO design procedure using the six steps presented in Section 2.1.4.

In general, the designer selects a desired reliability level, R , and looks up the corresponding z -value in the standard normal distribution curve table. Then the designer assumes an overall standard deviation, which is a single value that accounts for the variability associated with each of the input variables used in the traffic prediction and the performance prediction. For example, based on results from controlled conditions at the AASHO Road Test, AASHTO recommends selecting a standard deviation of 0.45 for all flexible pavements (AASHTO, 1993), regardless of the accuracy or precision of each of the discrete input values. In fact, since the traffic loads during the road test were controlled, there was no traffic variability in the AASHTO estimation of standard deviation.

This procedure is an accepted practice because of its simplicity. However, it is virtually impossible to measure the overall standard deviation term used in the AASHTO Design Guide. The use of a value derived from the construction of a controlled research project in the 1950's to represent modern pavement construction is questionable.

3.2 A RISK-BASED APPROACH TO PAVEMENT DESIGN

The initial pavement design in this study involves using a deterministic form of the AASHTO pavement design equation with a risk-based (probabilistic) approach. The AASHTO approach to pavement design is a widely accepted approach to pavement design, and thus was used as a basis for this research. The general AASHTO pavement design equation used in this research is Equation 2.7. The probabilistic terms, Z_R and S_0 , are not included in this equation. Instead, probability theory is incorporated into the procedure by using risk analysis.

Each input variable is defined in terms of a probability distribution. The pavement design is derived using risk analysis and simulation methodology to incorporate the probabilistic input variables with a general form of the AASHTO flexible pavement design procedure. A feature of this approach is that the variability associated with the traffic prediction and the performance prediction is separated into two distributions. Since the variability associated with each variable is known, it is no longer necessary to combine all of the variability together into a single estimated value.

Once the distributions for the traffic prediction and the performance prediction are established, the area associated with pavement failure can be defined. For this research, pavement failure may occur at any point within the area of intersection between the traffic prediction and performance prediction probability curves. Failure occurs in this area because the probability exists that the expected traffic load (traffic prediction) is greater than the allowable traffic load, as calculated using the design equation (performance prediction). This failure area is illustrated in Figure 3.1.

One goal of the designer is to design the pavement so that it is capable of carrying the expected traffic load over the design period. Therefore, the designer wants to limit the probability of pavement failure, or the area of intersection of the traffic prediction and performance prediction curves. With two separate distributions, this can be accomplished by moving the distributions closer together or further apart so that the area of intersection is equal to some specified value. The traffic prediction curve is governed by inputs provided by the user, so this distribution cannot be adjusted. Consequently, the distribution that must be adjusted is the performance prediction. The traffic, initial PSI, terminal PSI, and roadbed soil resilient modulus

are parameters associated with the performance prediction that are fixed values provided by the user. Therefore, the structural number is the performance prediction parameter that may be adjusted in the pavement design process so that an adequate structural design may be achieved. Layer thicknesses may be increased or decreased so that the area of intersection between the traffic prediction curve and the performance prediction curve is some specified value. For example, if a designer wishes to design a pavement with 50 percent probability of failure, the area of intersection between the two distributions should be 0.5. The remainder of this chapter describes the application of this theory in detail.

Figure 3.1 Area Associated with Pavement Failure.

3.3 DETAILED DESCRIPTION OF THE PAVEMENT DESIGN MODEL

The objective of the initial pavement design model in this research is to determine a structural design with a specified probability of failure that will require a 1.5-, 2.0-, or 2.5-inch overlay some year during the analysis period (the period of time that a design strategy must cover). The structural overlay thickness is selected by the user, and the default value for this program is set at 2.0 inches.

It is assumed that the structural overlay will not be performed prior to year 8. Therefore, the performance period (the period of time that an initial pavement will last before it needs rehabilitation) is initially set equal to 8 years, and is increased by one year until the specified overlay thickness is required. The overlay design procedure is described later in Chapter 4.

A general flowchart outlining the pavement design process is shown in Figure 3.2. The flowchart contains two essential simulations for deriving the traffic prediction and the performance prediction. The flowchart is described in the following sections.

3.3.1 Traffic Prediction

The first step in the initial pavement design model is to determine the traffic prediction. First, a value is randomly selected for each parameter, based on probability distributions defined by the user. The parameters that are required for deriving the traffic prediction include the expected average daily traffic (ADT), percent trucks, an estimated truck factor, and traffic growth rate. ADT is the average daily traffic, or the number of vehicles that use the road daily, and is expressed in terms of vehicles per day (vpd). Percent trucks is the percentage of ADT that are classified as trucks. The estimated truck factor is a value that is applied to all trucks. It is defined as the sum of equivalent single axle loads (ESALs) for all trucks weighed divided by the total number of trucks weighed. For rural principal roads, a truck factor of 0.38 is a suggested estimate (Huang, 1993). The growth rate is the yearly rate of growth of traffic that is using the facility, and is expressed as a percentage.

Figure 3.2 Flowchart for Initial Pavement Design Procedure (page 1).

Figure 3.2 Flowchart for Initial Pavement Design Procedure Continued (page 2).

There are two additional parameters used in the traffic prediction derivation that are not defined by the user. The first parameter is the directional distribution, which is the percentage of traffic traveling in the direction of the design lane. Unless directional traffic volumes are known, the directional distribution is usually assumed to be 50 percent. For this study, a directional distribution of 50 percent is assumed since directional traffic volumes are not known and for simplification in estimating user delay caused by work zones and accidents. The second parameter that is used in the traffic derivation but not defined by the user is the lane distribution. Since the model is developed for two-lane roads only, the lane distribution factor is 100 percent.

For this study, each parameter is defined by a normal probability distribution, with a mean and a standard deviation (a truncated normal probability distribution may be used in order to eliminate the possibility of a negative parameter value). The user may either input a mean and standard deviation for each parameter based on available data, or the user may opt to select default values when data are not available. The default values for the initial pavement design parameters are identified in Table 3.1.

Once the parameter distributions are defined, a value is randomly selected for each required input, and the traffic prediction for a specific performance period is calculated using Equation 2.6. The units of Equation 2.6 are equivalent single axle loads, or ESALs, where the standard axle load is defined as the 18-kip (80-kN) single-axle load. Since the units of the performance prediction described in the following section are log ESALs, the log of the traffic prediction is calculated so that the two distributions may be compared.

As illustrated in Figure 3.2, the random selection of traffic parameters and the traffic prediction calculation define one iteration of a simulation. At the end of each iteration, the traffic prediction is saved and new input values are randomly selected for the next iteration. The simulation is complete when the process above is iterated until the change in output values becomes stable (convergence is reached). After the simulation is complete, the traffic prediction probability distribution for a given performance period is defined by the mean and standard deviation of the traffic predictions.

Table 3.1 Allowable Ranges and Default Values for Pavement Design Parameter Means and Coefficients of Variation.

TRAFFIC PARAMETERS	PARAMETER MEAN		COEFFICIENT OF VARIATION	
	ALLOWABLE RANGE	DEFAULT VALUE	ALLOWABLE RANGE	DEFAULT VALUE
Average Daily Traffic (ADT)	1,000 to 10,000 vpd	5000 vpd	1% to 20%	10%
Percent Trucks	1% to 20%	10%	1% to 20%	10% ²
Truck Factor	0.12 to 0.52 ²	0.38	1% to 20%	10%
Growth Rate	1.0% to 5.0%	3%	1% to 20%	10% ²
DESIGN PARAMETERS				
Analysis Period	25 to 40 years	30 years	-	-
Initial PSI	4.0 to 4.5	4.2 ¹	1% to 10%	6.7% ²
Terminal PSI	1.5 to 2.5	2.0 ¹	-	-
Effective M _R	1,000 to 40,000 psi ¹	5,000 psi ³	1% to 20%	10%
Overlay Thickness	1.5, 2.0, 2.5 inches	2.0 inches	-	-
MATERIAL PROPERTIES				
Surface Aggregate Type	1 = SRA 2 = unrestricted	1 = SRA	-	-
Surface Layer Coefficient	0.20 to 0.50 ¹	0.44 ³	1% to 20%	10% ²
Base Layer Coefficient	0.04 to 0.20 ¹	0.14 ³	1% to 20%	10%
Base Drainage Coefficient	0.5 to 1.5 ¹	1.0 ³	1% to 20%	10% ²
Subbase Layer Coefficient	0.04 to 0.20 ¹	0.11 ³	1% to 20%	10%
Subbase Drainage Coefficient	0.5 to 1.5 ¹	1.0 ³	1% to 20%	10% ²
CONSTRAINTS				
Probability of Failure Threshold	5% to 50%	50%	-	-

1 = Recommended value(s) obtained from AASHTO Design Guide (1993).

2 = Recommended value(s) obtained from Huang (1993).

3 = Recommended value(s) obtained from Head, et al. (1988).

3.3.2 Performance Prediction

The next step of the initial pavement design model is to determine the performance prediction for a specific performance period. First, the minimum thickness of each pavement layer is determined. Minimum thicknesses are based on guidelines provided in the AASHTO Design Guide (shown in Table 2.1), where the minimum thickness of each layer is a function of the traffic prediction.

After minimum thicknesses are established, the performance prediction is determined in a manner similar to that used for determining the traffic prediction. First, a value for each design parameter is randomly selected based on probability distributions input by the user. The required input parameters are defined below.

3.3.2.1 Input Parameters for the Performance Prediction

The parameters required for designing the pavement structure include the analysis period, the performance period, the initial Present Serviceability Index (PSI), the terminal PSI, the effective roadbed soil resilient modulus, or the effective M_R , and the structural number (SN). The parameters and input ranges for mean values and coefficients of variation are identified in Table 3.1.

The analysis period is defined as the period of time that any design strategy must cover (Huang, 1993). AASHTO Design Guide recommends an analysis period of 20-50 years for high-volume rural roads, and an analysis period of 15-25 years for low-volume paved roads. Typically, some form of maintenance must be performed on the pavement during the analysis period. The performance period refers to the time that an initial pavement will last before rehabilitation. It is equivalent to the time elapsed as a new or reconstructed pavement structure deteriorates from its initial serviceability to its terminal serviceability. Since the initial pavement design is established so that the required overlay thickness is a specified thickness, the user is not required to input the performance period. Rather, this parameter is determined through an iterative process within the computer program.

The initial Present Serviceability Index (PSI) is a function of pavement type and construction quality. A typical mean value for the initial PSI of a flexible pavement is 4.2. AASHTO recommends a coefficient of variation of 6.7 percent for the initial PSI (AASHTO, 1993). The terminal PSI is the lowest level of serviceability that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. An index of 2.5 or higher is suggested for design of major highways and 2.0 or higher for highways with lower traffic volumes (Huang, 1993).

The effective roadbed soil resilient modulus, or the effective M_R , describes the reaction of the roadbed soil to an imposed load and to changes in environmental conditions (Head, et al., 1988). The effective M_R is a modulus value that would result in the same damage as if seasonal modulus values were used.

The structural number (SN) is an abstract number that describes the structural strength of a pavement (AASHTO, 1993). The SN is a function of layer thicknesses, layer coefficients, and drainage coefficients. A layer coefficient is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement (Huang, 1993). A layer coefficient is assigned to each layer in the flexible pavement structure, and is used to convert the layer thicknesses into a structural number. If layer coefficient values are not available for the user, AASHTO provides guidelines for selecting estimated values based on results from the AASHO Road Test. For example, AASHTO recommends using layer coefficients of 0.44, 0.14, and 0.11 for the asphalt concrete surface, aggregate base, and granular subbase layers, respectively.

Drainage coefficients are applied to granular bases and subbases to modify the layer coefficients depending on the quality of drainage and the percentage of time the pavement structure is exposed to moisture levels approaching saturation. At the AASHO Road Test, all drainage coefficients were equal to 1. The AASHTO Design Guide (1993) presents the recommended drainage coefficients for untreated bases and subbases as a function of the quality of drainage and the percent of time during the year that the pavement structure would normally be exposed to moisture levels approaching saturation.

The user is also required to input the surface aggregate type, which is a unique input for this LCCA. The two options from which the user must select for this parameter are skid resistant aggregate (SRA) or unrestricted aggregate. Since this model was developed in West Virginia, where limestone is a readily available aggregate, the unrestricted aggregate type is assumed to be limestone aggregate that does not qualify as SRA. Surface aggregate type is included in this model to aid in predicting the reduction of skid resistance over time, and it is an important factor in calculating agency costs of construction and maintenance and user costs due to accidents.

Finally, the user must input a probability of failure threshold. This threshold is a percentage that establishes failure criteria for the pavement design. The probability of failure threshold may range from 1 percent to 50 percent. For low volume roads, AASHTO (1993) recommends a reliability level of 50 percent, which corresponds to a probability of failure of 50 percent.

3.3.2.2 Determining the Performance Prediction Probability Distribution

The performance prediction simulation begins by randomly selecting a value for each input parameter, based on the parameter's probability distribution defined by the user. The initial structural number is calculated using Equation 2.2 with the minimum layer thicknesses, and randomly selected layer coefficients and drainage coefficients. The performance prediction, in log ESALs, is then calculated using Equation 2.7. The performance prediction is saved, and new input values are randomly selected. After this process is iterated over a specified number of times, the average and standard deviation describing the performance prediction are calculated.

The traffic prediction and the performance prediction are two separate probability distributions describing the expected number of log ESALs over the performance period, and the expected performance, in log ESALs, of the pavement over the performance period, respectively. At this point, the traffic prediction distribution is compared to the performance prediction distribution. If the average of the performance prediction is less than the average of the traffic prediction, then the pavement is likely to fail (there is greater than 50 percent probability of failure). In this case, the structural capacity of the pavement must be increased, which can be accomplished by increasing one or more of the layer thicknesses.

The designer may determine which layer thickness to increase by finding the most cost-effective layer based on ratios of unit costs and products of layer coefficients and drainage coefficients. For example, if the inequality shown in Equation 3.1 is true, then increasing the surface layer thickness would be the most cost-effective way to increase the pavement thickness.

$$\frac{\text{unit cost of surface layer material}}{\text{unit cost of aggregate base}} \leq \frac{a_1}{a_2 m_2} \quad (3.1)$$

where: a_1 = layer coefficient of layer 1 (surface),

a_2 = layer coefficient of layer 2 (aggregate base), and

m_2 = drainage coefficient of layer 2 (aggregate base).

For this model, if the most cost-effective layer is the surface layer, the surface layer thickness is increased by one-half inch. If the most cost-effective layer is the base or subbase, the appropriate layer thickness is increased by one inch.

After layer thicknesses are adjusted, the performance prediction simulation is re-run, and a new performance prediction distribution is determined. Again, the traffic prediction average is

compared to the performance prediction average. If the performance prediction average is still less than the traffic prediction average, the pavement thickness must be increased in a similar manner as described above, and the performance prediction simulation must be re-run. This process continues until the performance prediction average is greater than the traffic prediction average.

When the performance prediction average becomes greater than the traffic prediction average, the point of intersection of the two distributions can be determined. The two probability distributions, as well as their point of intersection, are shown in Figure 3.3. Since the traffic prediction and performance prediction probability distributions are normal, they are each defined by a probability density function of the general form:

$$f(x) = \frac{1}{\sqrt{2\pi}s} e^{\left[\frac{(x-\bar{x})^2}{2s^2} \right]} \quad (3.2)$$

where: s = standard deviation of the distribution and
 $\bar{x}$ = mean of the distribution.

Given the mean and standard deviation of each distribution, it is possible to set the probability density functions equal to each other and solve for x , the point of intersection of the two distributions. Once the point of intersection is known, the probability of pavement failure, or the failure area shown in Figure 3.3, can then be determined.

Figure 3.3 Probability of Failure.

The failure area can be determined by first transforming the traffic prediction distribution and the performance prediction distribution to standard normal distributions and then using integration or the standard normal curve area table to determine the one-tailed area under each curve. The transformation to the standard normal distribution is performed using Equation 3.3. The area of interest under the traffic prediction curve is the area to the right of the intersection point, and the area of interest under the performance prediction curve is the area to the left of the intersection point.

$$z = \frac{x - \bar{x}}{s} \quad (3.3)$$

where: z = standard normal random variable,

$\bar{x}$ = mean of normal distribution, and

s = standard deviation of normal distribution.

The calculated area is the probability of failure, or the probability that the expected number of load repetitions over the design period is greater than the number of load repetitions the pavement can carry before failure. Thus, design reliability is included in this approach by limiting the area of intersection between the traffic prediction curve and the performance prediction curve. When appropriate values for all input parameters are selected, the probability of design failure can be limited to any percent desired by the designer.

If the probability of failure is greater than the probability of failure threshold input by the user, the pavement thickness must be increased so as to reduce the probability of failure of the pavement design. The cost-effectiveness ratios described previously are used to determine which layer thickness should be increased. If the surface layer is the most cost-effective, the surface layer thickness is increased by one-half inch. If the base or subbase layer is the most cost-effective, the appropriate layer thickness is increased by one inch. If the pavement thickness is increased, the entire performance prediction simulation is re-run, and a new probability of failure is calculated.

When the calculated probability of failure becomes less than the probability of failure threshold defined by the user, the initial pavement design is complete. After an acceptable performance prediction is determined, a “check” is performed to verify that the probability of failure did not become too low with the last increase in layer thickness. This may occur if the most cost-effective layer is the surface layer, which has a relatively high layer coefficient, thus significantly increasing the structural number with a small increase in layer thickness.

For this research, the probability of failure is considered to be too low if it is less than the probability of failure threshold by 5 percent or more. If this is the case, an adequate pavement design may be obtained at a lower cost by reducing the previous layer thickness increase and adding thickness to the second-most cost-effective layer. The corrected layer thickness depends on which layer was defined as the second-most cost-effective layer (either one-half inch for the surface layer or one inch for the base or subbase layer). If this correction is made, the performance prediction simulation must be re-run, a new performance prediction probability distribution is determined, and the initial pavement design is complete.

3.4 INITIAL PAVEMENT DESIGN MODEL APPLICATION

The initial pavement design model is used to determine a structural pavement design with a specified probability of failure that will require either a 1.5-, 2.0-, or 2.5-inch overlay some year during the analysis period. The time period between the initial construction of the pavement and

the overlay is referred to as the performance period. The performance period is initially set equal to 8 years, and is then increased until the average overlay thickness required is equal to the overlay thickness selected by the user. Each time the performance period is increased by one year, the initial pavement design model is re-run, resulting in new traffic prediction and performance prediction probability distributions in addition to new layer thicknesses. The remainder of this section identifies the required user inputs, defines default values for input parameters, and describes the output provided by the initial pavement design model.

3.4.1 User Inputs

The required user inputs were described earlier in this chapter. These inputs were identified in the right-hand column of Table 3.1. The allowable range and default value for the mean and coefficient of variation for each input parameter were also shown in Table 3.1.

3.4.2 Initial Pavement Design Model Output

The output of the initial pavement design component of the LCCA is a pavement design with a limited probability of failure, based on the expected number of load repetitions over a specified time period. Once the initial pavement design is established, several parameters are saved and used in subsequent components of the LCCA model. For example, after each iteration of a simulation, the layer thicknesses are saved in arrays and used later in the LCCA model for calculating the initial cost of construction. Also, arrays containing the structural number, initial PSI, terminal PSI, effective roadbed soil resilient modulus (M_R), traffic prediction, and performance prediction are saved for later use in the overlay design procedure and for determining a preventive maintenance strategy.

3.5 SUMMARY OF PAVEMENT DESIGN METHOD

The initial pavement design is the first model component utilized in the LCCA. For this research, a pavement is designed such that an overlay of a specified thickness is required at some point in time during the analysis period. A risk analysis-based approach is used to incorporate variability into the model. Parameters are defined by probability distributions so that the variability associated with each parameter can be included in the model. Once the pavement design is established, several key factors are saved for use in other components of the LCCA, such as the preservation strategy design and present worth cost calculations.

The feature of this initial pavement design model is the unique incorporation of variability into the design procedure. Previously, the variability associated with all aspects of the

pavement design was combined into a single estimated value based on results from the AASHO Road Test. However, the pavement design model developed for this LCCA allows the user to input the variability associated with each input parameter. Risk analysis is then used to establish separate probability distributions for the traffic prediction and the performance prediction, thereby capturing the effects of variability within the design process.

CHAPTER 4

PRESERVATION STRATEGIES MODELS

Once the initial pavement design is established, the next step of the LCCA model is to develop preservation strategies for the pavement structure over the analysis period. For this research, the preservation strategy alternatives include preventive maintenance and rehabilitation. The preventive maintenance strategy includes thin preservation treatments that are applied regularly to a pavement, thereby maintaining the pavement in relatively good condition. The preventive maintenance strategy also includes routine maintenance activities. Average annual maintenance costs are added to the preventive maintenance strategy for the years when preventive maintenance treatments are not applied. The rehabilitation strategy includes a structural overlay applied to the pavement at some point in time during the analysis period, which is determined using the AASHTO pavement design procedure. In addition, the rehabilitation strategy includes routine maintenance activities and occasional thin surface treatments when required, depending on the serviceability and the skid resistance of the pavement section.

The preservation strategies models component allows the user to input a serviceability threshold and a skid resistance threshold, which are unique aspects of this LCCA model. If either the predicted serviceability or the predicted skid resistance deteriorates below its corresponding threshold value, a maintenance action is triggered. The term maintenance action may refer to a preventive maintenance treatment for the preventive maintenance strategy or a surface treatment for the rehabilitation strategy. The inclusion of a serviceability threshold and a skid resistance threshold allows the user to derive and analyze preservation strategies that maintain pavement serviceability (ride quality) and skid resistance (safety).

The remainder of this chapter describes the generation of preservation strategies, including the preventive maintenance strategy, followed by the structural overlay design procedure. Pavement serviceability and skid resistance models are addressed for each preservation strategy, as well as the scheduling of preventive maintenance treatments and rehabilitation. Finally, a summary of the preservation strategies models component is presented.

4.1 PREVENTIVE MAINTENANCE

As discussed in Chapter 2, the objective of preventive maintenance is to extend the functional life of a pavement by applying relatively inexpensive treatments before the pavement deteriorates to a condition that requires corrective maintenance. There are several types of

preventive maintenance treatments that are applied to pavements in the United States today. However, only a limited amount of data describing the effects of preventive maintenance exists. As more long-term data are collected, the effects of this preservation strategy will become more apparent and performance models can be refined, resulting in better analyses.

For LCCA purposes, the designer is interested in the immediate effects as well as the long-term effects of preventive maintenance on the pavement life. Regarding immediate effects, research documented by Al-Mansour and Sinha (1994) showed that preventive maintenance is most effective when applied to a pavement in relatively good condition. As the pavement condition deteriorates, preventive maintenance treatments have decreasing immediate effects on improving the pavement condition. The theory presented by Al-Mansour and Sinha (1994) is accepted and applied to this research based on the knowledge that preventive maintenance treatments are thin surface treatments. Therefore, if the existing pavement is extremely rough, the surface treatment is simply an application of a thin layer of asphalt that conforms to the same rough profile as the pavement underneath.

An extensive literature search revealed limited and ambiguous information on the long-term effects of preventive maintenance. For example, Hickes, et al. (1997) presented information regarding the expected life of a typical preventive maintenance treatment, while Geoffroy (1996) documented results from a survey on the observed increase in pavement life due to a preventive maintenance treatment. However, no literature was found that addressed the long-term effects of preventive maintenance based on the pavement condition at the time of application.

As discussed above, Al-Mansour and Sinha (1994) reported that the immediate effects of preventive maintenance vary depending on the pavement condition at the time of application. Therefore, it seems logical that the long-term effects of preventive maintenance should also vary based on the pavement condition at the time of application. For example, one would not expect the increase in pavement life due to a chip seal applied to a pavement with a PSI of 3.5 to be the same as the increase in pavement life due to a chip seal applied to a pavement with a PSI of 2.0. Since no literature regarding the long-term effects of preventive maintenance was found, a unique modeling approach was developed for this research. This modeling approach is described in the following section.

4.1.1 Modeling the Effects of Preventive Maintenance on Serviceability

For LCCA, there are two key model components that are required in order to capture the effects of preventive maintenance on an existing pavement. The first model component describes the immediate effect of the preventive maintenance treatment on the serviceability of the

pavement. The second model component describes the long-term effects of the treatment on the serviceability of the pavement.

4.1.1.1 Immediate Effects of Preventive Maintenance

For this research, the model provided by Al-Mansour and Sinha (1994) was used to describe the change in PSI immediately after a surface treatment is applied. This model was presented as Equation 2.34 and is repeated below for convenience.

$$\Delta PSI = 0.3325 * (PSI - 1.433) \quad (4.1)$$

where: ΔPSI = gain in pavement serviceability due to treatment application, and
 PSI = PSI at time of preventive maintenance treatment application.

The Al-Mansour and Sinha (1994) model can be used to determine the timing for preventive maintenance applications such that the optimal effectiveness is obtained from each treatment. In this case, determining the optimal effectiveness refers to maximizing ΔPSI for a given initial serviceability. It is assumed that the pavement serviceability after preventive maintenance is applied does not improve beyond the initial serviceability of the pavement. Based on this assumption and Equation 4.1, the timing for optimal effectiveness can be estimated. For example, if a pavement has an initial PSI of 4.2, then the optimal effectiveness of a surface treatment (ΔPSI) is approximately 0.7, which is obtained by applying the surface treatment at the point in time when the PSI of the pavement is 3.5. The PSI value at which the optimal effectiveness is obtained is used as the PSI threshold for preventive maintenance. Therefore, the pavement condition is maintained throughout the analysis period and the optimal effectiveness is obtained from each preventive maintenance treatment application.

4.1.1.2 Long-Term Effects of Preventive Maintenance

A model that describes the long-term effects of preventive maintenance, or the trend in PSI over time, is the second model component that is required for this LCCA. The AASHTO pavement design model may be used to describe the trend in PSI over time for a pavement prior to the application of a preventive maintenance treatment. The AASHTO model, shown as Equation 4.2, is used to predict the PSI at any point in time t for a specific pavement design.

$$PSI_t = PSI_i - (4.2 - 1.5) * 10^{ab} \quad (4.2)$$

$$a = 0.4 + \frac{1094}{(SN + 1)^{5.19}}$$

$$b = \log W_t - 9.36 \log(SN + 1) + 0.2 - 2.32 \log M_R + 8.07$$

where: PSI_t = PSI at end of year t,

PSI_i = initial PSI of pavement,

W_t = cumulative ESALs applied to pavement from time of construction to time t,

SN = structural number of pavement at time of initial construction, and

M_R = effective roadbed soil resilient modulus.

When a preventive maintenance treatment is applied to a pavement in relatively good condition, the PSI immediately improves, which should subsequently extend the life of the pavement. Previous research has indicated that while a preventive maintenance treatment does not add structural capacity to the existing pavement, it does maintain the pavement's structural capacity, thereby extending the life of the pavement (AASHTO, 1993). Thus, the AASHTO equation, without modification, is not an accurate model for describing the pavement serviceability after a preventive maintenance treatment is applied. Since no such model was found in the literature, a model was developed for describing the trend in PSI over time for preventive maintenance applications, while taking into consideration the condition of the existing pavement prior to each treatment application.

The dashed lines in the plots shown in Figure 4.1 display the trend in pavement serviceability (PSI) over time or traffic, as determined using the AASHTO flexible pavement design equation for an arbitrary design condition. The design is based on the following input values; 5000 ADT, 10 percent trucks, 0.38 truck factor, 3 percent growth rate, SN equal to 3.22, initial PSI equal to 4.2, terminal PSI equal to 2.0.

Figure 4.1 Trends in Pavement Serviceability Over Time Or Traffic With and Without Preventive Maintenance.

The shape characteristics of the pavement performance curves vary, depending on the units assigned to the horizontal axis. For example, the performance curve most commonly shown in the literature is PSI versus cumulative ESALs plotted on a logarithmic scale. This curve gives the appearance of accelerated pavement deterioration over time (or cumulative traffic). However, the graph showing the change in PSI with cumulative ESALs on a linear scale reveals that accelerated pavement deterioration does not occur over time (or cumulative traffic) for this design condition, but instead a relatively linear relationship exists. Therefore, ESAL applications in year thirty do not result in more damage to the pavement than an equivalent number of ESAL applications in year one, as may appear to be the case in the PSI versus log ESALs graph.

Figure 4.1 shows performance curves for a single example of the AASHTO flexible pavement design equation. The PSI versus log ESALs performance curve has a convex shape for all combinations of feasible design equation parameters (structural number, initial PSI, terminal PSI, ESALs). However, as the initial pavement design becomes stronger, or the structural number increases ($SN > 3.25$), the PSI versus ESALs performance curve has a concave shape, or begins to “flatten out” as cumulative ESALs increase (Fwa, 1990). When the performance curve assumes a concave shape, there is decelerated pavement deterioration, meaning that ESAL applications in year thirty result in less damage than an equivalent number of ESAL applications in year one. This theory was the basis for developing the long-term pavement performance curves for a preventive maintenance strategy in this research.

In an attempt to reflect the “flattening out” of the pavement condition versus time or traffic performance curve after a preventive maintenance treatment is applied, the structural number and initial PSI used in the AASHTO design equation are altered. More specifically, the structural number is increased by 0.22 every time a preventive maintenance treatment is applied. The increase in structural number of 0.22 was derived by assuming a 0.5-inch application of asphalt concrete with a layer coefficient equivalent to 0.44. The new initial PSI value is equal to the PSI value immediately prior to the treatment application plus the • PSI value obtained from Al-Mansour and Sinha’s model. The pavement condition versus time or traffic curve for preventive maintenance is obtained by using the AASHTO flexible pavement design equation that was developed for the initial pavement design with the appropriate ESALs (including traffic volume growth rate), the altered structural number, and the new initial PSI value. The solid lines in the plots shown in Figure 4.1 reflect the long-term effects of preventive maintenance over time and traffic, obtained by the unique method developed for this research.

The method developed for modeling the long-term effects of preventive maintenance is accepted for this research because researchers have found that preventive maintenance does

preserve the pavement, thereby extending the pavement life (AASHTO, 1993). However, as mentioned previously, researchers have also found that the increase in structural capacity of a pavement after the application of a thin surface treatment is negligible (AASHTO, 1993). Therefore, it is not correct to use the altered structural number for future design purposes. At the time when corrective maintenance, rehabilitation, or reconstruction is required, the structural number of the existing pavement, or the effective structural number, should be determined by nondestructive testing methods, as recommended in the AASHTO Design Guide (1993).

4.1.2 Modeling Skid Resistance

Skid resistance is a unique aspect of this LCCA model that allows the user to consider the effects of surface aggregate type on skid resistance and accidents. Similar to serviceability, a model that describes the trend of skid resistance over time or traffic is required. The LCCA model developed for this research requires the user to select a surface aggregate type of either skid resistant aggregate, or SRA, (gravel) or unrestricted aggregate (limestone that does not qualify as SRA). Therefore, separate models for describing the trend in skid resistance over time for SRA and for unrestricted aggregate are required.

The skid resistance models incorporated into this LCCA were linear relationships taken from the existing literature presented in Chapter 2. The skid resistance model developed by Dierstein (1977) is used to predict the skid resistance of a pavement at any given time when SRA is selected as the surface aggregate type. Dierstein's research showed that the skid resistance for gravel (a type of SRA used in West Virginia) remained above an SN_{40} of 40, regardless of the traffic volumes. Therefore, a linear relationship with a slope of zero and an intercept of 42.5, taken from a graph presented by Dierstein (1977), is assumed for the skid resistance model when the user selects SRA as the surface aggregate type.

When the user selects unrestricted aggregate as the surface aggregate type, a linear relationship developed by Quinn (1975) is used to predict the skid resistance of the pavement at any given time. Quinn reported that the use of carbonate rock aggregate (limestone) in the surface course implies a more rapid decay of skid resistance with cumulative vehicle passes. The skid resistance model used for this research was presented as Equation 2.23, which is repeated below for convenience.

$$SN_{40} = (-0.3 * v) + 40.5 \quad (4.3)$$

where: SN_{40} = skid number at 40 mph, and

v = number of cumulative vehicle passes (in millions) since initial pavement construction or previous surface treatment application.

Since both relationships for SRA and for unrestricted aggregate are linear relationships, the slope and intercept are included in the LCCA program as default values, based on the surface aggregate type selected by the user. However, the user has the option to change these default values for the slope and/or intercept if so desired.

4.1.3 Establishing the Timing for Preventive Maintenance Treatments

Preventive maintenance is applied to pavements in good structural condition to extend the life of the pavement in a cost-effective manner (Zaniewski and Mamlouk, 1996). Therefore, the timing of treatment applications is a very important aspect of this alternative. For this research, there are two key factors that influence the timing of preventive maintenance treatments. The first factor is the serviceability of the pavement, and the second factor is skid resistance.

Section 4.1.1.1 described how to obtain the optimal effectiveness of a preventive maintenance treatment using the model presented by Al-Mansour and Sinha (1994). The optimal effectiveness of a preventive maintenance treatment is obtained when the treatment is applied to a pavement at a certain PSI level, which is determined based on the initial PSI of the pavement. The PSI level at which the optimal effectiveness is obtained is established as the PSI threshold for preventive maintenance. Therefore, when the pavement serviceability deteriorates below the PSI threshold, a preventive maintenance treatment is scheduled.

The second factor that influences the timing of preventive maintenance treatments is skid resistance. A skid resistance threshold is established so that if the predicted skid resistance of the pavement deteriorates below the threshold value, a preventive maintenance treatment is scheduled. The skid resistance threshold is defined in terms of a skid number at 40 mph as measured with a skid trailer (SN_{40}). The allowable range for the skid resistance threshold is $SN_{40} = 38$ to $SN_{40} = 0$. Selecting a skid resistance threshold of zero allows the user to eliminate the possibility that a preventive maintenance treatment is required due to low skid resistance. The

default value for the skid resistance threshold is set at $SN_{40} = 32$, although the user has the option to select another threshold value within the allowable range.

Within a single iteration, the serviceability and skid resistance are calculated for each year of the analysis period. If the serviceability (PSI) is less than the PSI threshold or the skid resistance (SN_{40}) is less than the skid resistance threshold, a preventive maintenance treatment is scheduled. Otherwise, annual routine maintenance costs are incurred during that year. This procedure is outlined in Figure 4.2.

Figure 4.2 Flowchart for Establishing Timing of Preventive Maintenance Treatments.

The process for deriving a preventive maintenance strategy described in this section is repeated until the simulation is complete. For each year within a single iteration of the simulation, the type of maintenance activity performed (either preventive maintenance or routine maintenance), the pavement serviceability, and the skid resistance are saved in arrays for use in calculating agency and user costs. The type of maintenance activity performed during a particular year is used to estimate agency costs for that year. The serviceability during a particular year is used to determine annual routine maintenance costs. Finally, the skid resistance of the pavement during a particular year is used to predict wet weather accident experience, which is necessary for calculating user costs associated with accidents.

4.2 REHABILITATION

The purpose of rehabilitation is to improve the load carrying capacity of the existing pavement structure over the analysis period. Rehabilitation is performed at the time when the initial (or existing) pavement reaches its terminal serviceability level. Typically, the designer specifies a performance period, or a time when the rehabilitation should occur. For this research, the performance period is defined as the time from when the pavement is constructed to the time at which either a 1.5-, 2.0-, or 2.5-inch overlay is required.

The rehabilitation strategy is derived using the AASHTO overlay design approach combined with risk analysis, which introduces variability into the derivation. Chapter 2 described the AASHTO overlay design approach in detail. Note that the condition survey, deflection testing, and coring and materials testing (Steps 3,4, and 5 of the AASHTO procedure described in Chapter 2) are not performed in this study, as this research is strictly theoretical. The following section provides a detailed description of the approach used to design the structural overlay for this research.

4.2.1 Detailed Description of the Structural Overlay Design

The model used to derive the structural overlay design works in conjunction with the initial pavement design model. An iterative process is used to determine a performance period such that a specified overlay thickness is required. The flowchart in Figure 4.3 outlines the structural overlay design procedure used in this research.

Figure 4.3 Flowchart for Structural Overlay Design (page 1).

Figure 4.3 Flowchart for Structural Overlay Design Continued (page 2).

The first step of the structural overlay design procedure is to determine the future traffic prediction from the time of the overlay to the end of the analysis period. The future ESALs are determined using the same random values selected in the initial pavement design for the ADT, percent trucks, truck factor, and growth rate. This maintains consistency between the initial traffic prediction and the future traffic prediction for a particular iteration. For example, if a high ADT is randomly selected for the initial traffic prediction, a high ADT should also be used for the future traffic prediction. A future traffic prediction probability distribution is derived based on the future ESALs predicted for each iteration of the simulation.

The next step in the overlay design procedure is to determine the required structural number for the overlay design period, or the time from which the overlay is applied to the end of the analysis period. First, the minimum required structural number (SN) is determined based on the traffic prediction. The minimum required structural number is calculated using Equation 2.2 with the minimum layer thicknesses suggested by AASHTO (Table 2.1), and the mean layer coefficients and drainage coefficients input by the user. This is a minimum value for the structural number, and is used as a starting point for determining the required SN.

The performance prediction, in log ESALs, is determined using randomly selected values for the initial PSI and material properties because the parameters may not necessarily be the same for the overlay as the initial pavement. The structural number for the overlay design also varies from the SN used for the initial pavement design, because the expected future traffic load is not necessarily equal to the initial traffic prediction.

If the performance prediction mean is less than the traffic prediction mean, the required SN is increased by 0.2, and a new performance prediction is determined. The SN increase is an arbitrary number, but 0.2 was chosen because it is approximately equal to the increase in SN due to adding one-half inch to the surface thickness (assuming the layer coefficient for the surface layer is 0.44).

If the performance prediction mean is greater than or equal to the traffic prediction mean, the point of intersection and probability of failure are determined in the same manner as described for the initial pavement design model in Chapter 3. If the probability of failure is greater than the probability of failure threshold input by the user, the required SN is increased by 0.2, and a new performance prediction is determined. When the probability of failure becomes less than or equal to the probability of failure threshold, the required SN has been determined, and the average and standard deviation for the required SN are calculated.

The next step of the overlay design procedure is to determine the effective structural number, or the structural number of the existing pavement. First, the ESALs to date must be

calculated. Since this factor cannot be measured at the design stage, the expected traffic, or the traffic prediction, is used. Therefore, the ESALs to date for iteration i is equal to the randomly estimated traffic prediction used in the initial pavement design for iteration i .

The next step in determining the effective SN is to determine the ESALs to failure. The AASHTO design guide states that when calculating ESALs to failure, a failure PSI equal to 1.5 and a reliability of 50 percent should be used to be consistent with the AASHO Road Test and the development of the design equations (AASHTO, 1993). Because of the deterministic nature of the AASHTO equations, it is simple to predict the difference in performance of a design with a reliability of 90 percent versus a design with a reliability of 50 percent. This is illustrated in Figure 4.4. Using a failure PSI of 1.5 and a reliability of 50 percent increases the number of ESALs to failure. This increase in the ESALs to failure drives the remaining life of the pavement toward one, which results in a higher effective SN and therefore requires a smaller overlay thickness.

Since the risk-analysis approach to pavement design does not include the reliability term in the design equation, this shift in ESALs to failure is not considered. Instead, the ESALs to failure are determined by randomly selecting, based on distributions input by the user, an initial structural number, initial PSI, and effective roadbed soil resilient modulus. The terminal PSI is a discrete value set equal to 1.5, and the AASHTO equation without the reliability and standard deviation terms is used to calculate the ESALs to failure.

Figure 4.4 Variations in Reliability Levels.

Next, the remaining life is calculated using Equation 2.39, and the corresponding effective SN is then calculated using Equation 2.40. The effective SN is the product of the condition factor and the initial SN. Note that the minimum condition factor is 0.5, meaning the effective structural number will never be less than half of the initial structural number. For consistency between the initial pavement design and overlay design, the initial SN used in the effective SN calculation is the same SN used to determine the ESALs to failure.

After the effective and required structural numbers are determined, the overlay thickness is calculated. The required SN value from each iteration is subtracted from the effective SN value from the same iteration, and the difference is divided by the surface layer coefficient (Equation 2.41). The result of this division is the required overlay thickness (in inches).

The overlay thickness determined during each iteration is placed in a “bin” representing the actual overlay thickness that would be required. For example, if the overlay thickness is less than or equal to 0, then no overlay is necessary. If the overlay thickness is greater than 0, but less than or equal to 0.5-inch, then a 0.5-inch overlay is required. Similarly, if the overlay thickness is greater than 0.5-inch, but less than or equal to 1 inch, then a 1-inch overlay is required, etc.

When the simulation is complete (after a specified number of iterations are performed), the average overlay thickness is determined. If the average overlay thickness is greater than the thickness specified by the user, the performance period is increased by one year, and the entire process, including the initial pavement design model and overlay design model, is simulated again. When the average overlay thickness becomes less than or equal to the user-specified thickness, the simulation is complete.

4.2.2 Maintaining Serviceability and Skid Resistance for Rehabilitation Strategy

After the performance period and average overlay thickness are determined for the rehabilitation strategy, the LCCA program determines a schedule for surface treatments and routine maintenance activities over the analysis period. This schedule is based on the year that the overlay is performed, as well as the serviceability and skid resistance of the pavement over time. The remainder of this section describes the serviceability and skid resistance thresholds established for the rehabilitation strategy. Next, the models used for describing the serviceability and skid resistance over time are defined. Finally, the procedure for determining a schedule for maintenance activities for the rehabilitation strategy is described.

4.2.2.1 Establishing Serviceability and Skid Resistance Threshold Values

For the rehabilitation strategy, the serviceability and skid resistance of the pavement are maintained above their corresponding threshold values in a similar manner as that described for

the preventive maintenance strategy. However, there are several differences between the techniques used to establish the serviceability threshold value for each preservation strategy. The techniques used for the rehabilitation strategy are described in this section.

Similar to the preventive maintenance strategy, the PSI threshold for the rehabilitation strategy is defined as the value above which the pavement serviceability is to be maintained. However, there is a difference between the level of serviceability that is typically maintained for a rehabilitation strategy versus a preventive maintenance strategy. Since rehabilitation is performed when the pavement deteriorates to a terminal PSI value, the level of serviceability associated with this preservation strategy tends to be significantly lower than the level of serviceability associated with a preventive maintenance strategy.

For the rehabilitation strategy, the user selects a value for the PSI threshold. The default value for the PSI threshold for rehabilitation is 2.0, although the user may select any value within the range of 0 to 3.0 for the PSI threshold. The range of allowable PSI threshold values includes relatively low PSI values (less than 3.0) so that a distinct difference between the two preservation strategies is maintained.

The skid resistance threshold for the rehabilitation strategy is the same value as the skid resistance threshold for the preventive maintenance strategy. As described previously, the skid resistance threshold is a value that may range from $SN_{40} = 38$ to $SN_{40} = 0$. A skid resistance threshold of $SN_{40} = 32$ is established as the default value, although the user may change the threshold to any other value within the allowable range.

4.2.2.2 Modeling Serviceability and Skid Resistance Over Time

Similar to the preventive maintenance strategy, models are needed to describe the serviceability and skid resistance over time for the rehabilitation strategy. Serviceability is modeled using the AASHTO design equation, presented earlier in this chapter as Equation 4.2. When calculating the PSI at a point in time prior to the structural overlay, the structural number (SN) derived in the initial pavement design is used in the AASHTO equation. After the structural overlay is performed, the required SN derived in the structural overlay design is used in the AASHTO equation. Also, it was assumed that the PSI immediately after a structural overlay is applied returns to the initial PSI of the pavement, based on research presented by deSolminihaç and Hudson (1995). Therefore, the initial PSI input by the user for the initial pavement design is used in the AASHTO equation for estimating the PSI at any point in time during the analysis period.

The rehabilitation strategy may also include occasional surface treatments if the serviceability or skid resistance deteriorates below its corresponding threshold value. The effect

of a surface treatment is not modeled for the rehabilitation strategy, because the PSI level at which a surface treatment is required is less than 3.0. Previous research has shown that when a surface treatment is performed on a pavement in poor condition (PSI less than 3.0), the effect of the surface treatment is minimal (Al-Mansour and Sinha, 1994). Since the surface treatment does not add structural capacity to the pavement or extend the life of the pavement, the application of a surface treatment to a pavement in poor condition does not have a significant effect on the pavement serviceability. Thus, no changes need to be made to the AASHTO equation for modeling PSI over time if a surface treatment is required for the rehabilitation strategy since the PSI at the time of the surface treatment application will not be greater than 3.0.

Skid resistance is modeled for the rehabilitation strategy in exactly the same manner as the preventive maintenance strategy described in Section 4.1.2. A linear model for skid resistance versus cumulative vehicle passes is assumed. The slope and intercept parameters for the skid resistance model are dependent on the surface aggregate type selected by the user. Default values for the slope and intercept are provided, although the user may change either one or both of these values if desired.

4.2.2.3 Scheduling Maintenance Activities for Rehabilitation Strategy

As described previously, the rehabilitation strategy includes a structural overlay at some point in time during the analysis period, routine maintenance activities, as well as occasional surface treatment applications, if necessary. The year during which a structural overlay is required is determined using the AASHTO structural overlay design procedure. For the remaining years throughout the analysis period, the type of maintenance activity performed during a particular year (either a surface treatment or routine maintenance) is dependent on the predicted serviceability and skid resistance of the pavement. A flowchart outlining the procedure for establishing the schedule for maintenance activities for the rehabilitation strategy is shown in Figure 4.5.

For each year of the analysis period, a maintenance activity is scheduled. If the year under analysis is equal to the end of the performance period, a structural overlay is scheduled. Otherwise, the pavement serviceability and skid resistance for that particular year are calculated. If either the PSI or the skid resistance is less than its corresponding threshold value, and the year under analysis is not within two years prior to the structural overlay application or the end of the analysis period, then a surface treatment is scheduled for that year. Otherwise, annual routine maintenance costs are incurred.

Figure 4.5 Flowchart for Establishing a Schedule for Maintenance Activities for the Rehabilitation Strategy.

When the end of the analysis period is reached, the entire process is iterated starting with the initial pavement design derived during the same iteration of the initial pavement design procedure. This procedure is repeated until the simulation is complete. For each year within a single iteration, the type of maintenance activity scheduled (either a structural overlay, routine maintenance, or a surface treatment), the pavement serviceability and the skid resistance are saved in arrays for use in calculating agency and user costs.

4.3 SUMMARY OF PRESERVATION STRATEGIES MODELS

The preservation strategies considered in this LCCA model include preventive maintenance and rehabilitation. Preventive maintenance treatments are applied regularly to a pavement in relatively good condition. Routine maintenance is assumed each year that a preventive maintenance treatment is not scheduled. The rehabilitation strategy includes a structural overlay applied to the pavement at some point in time during the analysis period. Routine maintenance and occasional surface treatments, if required, are also included in the rehabilitation strategy.

The serviceability and skid resistance of the pavement influence the scheduling of maintenance activities for both preservation strategies. The level of pavement serviceability maintained is dependent on the type of preservation strategy. For example, a high level of serviceability is maintained for the preventive maintenance strategy, since preventive maintenance treatments are most effective when applied to pavements in relatively good condition (PSI greater than 3.0). Skid resistance is maintained to the same degree for both preservation strategies. By maintaining certain levels of PSI and skid resistance, this LCCA model generates preservation strategies that provide adequate serviceability as well as safety for the pavement user over the analysis period.

Finally, the maintenance activity schedule derived during each iteration for each preservation strategy is saved for use in calculating agency and user costs. In addition, the serviceability and skid resistance associated with each year for each preservation strategy are also saved for use in calculating agency and user costs.

CHAPTER 5

LIFE-CYCLE COST MODELS

The life-cycle cost analysis developed for this research includes both agency and user costs that are discounted to a single net present worth value. Agency costs are costs incurred by the highway agency and include costs associated with the initial cost of construction, routine maintenance, rehabilitation, and preventive maintenance. The salvage value for each alternative is considered as a negative agency cost. User costs are costs incurred by the user and include costs due to accidents, user delay, and excess vehicle operating costs. Figure 5.1 is a flowchart that illustrates the general process used to determine the life-cycle costs associated with both preservation strategies.

This chapter presents a brief explanation of the present worth concept and then defines the present worth factor. Next, agency cost calculations associated with the rehabilitation strategy and the preventive maintenance strategy are discussed, followed by the procedures used to calculate user costs.

5.1 THE CONCEPT OF PRESENT WORTH COST

In order to make a proper comparison between the rehabilitation and preventive maintenance strategies, the costs associated with each preservation strategy must be converted to a common measure. For this research, the net present worth method was used with constant dollars and real discount rates. Constant (or real) dollars are dollars that have constant purchasing power over time (Walls and Smith, 1998). Thus, the cost of performing an activity (such as a surface treatment) would not change as a function of the future year in which it is applied. Real discount rates reflect the true time value of money with no inflation premium (Walls and Smith, 1998). Real discount rates should be used in conjunction with non-inflated dollar cost estimates of future investments, or constant dollars.

Figure 5.1 Flowchart for Present Worth Cost Calculations.

The net present worth of an alternative is the discounted monetary value of expected net costs and/or benefits. It involves the discounting of all future sums to the present, using an appropriate discount rate (AASHTO, 1993). The present worth factor is defined as:

$$PWF = \frac{1}{(1+i)^n} \quad (5.1)$$

where: PWF = present worth factor,

i = discount rate, and

n = year of expenditure.

Once the agency and user costs over the analysis period are established for a particular alternative, the future costs are discounted to the initial year and then added to the initial cost of construction. The result is the net present value of the LCCA alternative. The general net present value formula for discounting annual agency and user costs associated with a particular preservation strategy over an analysis period of N years is:

$$NPV = \text{Initial cost} + \sum_{k=1}^N (\text{Agency Costs} + \text{User Costs})_k \left[\frac{1}{(1+i)^k} \right] \quad (5.2)$$

where: NPV = net present value,

N = analysis period (in years),

i = discount rate, and

k = year of expenditure.

The NPV for the LCCA developed for this research includes both agency costs and user costs, which are described in detail in the following sections. Once the NPV for each alternative is determined, the alternatives may be compared based on this common factor.

5.2 AGENCY COST CALCULATIONS

Agency costs are costs directly incurred by the highway agency due to construction or maintenance activities. Agency costs include costs associated with the initial construction of the pavement and rehabilitation or preventive maintenance strategies. For this research, the highway agency may incur costs associated with rehabilitation from the structural overlay, annual routine maintenance, and occasional surface treatments, if necessary. For the preventive maintenance alternative, highway agencies may be responsible for the cost of regular preventive maintenance treatments as well as annual routine maintenance. Finally, the salvage value, or the value of the pavement at the end of the analysis period, is considered as a negative agency cost for both preservation strategies. The methods used to estimate agency costs for initial pavement construction, rehabilitation and preventive maintenance, and the salvage value are described in the following sections.

5.2.1 Initial Pavement Construction Costs

The method used for estimating agency costs associated with the initial pavement construction is relatively straightforward. The costs are estimated for a 2-lane, 1-mile section of pavement derived in the initial pavement design based on user inputs. As described previously, the pavement design consists of three layers, including the surface, base, and subbase. Agency costs are based on the quantity and unit cost of materials required for each pavement layer. The surface layer consists of hot-mix asphalt (HMA), where the unit cost of HMA is typically given in dollars per ton. The base and subbase unit costs are typically given in dollars per cubic yard (cy). Therefore, the following conversions were required in order to compute the agency costs associated with initial pavement construction:

- PAVEMENT SECTION LENGTH - convert 1 lane-mile to yards:
 $(\text{mile})(5280 \text{ ft}/1 \text{ mile})(1 \text{ yd}/3 \text{ ft}) \bullet 1 \text{ lane-mile} = 1760 \text{ yards}$
- PAVEMENT WIDTH - convert two 12-foot lanes to yards:
 $(2 \times 12 \text{ ft})(1 \text{ yd}/3 \text{ ft}) \bullet 24 \text{ ft} = 8 \text{ yards}$
- LAYER THICKNESSES - convert inches to yards:
 $(\text{inches})(1 \text{ ft}/12 \text{ inches})(1 \text{ yd}/3 \text{ ft}) \bullet 1 \text{ inch} = 1/36 \text{ yard}$
- UNIT COST - convert HMA \$/ton to \$/cy:
 $(\$/\text{ton})(1 \text{ ton}/2000 \text{ pounds})(4000 \text{ pounds}/1 \text{ cy}) \bullet \$1/\text{ton} \bullet \$2/\text{cy}$

The unit costs for each layer type are described by probability distributions defined by the user. Truncated normal probability distributions were used so that the probability of a negative cost was eliminated. Default values for the means and standard deviations are provided, although the user has the option to change these values if desired.

For each iteration of the simulation, required layer thicknesses are determined, as described in Chapter 3. In addition, a unit cost is selected for each pavement layer based on the probability distributions defined by the user. The quantity of material (in cubic yards) required for each pavement layer is multiplied by the unit cost of the material (in dollars per cubic yard). The quantity of material required for a particular pavement layer is determined by multiplying the required layer thickness in inches times the section length (1 mile) times the section width (two 12-foot lanes), using the unit conversions described previously. Finally, the initial cost of pavement construction is determined by adding together the products obtained from multiplying the quantity of material required times the unit cost for each layer.

The initial cost of construction is calculated each iteration of the performance prediction loop in the initial pavement design procedure. After the simulation is complete, the initial cost of construction is described by a probability distribution. This distribution of costs reflects the variability associated with the traffic and performance predictions. Since the costs of constructing the pavement are incurred by the agency at the present time, no present worth adjustments need to be made.

5.2.2 Rehabilitation Costs

In addition to the initial cost of construction, agency costs associated with rehabilitation include the cost of the structural overlay, annual routine maintenance costs, and the costs of surface treatments, if necessary. The cost of the structural overlay is calculated in a similar manner as the initial cost of construction. However, the actual overlay consists only of a single layer of HMA. Therefore, the estimated cost of the overlay is determined by selecting a unit cost value from the surface layer unit cost probability distribution and then multiplying the unit cost of HMA (in dollars per cubic yard) times the overlay thickness times the length of the pavement section times the width of the pavement section. Again, the conversion factors defined in the previous section are used for this calculation.

Chapter 4 described the method for establishing the rehabilitation strategy. The structural overlay design procedure was derived so that the average required overlay thickness is a specified thickness. Each iteration of the overlay design simulation results in a required overlay thickness that is used to estimate the overlay cost. Since the overlay is performed in some future year, the

overlay cost must be multiplied by a present worth factor (Equation 5.1) to determine the present worth cost of the overlay. The present worth costs obtained from each iteration of the overlay design simulation are combined to form a distribution of present worth overlay costs.

The rehabilitation strategy may also require the application of thin surface treatments if either the pavement condition or the skid resistance deteriorates below its corresponding threshold value. The agency costs associated with surface treatments are dependent on the unit cost of a surface treatment, which is also defined by a probability distribution. The advanced user option input screen allows the user to change the default distribution for the unit cost of a surface treatment, which is in units of dollars per lane-mile. For each iteration, a value is selected from the surface treatment unit cost probability distribution. Then, for each year that a surface treatment is required throughout the analysis period, the cost of each surface treatment is converted to present worth dollars. After the simulation is complete, the present worth costs associated with surface treatments from each iteration are combined to form a distribution of present worth costs for surface treatments that are required for the rehabilitation strategy. This distribution may have a wide spread, or a relatively high standard deviation, because each iteration of the simulation may require a different number of surface treatments and/or different timings for each treatment application.

Finally, if a structural overlay or surface treatment is not required for a particular year, annual routine maintenance costs are assumed. Typical routine maintenance costs are associated with activities such as crack sealing, patching, and maintaining drainage structures. Sinha, et al. (1994) developed a model for estimating annual maintenance costs (AMC) based on the PSI of the pavement section at the point in time that the maintenance is performed. On the other hand, some highway agencies use a single value for approximating annual routine maintenance costs. For example, Pennsylvania estimates annual routine maintenance costs for flexible pavements as \$1,825 per lane mile, while maintenance costs in Nevada are estimated at \$1,000 per year per directional mile for flexible pavements (Walls and Smith, Case Study Reports).

For this research, routine maintenance costs are estimated using the model developed by Al-Mansour and Sinha (1994), which was presented as Equation 2.33 in the literature review and is repeated below for convenience.

$$\log(AMC) = a + (b * PSI) \quad (5.3)$$

where: AMC = annual roadway maintenance expenditure (\$/lane-mile),

PSI = PSI at time of maintenance,

$$a = 4.0283 \text{ for AADT} > 2000 \text{ or } 3.7781 \text{ for AADT} \leq 2000, \text{ and}$$

$$b = -0.4621 \text{ for AADT} > 2000 \text{ or } -0.4252 \text{ for AADT} \leq 2000.$$

It should be noted that the annual maintenance cost model presented by Al-Mansour and Sinha (1994) was developed based on data collected from 1984 through 1987. Since the model yields annual maintenance cost estimates in 1987 dollars, the cost estimates must be converted to present day dollars. To convert 1987 dollars to current year dollars, an escalation factor for the dollar value of time is determined using changes to the *All Items Component* of the Consumer Price Index (CPI) for the base year (1987) and the current year (1999). The average *All Items Component* of the CPI was 113.6 and 166.6 in years 1987 and 1999, respectively (Bureau of Labor Statistics, 2000). The escalation factor is determined by dividing the current year CPI by the base year CPI. For this model, the escalation factor is equal to 166.6/113.6, or 1.47. The estimated routine maintenance cost in current year dollars can be determined by multiplying the cost obtained from Al-Mansour and Sinha's model by an escalation factor of 1.47.

Since a nonlinear relationship exists between PSI and the annual routine maintenance cost in the model developed by Al-Mansour and Sinha (1994), the costs must be estimated each year, based on the PSI at the time that routine maintenance is performed. For each year throughout the analysis period that routine maintenance is performed, the estimated costs for future years are converted to present worth dollars and added together to obtain a total present worth cost for routine maintenance. Each iteration, a present worth cost for routine maintenance over the entire analysis period is calculated. Upon completion of the simulation, the individual total present worth costs calculated during each iteration are combined to form a probability distribution describing the net present value of routine maintenance for the rehabilitation strategy.

5.2.3 Preventive Maintenance Costs

The present worth cost of a preventive maintenance strategy is estimated in a similar manner as described above for the rehabilitation strategy, except that there are no structural overlay costs incurred by the agency. Preventive maintenance costs include costs associated with each preventive maintenance treatment applied to the pavement over the analysis period as well as routine maintenance costs.

A preventive maintenance treatment is applied when either the pavement condition or skid resistance deteriorates below its corresponding threshold value. The pavement condition threshold value for preventive maintenance is established within the computer program, based on

the PSI value that yields the optimal effectiveness of a preventive maintenance treatment, as described in Chapter 4. The skid resistance threshold is established by the user. The agency costs associated with preventive maintenance are dependent on the unit cost of a surface treatment, as described in the previous section. Within a single iteration, for each year that a preventive maintenance treatment is applied throughout the analysis period, the cost of each treatment is estimated, and then converted to present worth dollars. After the simulation is complete, the present worth costs associated with preventive maintenance treatments are combined to form a distribution of present worth costs. The variability of this distribution is related to the variability associated with the pavement performance. If there is high variability in the pavement performance, there will be high variability in the number and timing of preventive maintenance treatments required over the analysis period.

If a preventive maintenance treatment is not required during a particular year of the analysis period, routine maintenance is assumed. The procedure used to estimate the present worth cost of routine maintenance is exactly the same as that described for the rehabilitation strategy. The output is a probability distribution defining the range and probability of routine maintenance costs that may be incurred by the agency over the analysis period.

5.2.4 Salvage Value

The salvage value, or the monetary value representing the forecasted worth of the pavement at the end of the analysis period, is considered as a negative agency cost. For this research, the salvage value is designated as a percentage of the original cost of the pavement, based on the condition of the pavement at the end of the analysis period. The salvage value is calculated in a similar manner for both the rehabilitation and preventive maintenance strategies.

Within a single iteration, the PSI at the end of the analysis period is estimated for each preservation strategy. Assuming a failure PSI of 1.5, the existing percent of the initial pavement condition is calculated using the following equation:

$$\text{percent existing} = \frac{PSI_N - 1.5}{PSI_i - 1.5} \quad (5.4)$$

where: *percent existing* = percent of initial PSI remaining at end of analysis period,

PSI_N = PSI at end of analysis period, and

PSI_i = initial PSI of pavement.

Note that if the PSI at the end of the analysis period is less than the failure PSI ($PSI_N < 1.5$), the salvage value is zero. Also, the initial PSI for a particular iteration is the same initial PSI used in the initial pavement design procedure for the same iteration.

The next step in determining the salvage value of each alternative is to multiply the percent of initial PSI remaining at the end of the analysis period times the initial cost of the pavement. This product yields the salvage value obtained in year N, which must then be multiplied by a present worth factor in order to convert the salvage value to present worth dollars. Finally, since the salvage value is considered as a negative cost, the present worth salvage value for each alternative is multiplied by negative one and then added to the total agency cost for the corresponding alternative. This process is iterated until the simulation is complete, and a distribution of present worth salvage values for each alternative is determined.

5.3 USER COST CALCULATIONS

User costs are costs incurred by the user of a pavement facility, including costs associated with accidents, user delay, and vehicle operation. These costs are difficult to quantify, and the values assigned to these factors are often disputed. For example, how does one assign a cost to an accident that resulted in a fatality? Furthermore, when traffic is delayed due to a work zone, is a truck driver's time more valuable than a typical passenger car driver's time? The answers to these questions and others may always be disputed. Therefore, the model developed in this research provides default values for unit costs associated with these factors based on the existing literature, but also allows the user to change the unit costs as desired.

User costs are broken down into two categories: user costs incurred during normal operation and user costs incurred during work zone conditions. Normal operation reflects user costs associated with using a facility during periods free of construction or maintenance activities (Walls and Smith, 1998). Work zone conditions reflect the user costs associated with using a facility during periods of construction or maintenance activities that generally restrict the capacity of the facility and disrupt normal traffic flow. The duration of normal operation and the duration of work zone conditions for each preservation strategy must be determined so that the user costs associated with each category may be quantified.

A general procedure is used for calculating user costs associated with normal operation and work zone conditions. However, some of the relationships used for normal operation may vary slightly from the relationships used for work zone conditions. For example, an accident rate may be used to predict the number of accidents occurring over a given time period. However, the

accident rate used to predict accidents during normal operation may be slightly lower than the accident rate used to predict accidents during work zone conditions. With this in mind, the procedures for estimating user costs due to accidents, delay, and excess vehicle operating costs are described below.

5.3.1 Accidents

User costs due to accidents are a function of the number and severity of accidents and the unit cost per accident for each severity level. The predicted number of accidents is broken down into normal operation and work zone operation, and then further broken down into wet accidents and dry accidents. Wet weather accident prediction is dependent on the skid resistance of the pavement surface. The skid resistance of a pavement at any given point in time may be estimated using the surface aggregate type and traffic volumes along with the appropriate relationships presented in Chapter 2. For this LCCA, the user selects a surface aggregate type of either skid resistant aggregate (SRA) or unrestricted. If “unrestricted” is selected for the surface aggregate type, the assumption is made that limestone aggregate is used in constructing the surface course.

5.3.1.1 Accidents During Normal Operating Conditions

For normal operating conditions, one of the following typical accident rates obtained from West Virginia Accident Data (1999) is used as a default value in the LCCA model, based on the route classification specified by the user (the user has the option to change the default value).

Rural Primary, 2 lanes (US and WV routes):	199 accidents/hmvm ¹
County and Local routes over 500 ADT:	241 accidents/hmvm

Typically, the appropriate accident rate above, or another value input by the user, is multiplied by the total number of vehicle-miles over a specified period of time. The product is the predicted number of accidents for a given section of pavement over a specified time period. However, the LCCA model developed in this research includes skid resistance and its effect on wet weather accidents. Therefore, a more detailed approach was used for predicting accidents.

Since skid resistance was incorporated into the LCCA model, the accident prediction must be separated into wet weather accidents and dry weather accidents. The accident rates shown above are average rates for all existing conditions. Weather history shows that in West Virginia, pavements are wet approximately 15 percent of the time. Theoretically, if a pavement has excellent skid resistance, the accidents would most likely be distributed such that 85 percent

of the total accidents occur on dry roads and 15 percent of the total accidents occur on wet roads. However, West Virginia Accident Data (1990-1999) shows that wet roads are a contributing factor in approximately 25 percent of all accidents in West Virginia. This may be a reflection of the assumption that as roads become wet, the skid resistance of the pavement decreases, thereby increasing the number of wet weather accidents.

Equation 5.5 may be used to estimate the dry weather accident rate for a particular pavement, given the typical accident rate (199 accidents/hmvm or 241 accidents/hmvm), the percent wet time (15 percent), and the statistic that 25 percent of all accidents in West Virginia occur on wet roads. The accident rate and percent wet time are values that may be changed by the user if so desired.

$$\text{dry accident rate} = \frac{\text{typical accident rate} * (1 - p)}{(1 - \text{wet time})} \quad (5.5)$$

where: *dry accident rate* = dry accidents/hmvm,

typical accident rate = accidents/hmvm,

p = percent of all accidents that occur on wet roads, and

wet time = percent of time roads are wet.

A linear relationship was used to describe the skid resistance of a pavement at any given point in time (or traffic). If skid resistant aggregate is selected for the surface aggregate type, data provided by Dierstein (1977) is used to describe the skid resistance versus cumulative traffic relationship. Dierstein (1977) reported that when gravel was used as the surface aggregate type, the skid resistance (SN_{40}) remained above 40, regardless of traffic volumes. Therefore, a slope of 0 is used, along with an intercept of 42.5. If unrestricted aggregate is selected for the surface aggregate type, a relationship by Quinn (1975) is used to estimate the skid resistance for a given number of cumulative vehicle passes over a particular time period. This relationship was developed based on pavement surfaces constructed with limestone aggregate only. In this case, a slope of -0.3 is used, along with an intercept of 40.5. Once again, the user has the ability to change the slope and intercept default values if desired.

¹ Accident rates are presented as accidents/hmvm, or accidents per one hundred million vehicle-miles.

After the skid resistance for a given time period is predicted, the wet weather accident rate is determined using a regression estimation developed by Rizenbergs, et al. (1977). The regression estimation yields a wet-to-dry accident ratio for a given skid number. The authors presented the relationships in graphical form (Figure 2.8), and equations for the 10-point moving averages with volume stratification were approximated. The equations are shown below.

$$\begin{aligned}
 & \text{For } AADT \leq 2700 \text{ vpd} \\
 & \text{wet / dry} = (-0.02 * SN) + 1.11 \quad \text{for } SN \leq 44.5 \\
 & \quad = (-0.01 * SN) + 0.665 \quad \text{for } 44.5 < SN \leq 47.5 \\
 & \quad = (-0.0012 * SN) + 0.2459 \quad \text{for } SN > 47.5
 \end{aligned} \tag{5.6}$$

$$\begin{aligned}
 & \text{For } AADT > 2700 \text{ vpd} \\
 & \text{wet / dry} = (-0.0151 * SN) + 0.8427 \quad \text{for } SN \leq 38.5 \\
 & \quad = (-0.0067 * SN) + 0.5167 \quad \text{for } 38.5 < SN \leq 40 \\
 & \quad = (-0.0014 * SN) + 0.3017 \quad \text{for } SN > 40
 \end{aligned} \tag{5.7}$$

where *wet/dry* = ratio of wet-to-dry-pavement accidents, and
SN = skid number at 40 mph.

After the wet-to-dry accident ratio is determined based on the skid resistance at any given time, the wet weather accident rate is calculated using a typical accident rate and the percent wet time.

$$\text{wwa rate} = \text{wet / dry ratio} * \frac{\text{typical accident rate} * (1 - p)}{\text{wet time}} \tag{5.8}$$

where: *wwa rate* = wet weather accident rate (accidents/hmvm),
wet/dry ratio = wet-to-dry accident ratio,
typical accident rate = accidents/hmvm,
p = percent of all accidents that occur on wet roads, and
wet time = percent of time roads are wet.

Once the dry accident rate and the wet weather accident rate are established for a particular iteration, each rate is multiplied by the demand during each corresponding weather condition for a specific year. The demand during wet weather is determined by multiplying the total demand during normal operation by the percent wet time. The demand during dry conditions, then, is simply the difference of the total demand during normal operation and the wet weather demand. After the dry and wet weather accident rates are multiplied by their corresponding demands, the products are added together to yield the total number of accidents expected to occur during normal operation for a particular year.

5.3.1.2 Accidents During Work Zone Conditions

A similar procedure is used for the work zone accident prediction. However, it was assumed that work zones were not in operation during wet weather. This assumption could be made because the model only considers two-lane rural roads with relatively low traffic volumes (ADT less than 10,00 vpd), where work zones are established so that one lane is closed at a time and flaggers are used for traffic control. Since long-term work zones are not considered in this model, it was assumed that work zones would not be maintained during wet weather.

A work zone accident rate was determined based on a study conducted by Pigman and Agent (1990). The authors reported that on the average, work zone accidents were approximately 1.7 times the normal accident rate. Therefore, a work zone accident rate of 1.7 times the normal accident rate was used as the default value, which may be altered by the user if desired.

Rural Primary, 2 lanes (US and WV routes):	$199 \text{ acc/hmvm} * 1.7 = 338 \text{ accidents/hmvm}$
County and Local routes over 500 ADT:	$241 \text{ acc/hmvm} * 1.7 = 410 \text{ accidents/hmvm}$

In addition to the work zone accident rate, the work zone traffic demand must be estimated for each year in the analysis period. The work zone traffic demand is a function of the time required to perform the maintenance activity, the duration of a typical work-day, and the ADT. The length of time that the work zone is in place depends on productivity estimates based on the type of maintenance activity being performed.

Default values for paving (overlay) productivity, surface treatment/preventive maintenance application productivity, and routine maintenance productivity are provided in the LCCA program. Paving productivity is defined in terms of tons per hour. The default mean value for paving productivity is 125 tons per hour, and the default COV is 10 percent. The surface treatment/preventive maintenance treatment productivity is defined in terms of square yards per hour, and the default is set at 1,500 square yards per hour with a COV of 10 percent.

The mean paving productivity and surface treatment productivity default values were assigned based on estimates provided by a West Virginia Division of Highways employee (Ford, 2000).

The work zone duration estimate for routine maintenance activities is determined in the following manner. The model presented by Al-Mansour and Sinha (1994) is used to estimate annual routine maintenance costs based on the PSI of the pavement at the time that the maintenance is performed. In order to estimate the work zone duration based on routine maintenance costs, a relationship between cost and work zone duration must be established. Therefore, for routine maintenance, the user may input a value for dollars per crew hour. The default value for this relationship was derived based on the following assumptions. First, an average maintenance crew was defined as six people, including one foreman, two flaggers, and three workers. Assuming each crew member earns \$15 per hour, the total cost of the crew is approximately \$90 per hour. It was also assumed that the cost of the equipment plus the cost of materials is approximately equal to the cost of the crew. Therefore, in order to account for the cost of the crew, equipment, and materials, the default value for routine maintenance is set equal to \$180 per crew-hour, with a COV of 10 percent.

In addition to productivity estimates, a work zone schedule must be established to estimate the traffic demand during the work zone hours. The LCCA program assumes an 8-hour work-day starting at 8:00 am and ending at 4:00 pm. However, the user has the option to change the default values for the productivity rates and/or the work-day hours if so desired.

Next, the hourly traffic demand while the work zone is in place is estimated using the traffic demand distribution for rural minor arterial roadways shown in Table 5.1, which was established by PennDOT for work zone conditions. The percentage of demand for each hour the work zone is in place is multiplied by the ADT (considering the growth rate) to yield the hourly work zone demand.

The hourly work zone demands are added together to yield the work zone traffic demand for a particular year. Then, the work zone accident rate is multiplied by the work zone traffic demand during a particular year to obtain the accident prediction for the work zone during that year. The work zone accident prediction is used in conjunction with the accident prediction for normal operating conditions to calculate accident costs for a particular year within a single iteration of the simulation.

Table 5.1 PennDOT AADT Distribution for Work Zone Conditions(Hourly Percentages) (Walls and Smith, 1998).

HOURL (24 HR CLOCK)	TRAFFIC PATTERN FOR RURAL, MINOR ARTERIAL
12 – 1	0.7
1 – 2	0.4
2 – 3	0.3
3 – 4	0.4
4 – 5	0.8
5 – 6	2.2
6 – 7	4.5
7 – 8	5.5
8 – 9	5.3
9 – 10	5.4
10 – 11	5.8
11 –12	6.0
12 – 13	6.2
13 – 14	6.4
14 – 15	7.2
15 – 16	8.1
16 – 17	8.0
17 – 18	7.1
18 – 19	5.4
19 – 20	4.4
20 – 21	3.6
21 – 22	2.8
22 – 23	2.1
23 – 24	1.4

5.3.1.3 Calculating Accident Costs

Once the accident predictions are determined for normal operation and work zone conditions for a particular year, the accident cost for that year is calculated. Accident cost is a function of the number and severity of accidents and the unit cost per accident. The severity breakdown was determined using statewide data from West Virginia Accident Data (1990-1999). First, the percentages of accidents by severity type were determined. On the average, 0.72 percent of all accidents were fatal accidents, 33.65 percent of all accidents were injury accidents, and 65.63 percent of all accidents were PDO accidents. Furthermore, there were approximately 1.10 fatalities per fatal accident and 1.54 injuries per injury accident.

The unit cost per accident is defined by a probability distribution, where mean values are established based on the accident costs provided by the Federal Highway Administration and

presented in Chapter 2. The three unit costs for an injury were averaged to yield the mean unit cost per injury shown below. A coefficient of variation (COV) of 10 percent was assumed for each distribution.

Fatality	\$2,600,000
Injury	\$78,333
PDO	\$2,000

A weighted average method was used to estimate the user costs due to accidents for a particular year. First, unit costs for each severity level are selected from the unit cost probability distributions. Then, the total number of accidents for a given year is multiplied by a weighted average cost as shown in Equation 5.9.

$$\text{accident cost} = \text{accidents} * [(0.0072 * 1.1 * \$2,600,000) + (0.3365 * 1.54 * \$78,333) + (0.6563 * \$2,000)] \quad (5.9)$$

where: *accident cost* = total accident cost for a particular year, and

accidents = total number of accidents predicted to occur during a particular year.

Finally, the accident cost determined using the equation above must be multiplied by the present worth factor to obtain a present worth cost of accidents for a particular year of the analysis period. For each iteration, the present worth costs for each year over the analysis period are added together to yield a net present value of accident costs for a particular preservation strategy. This procedure is iterated until the simulation is complete, and a probability distribution of present worth costs due to accidents is generated.

5.3.2 User Delay

User costs due to delay vary slightly between normal operation and work zone conditions. For normal operation, user delay is caused by accidents, and therefore the total delay hours are a function of the number and severity of accidents, the estimated delay time per accident, and the ADT. The user delay cost is calculated by multiplying the total delay hours by the unit cost per hour of delay. For work zone conditions, user delay is a function of delay hours

that result from the work zone or from accidents that occur within the work zone. In this case, the total delay hours are a function of the duration of the work zone, the estimated delay time per vehicle due to the work zone, the number and severity of accidents occurring within the work zone, the estimated delay time per accident, and the ADT. The user delay cost for work zone conditions is calculated by multiplying the estimated user delay hours by the unit cost per hour of delay, as defined by the user. Combining the delay costs incurred during normal operation and during work zone conditions yields the total user delay cost of a particular preservation strategy over the analysis period.

5.3.2.1 User Delay During Normal Operation

User delay during normal operation is a function of the number and severity of accidents and the ADT. However, it is difficult to estimate the delay hours caused by accidents, because each accident is unique. The time during which an accident affects traffic flow can be broken down into three categories, including accident detection, response, and clearance. Each category of time may be defined by a probability distribution. For this research, a truncated normal probability distribution is used to describe each of the three categories, eliminating the probability of a negative detection, response, or clearance time. The default values for each category are described in this section; however, the user may change the mean and/or standard deviation of any or all of the default values.

The first step in estimating user delay hours caused by accidents is to estimate the elapsed time from when the accident occurred to when it is detected and reported. The detection time was defined by a normal probability distribution with a mean of 5 minutes and a standard deviation of 1 minute. This distribution was selected based on research by Brodsky (1990).

The next step is to estimate the time required for the police, emergency medical service (EMS), and/or tow-truck to respond to the scene. The response time distribution used for this research was also based on research by Brodsky (1990), who reported that mile-a-minute ambulance speeds are typical speeds for ambulances responding to accidents in rural areas. It was assumed that the distance from the pavement section under analysis to the nearest EMS station is approximately 20 miles, with a standard deviation of 2 miles. Therefore, assuming mile-a-minute ambulance speeds (Brodsky, 1990), the probability distribution used in this research to define response time has a mean of 20 minutes and a standard deviation of 2 minutes. Again, the user may change these default values to more accurately describe the location of the pavement section being analyzed.

Finally, the vehicles and occupants, as well as responders must be cleared from the roadway before normal operating conditions can resume. However, each accident is unique and

estimating distributions for both response and clearance times is a challenge. For this research, the clearance time is dependent on the severity of the accident. For example, when a fatal accident occurs, several activities may be performed. First, the law enforcement must perform their investigation (measurements, interviewing witnesses, etc.). In addition, a coroner and a police photographer are called in, which may also take time. An accident reconstructionist may be called to the scene to take more detailed photographs and measurements than are made in the standard investigation.

The time required to clear the vehicle from the roadway is dependent on the type of vehicle involved in the accident, and the type of equipment the towing firm brings to the scene. In addition, when fatal accidents occur, it is likely that the roadway will have to be flushed with water or broomed to clear off the debris. If hazardous materials are present, the clearance time increases even more if special clean-up crews or precautions such as evacuations are required. Finally, if the roadway or appurtenances have been damaged, the Department of Transportation (DOT) is likely to be called in to make temporary repairs or erect temporary traffic control.

Depending on the individual accident, some or all of these actions may be performed, requiring an hour or more. Based on the events described above and information presented in the Incident Management Workshop by the Federal Highway Administration (FHWA, 1995) and a study by Bartlett, et al. (1988), probability distributions describing the clearance times based on accident severity were estimated. For fatal accidents, a truncated normal distribution with a mean of 60 minutes and a standard deviation of 20 minutes was used to define the clearance time. For injury accidents, the clearance time was defined by a mean of 30 minutes and a standard deviation of 10 minutes. Finally, the clearance time for PDO accidents was defined by a mean of 15 minutes and a standard deviation of two minutes.

After the total time during which the traffic flow is affected by an accident is determined, the next step in estimating user delay hours is to determine the traffic demand from the time the accident occurred to the time that normal operation resumes. Pennsylvania has established an hourly traffic demand distribution for rural minor arterial roadways, which PennDOT uses in estimating user delay costs (Walls and Smith, 1998). This traffic distribution, shown as Table 5.1, is useful in estimating demand only if the actual time of the accident is known. Since it is not possible to predict exactly when an accident will occur, the hourly demand distribution is not practical. Instead, the average hourly demand is used to estimate the number of vehicles affected by an accident. The average hourly demand for a particular year is calculated by dividing the yearly ADT in vehicles per day (considering the growth rate) by 24 hours per day. Thus, the units of the average hourly demand are vehicles per hour.

The average delay per vehicle due to an accident is estimated in a unique manner, based on the procedure for analyzing signalized intersection delay presented in the Highway Capacity Manual (1998). This procedure is a widely used and accepted method for estimating the average delay time at a signalized intersection. This procedure has also been used by highway agencies to estimate expected delay times for maintenance activities on bridge decks, where only one lane is open to traffic, and a temporary traffic signal is installed for traffic control. In this case, a two-phase signal is established, where green time is allocated to only one of the two directions at a time. In addition, all-red time is allocated between each green time in order to give the last vehicle to pass through the green light sufficient time to clear the bridge before green time is allocated to vehicles traveling in the opposite direction.

In this LCCA model, the occurrence of an accident may result in a similar situation as that described above. For example, the accident occurs on a two-lane rural road, and pavement shoulders are not included in the analysis. Therefore, it is likely that if an accident was to occur, one lane of the road would be at least partially blocked, enabling only one direction of traffic to flow at a time. It should be noted at this point that many other situations may arise when an accident occurs. A vehicle may run off of the road, resulting in little or no lane blockage. On the other hand, oncoming vehicles may collide into each other, completely blocking both lanes of travel. Since it is impossible to predict the outcome of an accident, a single-lane blockage is assumed for this research. As a result, the similarities between estimating the delay per vehicle at a signalized intersection and estimating the delay per vehicle due to an accident (under the conditions specifically defined for this model) allow the procedure presented in the Highway Capacity Manual to be simplified for use in this model.

Obviously, the pavement area occupied by an individual accident varies depending on the accident situation. However, for simplicity in the model, an assumption was made that an accident occupies a pavement area approximately 60 feet in length by one lane-width. This area accounts for the presence of the vehicle(s) involved in the accident, as well as for emergency vehicles and personnel such as the police and/or EMS. In addition, a length of 50 feet was added to each end of the accident area defined above to account for the taper, or the area where traffic traveling in the direction of the blocked lane must transition to the opposing lane, and then return to the appropriate lane after passing the accident scene (FHWA, 1993). Assuming that the traffic passes the accident scene defined above at a speed of 15 miles per hour (mph), it will take approximately 8 seconds for one vehicle to completely pass the accident scene and return to the proper lane, if necessary.

Based on this information, a green time and an all-red time were assigned to the two phases in order to perform the analysis. The allotted times for each direction of travel were equivalent, because a 50/50 directional distribution was assumed. A 20-second green time was allotted, followed by an 8-second all-red time, which allows sufficient time for vehicles traveling in one direction to clear the open lane so that the opposing direction of traffic can proceed. With two phases each consisting of 20 seconds green time plus 8 seconds all-red time, the total cycle length used for this analysis is 56 seconds.

After the cycle length is estimated, the delay per vehicle due to an accident can be estimated. First, the adjusted saturation flow rate, s_i , is calculated by multiplying the ideal saturation flow rate per lane by a series of adjustment factors. For this research, the ideal saturation flow rate was selected as 1200 vehicles per hour per lane. This rate is a typical value used for work zone conditions on two-lane roads where lanes are narrow and objects such as people, vehicles, and/or barriers are in close proximity of the traveled way, which is similar to an accident scene where one lane is blocked. The only adjustment factors applicable to this situation are the number of lanes per lane group (N) and the adjustment factor for heavy vehicles in the traffic stream (f_{HV}). Since the model only considers two-lane roads, the number of lanes per lane group (in this case, number of lanes per direction) is one. The heavy vehicle factor accounts for the additional space occupied by heavy vehicles and for the differential in the operating capabilities of heavy vehicles with respect to passenger cars (Highway Capacity Manual, 1998). Assuming each heavy vehicle is 2.0 passenger car units, the adjustment factor for heavy vehicles is:

$$f_{HV} = \frac{100}{100 + \%HV} \quad (5.10)$$

where: f_{HV} = heavy vehicle adjustment factor, and

$\%HV$ = percent heavy vehicles ($0 \leq \%HV \leq 100$).

After the adjusted saturation flow rate is determined, the flow ratio is calculated. The flow ratio, v_i , is the average hourly demand per lane divided by the adjusted saturation flow rate. Next, the lane group capacity, c_i , is calculated by multiplying the adjusted saturation flow rate, s_i , by the g/C ratio, or the ratio of green time to cycle length. The g/C ratio for this analysis was

established as 20 seconds/56 seconds, or 0.36. The v/c ratio, X_i , for a single lane group is computed by dividing the flow ratio v_i , by the lane group capacity c_i .

Finally, the average stopped delay per vehicle for a given lane group is calculated by adding together the uniform delay and the incremental delay. The uniform delay is the delay that will occur in a lane group if vehicles arrive with a uniform distribution and if saturation does not occur during any cycle. Incremental delay takes into consideration that the arrivals are not uniform but random, and that some cycles will overflow. Equations 5.11, 5.12, and 5.13 are used to calculate the average stopped delay per vehicle due to the occurrence of an accident.

$$d_i = d_{1i} + d_{2i} \quad (5.11)$$

$$d_{1i} = 0.5C \frac{(1 - g_i / C)^2}{1 - (g_i / C) [\min(X_i, 1.0)]} \quad (5.12)$$

$$d_{2i} = 900 * 0.25 \left[(X_i - 1) + \sqrt{(X_i - 1)^2 + \frac{8k_i I_i X_i}{c_i * 0.25}} \right] \quad (5.13)$$

where: d_i = the average delay per vehicle for a given lane group (sec/vehicle),

d_{1i} = uniform delay (sec/vehicle) for lane group i,

d_{2i} = incremental delay (sec/vehicle) for lane group i,

C = cycle length (sec),

g_i = effective green time for lane group i (sec),

X_i = v/c ratio for lane group i,

c_i = capacity of lane group i

k_i = incremental delay factor ($k=0.5$ for pretimed signals), and

I_i = upstream filtering metering adjustment factor ($I=1$ for isolated intersections).

The average stopped delay per vehicle is multiplied by the average demand during the time period starting when the accident occurs and ending when the accident is completely cleared from the roadway. The product is converted from seconds to hours, yielding an estimate for the average delay hours caused by an accident. The average delay hours due to the occurrence of an accident is multiplied by the number of accidents expected to occur during normal operation to determine an estimate for the total delay hours during normal operation.

5.3.2.2 User Delay During Work Zone Conditions

User delay during work zone conditions is a function of the work zone duration, the demand during work zone hours, and the number and severity of work zone-related accidents. As described previously, the work zones for this LCCA are established so that a one-half mile section of one lane is closed at a time and flaggers are used for traffic control. The procedure for estimating user delay during work zone conditions is very similar to the procedure for estimating user delay during normal operation that was described in the previous section.

The first step in estimating user delay during work zone conditions is to determine the traffic demand while a work zone is in place. The procedure for estimating the hourly work zone traffic demand for a particular year of the analysis period was described in Section 5.3.1.2. The average delay per vehicle is then calculated for a particular hour, based on the hourly work zone traffic demand.

The procedure described in the previous section is also used here to calculate the delay per vehicle. However, the cycle length was adjusted to account for the distance occupied by the work zone. The actual work zone area is established as one-half mile in length by one lane-width. In addition, a taper of 100 feet is added to both ends of the work zone, which is the recommended taper length for two-lane roads (FHWA, 1993). If the work zone speed limit is 25 mph, it will take approximately 80 seconds for a vehicle to traverse the work zone and resume normal two-lane conditions. With this in mind, a 60-second green time was allotted, followed by an 80-second all-red time. Therefore, the cycle length used for estimating user delay during work zone conditions is 280 seconds, and the g/C ratio is $60/280$, or 0.32.

The series of formulas used to estimate the delay per vehicle for normal operating conditions are identical to the formulas used to estimate the delay per vehicle for work zone conditions. First, the adjusted saturation flow s_i is calculated where again the ideal saturation flow rate used was 1200 vehicles per hour per lane. Next, the flow ratio v_i , lane group capacity c_i , and v/c ratio X_i are calculated. Finally, the average delay per vehicle for a particular hourly demand is calculated using Equations 5.11, 5.12, and 5.13. The delay per vehicle is calculated for each hour, and the average delay per vehicle for a particular work zone is calculated by dividing the sum of all hourly delay estimates by the work zone duration. The average delay per vehicle is multiplied by the work zone demand to obtain the total delay hours for work zone conditions.

Finally, user delay caused by accidents in the work zone must be addressed. The procedure used to predict the number of work zone accidents was described in Section 5.3.1.2. Since the time during which the work zone is in place is very limited and the ADT is relatively small, the probability of an accident occurring during a particular work zone hour is infinitesimal.

In fact, the predicted number of work zone accidents occurring during a particular year is also extremely small, making it very difficult to assign the accident(s) to a particular hour in order to estimate the delay caused by the accident. Therefore, the delay hours caused by the work zone accident(s) and the user costs associated with the delay hours are not included in this analysis.

5.3.2.3 Calculating User Delay Costs

For each iteration of the simulation, values for detection, response, and clearance times are selected from the probability distributions established by the user. Delay hours for normal operation and work zone conditions are estimated for each year and then added together to obtain the total delay hours. The procedure used to calculate user delay costs in this LCCA model follows the procedure presented in Chapter 2 (Section 2.5.2). The current year unit cost of time estimates used in this analysis are divided into two categories including passenger cars and trucks. Each unit cost is defined by a probability distribution. The default values are the following: for passenger cars, the mean cost of time is \$12.88 per vehicle-hour with a coefficient of variation (COV) equal to 10 percent, and for trucks, the mean cost of time is \$21.47 per vehicle-hour with a COV equal to 10 percent. The mean values were obtained by converting the cost of time estimates presented in NCHRP Report 133 (Curry and Anderson, 1972) to current year dollars, as described in Chapter 2. The user may change the default values (the means or COVs) for the cost of time estimates if desired.

The total delay hours calculated for a particular year for a preservation strategy are separated into passenger car delay hours and truck delay hours, using the percent trucks input by the user. The total passenger car delay hours are multiplied by the unit cost of time for passenger cars to yield a total cost due to delay for passenger cars during a particular year. A similar multiplication is performed with the truck delay hours and the unit cost of time for trucks to obtain a total delay cost for trucks during the same year.

The delay costs for passenger cars and trucks are added together to estimate the total cost due to delay for a particular year during the analysis period. This total user delay cost is then multiplied by the present worth factor in order to obtain a present worth cost for user delay. The entire user delay procedure is performed each year of the analysis period for each preservation strategy, and then the entire process is iterated until the simulation is complete. Upon completion of the simulation, the present worth costs obtained from each iteration are combined to form a probability distribution describing the net present value of user delay costs for a particular preservation strategy.

5.3.3 Excess Vehicle Operating Costs

Finally, excess vehicle operating costs (VOC) are calculated. Vehicle operating costs include fuel, oil, tires, maintenance and repair, and depreciation. Under normal operating conditions, it is disputed whether or not there are significant differences in VOC for various pavement surfaces and conditions. However, speed changes or traffic delay cycles attributed to pavement design or conditions can produce significantly different user costs. For example, accidents and work zone conditions may require additional speed changes, stops, and hours of idling that the user does not usually experience during normal operation. Therefore, for this research, only the VOC due to delay caused by accidents and work zones is considered.

As discussed in Section 5.3.2, delay may result from work zone conditions or from accidents that occur during normal operation. The VOC components considered in this analysis include idling costs and speed change costs. The current year unit costs for idling and speed changes are defined by probability distributions. The default distributions are shown in Table 5.2. The mean values were obtained by converting the VOC estimates presented in NCHRP Report 133 (Curry and Anderson, 1972) to current year dollars, as described in Chapter 2 (Section 2.5.3). The user may change the default values for the current year VOC unit cost distributions if desired.

Table 5.2 Current Year VOC Unit Cost Distributions.

VOC COMPONENT	PASSENGER CARS		TRUCKS	
	MEAN	COV (%)	MEAN	COV (%)
Idling (\$/vehicle-hour)	\$0.70	10	\$0.83	10
Speed Change 55-0-55 (\$/1000 stops)	\$84.41	10	\$729.92	10

The excess VOC due to idling is a function of the total delay hours (delay during normal operation and work zone conditions) and the current year VOC unit cost distributions for idling. The total delay hours estimated for one year for a particular preservation strategy are separated into passenger car delay hours and truck delay hours, using the value for percent trucks input by the user. For passenger cars, the excess VOC due to idling is calculated by multiplying the total passenger car delay hours for one year by a value selected from the probability distribution

describing the idling unit cost for passenger cars. For trucks, the excess VOC due to idling is calculated by multiplying the total truck delay hours for that year times a value selected from the probability distribution describing the idling unit cost for trucks. The products are added together to obtain an estimate for the total excess VOC due to idling during a particular year.

Similarly, the excess VOC due to speed changes is a function of the demand during accidents and work zone conditions and the current year VOC unit cost distributions for speed changes. An assumption was made that since the analysis is performed for two-lane roads, all vehicles that are delayed come to a stop. Therefore, the number of stopped vehicles is equal to the demand during accidents plus the demand during work zone conditions, both of which were calculated previously. The number of stopped vehicles is separated into passenger cars and trucks, based on the value for percent trucks input by the user. The excess VOC due to speed changes is the sum of the products of stopped vehicles times VOC unit costs for both passenger cars and trucks.

The excess VOC for idling and speed changes for passenger cars and trucks are added together to yield the total excess VOC for a single year of a particular preservation strategy. The total excess VOC is then multiplied by the present worth factor in order to obtain the present worth excess VOC for a particular preservation strategy. This procedure is performed each year of the analysis period for each preservation strategy, and then the entire process is iterated until the simulation is complete. Upon completion of the simulation, the present worth costs obtained from each iteration are combined to form a probability distribution describing the net present value of excess vehicle operating costs for a particular preservation strategy.

5.4 SUMMARY OF LIFE-CYCLE COST MODELS

The LCCA developed for this research considers both agency and user costs associated with the rehabilitation and preventive maintenance strategies. The costs are discounted to a single net present value, allowing for the comparison of the two alternatives. Agency costs include costs associated with the initial cost of construction, routine maintenance, rehabilitation, and preventive maintenance. The salvage value for each alternative is considered as a negative agency cost. User costs considered in this analysis include costs due to accidents, user delay, and excess vehicle operating costs.

The incorporation of risk analysis introduces variability into the cost calculations. Unit costs for construction materials, accidents, delay, and vehicle operation are described by probability distributions input by the user. At the beginning of each iteration, a unit cost for each

parameter is selected from its corresponding distribution, and the agency and user costs are calculated by the procedures described in this chapter.

After the simulation is complete, the agency and user present worth costs obtained from each iteration are combined to form probability distributions that describe the variability associated with various cost components for each preservation strategy. This unique analysis method allows the user to consider the variability of various cost components and thereby assess the risk associated with each alternative. The incorporation of variability into this LCCA model exposes uncertainty that may be hidden in a deterministic model, which greatly enhances the decision-making process.

CHAPTER 6

LCCA MODEL EXECUTION

The coding for the life-cycle cost analysis (LCCA) developed in this research was written using Visual Basic for Applications (VBA) in Microsoft® Excel 97. The LCCA model is executed as a macro within Excel. The first section of this chapter presents a discussion on the procedure for running the LCCA program. The following two sections contain detailed descriptions of the input screens and the output screens, respectively. A discussion on the interpretation of the LCCA results is presented, followed by a case study.

6.1 RUNNING THE LCCA MODEL

In order to run the LCCA model, the user must have access to Microsoft® Excel 97 or Excel 2000. Opening the LCCA program automatically affixes a toolbar with a button labeled “Run LCCA” to the toolbar area at the top of the Excel window. LCCA program execution is initiated when the user clicks on the “Run LCCA” button.

The length of time required to run the LCCA program depends on the number of iterations that the program runs. For this research, the number of iterations that the model is programmed to run was set equal to the minimum number of iterations required in order for the output distributions to reach convergence. Convergence is reached when the output distributions change by less than a specified percent. Based on a risk analysis user’s manual (@Risk, 1997), the recommended level of convergence is 1.5 percent. In addition, the manual recommends that the level of convergence should not be set below 0.75 percent. The level of convergence established for this research was 1 percent.

In order to determine the number of iterations required to reach a convergence level of 1 percent, the LCCA model was first initialized to run 1,000 iterations. The program was executed approximately 75 times, and the present worth cost distributions for the total cost, agency cost, and user cost for both preservation strategies were combined with the cost distributions developed from the previous runs. The change in percent of the output of the current run versus the cumulative output to that point was calculated and the number of iterations required to reach a level of convergence of 1 percent was determined. Plots of the number of iterations versus the percent change for the total present worth cost and agency cost associated with each preservation strategy are shown in Figure 6.1. User costs were also considered, however the plots for user

costs were similar to the plots for the total present worth cost, since user costs significantly influence the total present worth cost.

Based on Figure 6.1, a convergence level of 1 percent is reached at 3,000 iterations. Therefore, for each analysis, the LCCA model is programmed to run 3,000 iterations, resulting in a computer run-time of approximately three minutes.

6.2 USER INPUT SCREENS

When the user clicks on the “Run LCCA” button in the toolbar area, the program begins to run and a series of input screens appear on the computer monitor. The user input screens are listed in Tables 6.1 and 6.2, along with the required and advanced user input parameters associated with each input screen. The remainder of this section describes the interaction between the user and the LCCA program, and then identifies the required user inputs and the advanced user inputs.

All of the input screens follow a general format. Each input value is entered into the LCCA program through the use of an input box, a pull-down box, an option box, or a check box. Input boxes are boxes that the user types in a value. Input boxes are often used for inputting mean values, such as those shown in Figure 6.2. Pull-down boxes, also shown in Figure 6.2, are often used for inputting coefficients of variation (COV). The user clicks on the down arrow and selects any value contained within the list, or the user may click in the box and enter a value. Option boxes, such as the one for selecting a structural overlay thickness illustrated in Figure 6.3, are used when there are limited options from which the user may select. The user simply clicks on the circle next to the option he or she wishes to select. Finally, check boxes are used to allow the user to select one or more options at a time. Check boxes are shown in Figure 6.10. When the user clicks in a check box, a check mark appears and that option is selected. The user may deselect an option by clicking in a box that has a check mark showing.

Each input variable is initialized to some value. Allowable ranges are provided for the input values. If the user enters text or a value that is out of the allowable range for any of the inputs, the program informs the user that the input is invalid, and asks the user to enter a value within the specified range. In order to advance to the next input screen in the series, the user must click the OK button in the bottom right-hand corner of the screen. When all input values are entered successfully, the program prompts the user to select one of two options: either run the LCCA or return to the input screens to view or edit the input values. The user initiates the analysis procedure by selecting the run LCCA option and clicking the OK button.

Figure 6.1 Plots Used for Determining the Required Number of Iterations for the LCCA.

Table 6.1 Required User Input Screens.

INPUT SCREEN	PARAMETERS	PARAMETER DEFINED BY:	
		Probability Distribution	Discrete Value
Traffic	Average Daily Traffic (ADT)	X	
	Percent Trucks	X	
	Truck Factor	X	
	Growth Rate	X	
Pavement Design	Analysis Period		X
	Initial PSI	X	
	Terminal PSI		X
	Effective Roadbed Soil Resilient Modulus	X	
	Structural Overlay Thickness		X
Material Properties	Surface Layer Coefficient	X	
	Base Layer Coefficient	X	
	Base Drainage Coefficient	X	
	Subbase Layer Coefficient	X	
	Subbase Drainage Coefficient	X	
Aggregate Type	Surface Aggregate Type		X
Design Constraints	Probability of Failure for Initial Pavement Design		X
	Probability of Failure for Structural Overlay Design		X
Strategy Derivation Constraints	Skid Resistance Threshold		X
	Serviceability Threshold for Rehabilitation Strategy		X
Route Classification	Route Classification		X
Discount Rate	Discount Rate (percentage)		X

Table 6.2 Advanced User Input Screens.

INPUT SCREEN	PARAMETERS	PARAMETER DEFINED BY:	
		Probability Distribution	Discrete Value
Unit Cost of Materials	Surface Layer Unit Cost	X	
	Base Layer Unit Cost	X	
	Subbase Layer Unit Cost	X	
	Surface Treatment Unit Cost	X	
Productivity Estimates	Routine Maintenance Productivity	X	
	Preventive Maintenance Productivity	X	
	Paving Productivity	X	
Work Zone Schedule	Work Zone Start Time		X
	Maximum Work Zone Hours Per Day		X
Skid Resistance Model	Slope		X
	Intercept		X
Percent Wet Time	Percent Wet Time		X
Accident Rates	Typical Accident Rate		X
	Work Zone Accident Rate		X
Accident Costs	Unit Cost of Fatality	X	
	Unit Cost of Injury	X	
	Unit Cost of PDO	X	
Delay Due to Accidents	Accident Detection Time	X	
	Response Time	X	
	Clearance Time for Fatal Accident	X	
	Clearance Time for Injury Accident	X	
	Clearance Time for PDO Accident	X	
Cost of Time	Cost of Time: Passenger Cars	X	
	Cost of Time: Trucks	X	
Vehicle Operating Costs	Speed Change Cost: Passenger Cars	X	
	Speed Change Cost: Trucks	X	
	Idling Cost: Passenger Cars	X	
	Idling Cost: Trucks	X	

6.2.1 Required User Inputs

The first input screen that appears is the Traffic Parameters input screen, as shown in Figure 6.2. The user must input a mean and coefficient of variation (COV) for the average daily traffic (ADT), percent trucks, truck factor, and growth rate. For the mean values, the input screen identifies the range of values accepted for each parameter. For the COVs, pull-down boxes are provided so that the user may select an appropriate value. After all traffic inputs are successfully entered and the user clicks the OK button in the bottom right corner of the screen, the Design Parameters input screen appears.

Input Traffic Parameters

	Mean	Coefficient of Variation (%)
ADT (Average Daily Traffic) (ranges from 1,000 - 10,000 vpd)	5000	33
Percent Trucks (Enter Percent) (ranges from 1% - 20%)	10	33
Truck Factor (ranges from .12 - .32)	0.38	33
Growth Rate (Enter Percent) (ranges from 1% - 5%)	3	33

Click OK When Complete

OK

Figure 6.2 Traffic Parameters Input Screen.

Input Design Parameters

	Mean	Coefficient of Variation (%)
Analysis Period (ranges from 25 to 40 years)	30	
Initial PSI (at time of construction) (ranges from 4.0 to 4.5)	4.2	6.7
Terminal PSI (ranges from 1.5 to 2.5)	2.8	
Effective Roadbed Soil Resilient Modulus (ranges from 1,000 to 40,000 psi)	9000	10
Select Desired Thickness of Structural Overlay	☐ Structural Overlay Thickness ☐ 1.5-Inch Overlay ☒ 2.0-Inch Overlay ☐ 2.5-Inch Overlay	

Click OK When Complete

OK

Figure 6.3 Design Parameters Input Screen.

The Design Parameters input screen is shown in Figure 6.3. The user must input the analysis period, a mean and COV for the initial PSI, a terminal PSI, and a mean and COV for the effective roadbed soil resilient modulus. In addition, the user must select an average overlay thickness for the structural overlay. The default value for the structural overlay thickness is 2.0 inches, although the user may instead select a thickness of either 1.5 or 2.5 inches. After the design parameters are input, the user clicks the OK button and the Material Properties input screen appears.

The Material Properties input screen, along with the default values for each parameter, is shown in Figure 6.4. The user is required to input a mean and COV for the surface layer coefficient, base layer coefficient, base drainage coefficient, subbase layer coefficient, and subbase drainage coefficient, which are essential for the pavement design as well as for the structural overlay design.

The Surface Aggregate Type input screen, shown in Figure 6.5, is next in the series of input screens. The user is required to select either the skid resistant aggregate (SRA) option or the unrestricted option. For this research, unrestricted aggregate implies the use of limestone as the surface aggregate type. This input is used to predict the skid resistance of the pavement over time, and is also used to estimate wet weather accidents. After the user selects the surface aggregate type and clicks on the OK button, the Design Constraints input screen appears.

The Design Constraints input screen is shown in Figure 6.6. First, the user must select a value for the probability of failure for the initial pavement design. This value may range from 1 percent to 50 percent. The default value for the probability of failure is set at 50 percent, corresponding to the reliability level recommended by AASHTO (1993) for low volume roads. In addition to the initial pavement design, the structural overlay design also requires that the user input a probability of failure between 1 percent and 50 percent. The default value is 50 percent.

Next is the Strategy Derivation Constraints input screen, shown in Figure 6.7. This screen allows the user to input a skid resistance threshold, which is the skid number below which a maintenance action is triggered for either the preventive maintenance strategy or the rehabilitation strategy. The allowable range for the skid resistance threshold is $SN_{40} = 0$ to $SN_{40} = 38$, and the default value is set at $SN_{40} = 32$. The PSI threshold for the rehabilitation strategy is also included in this input screen. The PSI threshold is the PSI level below which a surface treatment is required for the rehabilitation strategy. The PSI threshold may range from 0 to 3.0, and the default value is set equal to 2.0.

The 'Input Material Properties' dialog box contains a table for inputting material coefficients. The table has two columns: 'Mean' and 'Coefficient of Variation (%)'. There are five rows of input fields, each with a label and a range in parentheses.

	Mean	Coefficient of Variation (%)
Surface Layer Coefficient (ranges from 0.25 to 0.50)	0.44	10
Base Layer Coefficient (ranges from 0.04 to 0.20)	0.14	10
Base Drainage Coefficient (ranges from 0.5 to 1.5)	1.0	10
Subbase Layer Coefficient (ranges from 0.04 to 0.20)	0.11	10
Subbase Drainage Coefficient (ranges from 0.5 to 1.5)	1.0	10

At the bottom right, there is a checkbox labeled 'Click OK When Complete' and an 'OK' button.

Figure 6.4 Material Properties Input Screen.

The 'GRA versus Unrestricted Aggregate Type' dialog box is used to select the surface aggregate type. It features a label 'Select the Surface Aggregate Type:' followed by two radio button options: 'Glad Resistant Aggregate' (which is selected) and 'Unrestricted'. An 'OK' button is located at the bottom right.

Figure 6.5 Aggregate Type Input Screen.

Select Probabilities of Failure

Select the probability of failure for the initial pavement design. (ranges from 1% to 50%)

90

Select the probability of failure for the structural overlay design. (ranges from 1% to 50%)

90

Click OK When Complete

OK

Figure 6.6 Design Constraints Input Screen.

Establish Constraints

Skid Resistance Threshold

The Skid Resistance Threshold is the skid number below which a maintenance action is triggered for either the preventive maintenance strategy or the rehabilitation strategy.

The Skid Resistance Threshold ranges from SM40 = 0 to SM40 = 36.

Select a Skid Resistance Threshold:

32

PSI Threshold

The PSI Threshold is the PSI level below which a surface treatment is required for the rehabilitation strategy.

The PSI Threshold ranges from 0.0 to 3.0.

Select a PSI Threshold:

2.0

Click OK When Complete

OK

Figure 6.7 Strategy Derivation Constraints Input Screen.

The Route Classification input screen, shown in Figure 6.8, is next. This screen requires the user to select a road classification of a U.S. route, a state route, or a county route. This input is the basis for establishing appropriate accident rates used for predicting accident experience. West Virginia Accident Data (1998) documented that accident rates are significantly higher on county routes than U.S. and state routes (the three-year accident rates were presented in Table 2.17). After the user selects the route classification and clicks the OK button, the Discount Rate input screen appears.

The Discount Rate input screen is shown in Figure 6.9. The discount rate is used for discounting the costs incurred throughout the analysis period to present worth costs. A discount rate of 5 percent is set as the default. The allowable range is 1 percent to 10 percent.

Next is the Advanced User form, shown in Figure 6.10. The topics listed on the Advanced User form consist of the following; unit costs of materials and surface treatments, productivity estimates, the work zone schedule, the skid resistance prediction model, the percent wet time, typical accident rates, accident costs, accident detection, response, and clearance times, the cost of time (user delay), and excess vehicle operating costs (VOC). Default values are provided for each of the inputs. In order to view or edit the values assigned to a particular variable, the user must click on that topic and a check mark will appear in the box. The user may select any combination of topics, ranging from none to all of the topics. If no topics are selected, the LCCA program assumes default values for each of the inputs. After the user has selected the topics he or she wishes to view, the user clicks the OK button. If any boxes on the Advanced User form are checked, the screens corresponding to the selected topics appear one by one. The advanced user topics are described in detail in the following section. If no boxes on the Advanced User form are selected, or when all selected topics have been viewed, the Run LCCA or Edit Input Values form appears.

The Run LCCA or Edit Input Values form is shown in Figure 6.11. This form allows the user to select one of two options. If the “Run LCCA” option is selected, the LCCA simulation begins to run. If the “Edit Input Values” option is selected, the program returns to the series of input screens and the user may view or edit the input values he or she selected.

A dialog box titled "Select Route Classification" with a blue header bar. The main area is light gray. In the center, the text "Select the Route Classification:" is followed by two radio button options. The first option, "U.S. or State Route", is selected with a checked radio button. The second option, "County Route", is unselected with an unchecked radio button. In the bottom right corner, there is an "OK" button.

Figure 6.8 Route Classification Input Screen.

A dialog box titled "Discount Rate for Present Worth Cost Calculations" with a blue header bar. The main area is light gray. In the center, the text "Discount Rate (percent)" is followed by a text input field containing the value "5". Below this text, it says "default = 5%". In the bottom right corner, there is an "OK" button. At the bottom center, there is a label "Click OK When Complete".

Figure 6.9 Discount Rate Input Screen.

Advanced User Form

Select any topics you wish to view and/or edit default values:

☐ Unit Costs of Materials and Surface Treatments	☐ Typical Accident Rates
☐ Productivity Estimates	☐ Accident Costs
☐ Work Zone Schedule	☐ Accident Detection, Response, and Clearance
☐ Skid Resistance Prediction Model	☐ Cost of Time (User Delay)
☐ Percent Wet Time	☐ Excess Vehicle Operating Costs

Click OK When Complete

OK

Figure 6.10 Advanced User Form.

Execute LCCA Program or Edit Input Values?

Select appropriate option:

☒ Run LCCA Program

☐ Edit Input Values

Click OK To Continue

OK

Figure 6.11 Run LCCA or Edit Input Values Form.

6.2.2 Advanced User Inputs

As mentioned in the previous section, the user may select to view and/or edit any or all of the advanced user topics. If no topics are selected, none of the following screens will appear. If one or more of the advanced user topics are selected, one screen per selected topic will appear. The advanced user input screens are designed using the same format as that described for the required input screens. The advanced user screens allow the user to alter default parameters. Each figure showing an input screen in this section displays the programmed default values. The remainder of this section describes the advanced user input screens.

The first advanced user topic includes the unit costs of materials and surface treatments. The input screen and the default values are shown in Figure 6.12. The user may view or edit the mean and/or coefficient of variation (COV) for the unit cost of the surface layer, the base layer, and the subbase layer. In addition, the unit cost of a surface treatment or preventive maintenance treatment is also included on this screen. These values are used to calculate agency costs for both preservation strategies.

The next advanced user topic involves productivity estimates, shown in Figure 6.13. Routine maintenance, preventive maintenance, and paving productivity estimates are described by distributions defined by a mean and COV. Productivity estimates are required so that the work zone duration can be calculated for use in estimating accidents, user delay, and excess vehicle operating costs due to work zones. For routine maintenance, the user may input a value for dollars per crew-hour. The default value for this input is \$180 per crew-hour with a COV of 10 percent. For preventive maintenance productivity, the default value is set at 1500 square yards per hour with a COV of 10 percent. Finally, the mean default value for paving productivity is 125 tons per hour, and the default COV is 10 percent. The justification for these default values was presented in Chapter 5.

Unit Costs of Materials and Surface Treatments

	Mean	Coefficient of Variation (%)
Surface Layer (HMA) Unit Cost (in \$/cy)	\$100	10
Base Layer Unit Cost (in \$/cy)	\$42	10
Subbase Layer Unit Cost (in \$/cy)	\$36	10
Unit Cost of Surface Treatments (in \$/base mile)	\$10,000	10

Click OK When Complete

Figure 6.12 Unit Cost of Materials Input Screen.

Productivity Estimates

	Mean	Coefficient of Variation (%)
Routine Maintenance Productivity (in \$/crew-hour, where crew-hours are based on 75% of time of maintenance activity) default = \$150/crew-hour	100	10
Preventive Maintenance Productivity (in sq yd / hour) default = 1500 sq yd / hour	1500	10
Paving Productivity (in tons / hour) default = 125 tons / hour	125	10

Click OK When Complete

Figure 6.13 Productivity Estimates Input Screen.

The Work Zone Schedule input screen is shown in Figure 6.14. Since traffic demand fluctuates throughout the day, it is important to consider the time during which a work zone is in place when estimating work zone demand. The work zone demand is used to predict accidents, user delay, and vehicle operating costs associated with the work zone. The user may input a start time for the work zone, ranging from 0 (midnight) to 23 (11:00 PM). In addition, the user may input the maximum work zone hours per day, ranging from 4 hours to 16 hours. The default values are initialized to a work zone start time of 8 (8:00 AM) and a maximum of 8 work zone hours per day.

The next advanced user topic is the skid resistance model. The Skid Resistance Model input screen is shown in Figure 6.15. As described in Chapter 4, a linear relationship is assumed for this research when describing the skid number versus cumulative traffic. The default values for the slope and intercept of the skid resistance model vary depending on the surface aggregate type selected by the user. For skid resistant aggregate (SRA), the slope is set equal to zero and the intercept is set equal to 42.5 (Dierstein, 1977). For unrestricted aggregate, the slope is set equal to -0.3 , and the intercept is set equal to 40.5 (Quinn, 1975).

The Percent Wet Time input screen is shown in Figure 6.16. This screen allows the user to input the percentage of time that the pavement section under analysis is subjected to wet conditions. The percent wet time is a factor used in predicting wet weather accidents. The default value for the percent wet time is 15 percent, which is a typical percentage of time that pavements are wet in West Virginia.

The Accident Rate input screen is shown in Figure 6.17. The user may input a typical accident rate as well as a work zone accident rate. The accident rates are defined in terms of accidents per one hundred million vehicle miles (hmv). Accident rates are used to predict accident experience during normal operation and work zone conditions. The default value for the typical accident rate is dependent on the route classification input by the user. For U.S. and state routes, the default value is set at 199 accidents per hmv, and for county routes the default value is set at 241 accidents per hmv. These values are based on data presented by the West Virginia Division of Highways (West Virginia Accident Data, 1998). The default value for the work zone accident rate is set at 1.7 times the typical accident rate, based on research presented by Pigman and Agent (1990).

Work Zone Schedule

Work Zone Start Time
(0=midnight, 23=11pm)

Maximum Work Zone Hours Per Day
(ranges from 1 to 35)

Click OK When Complete

OK

Figure 6.14 Work Zone Schedule Input Screen.

Skid Resistance Model

Slope Intercept

Skid Resistance Model
(A linear relationship is assumed
for SR versus Cumulative Traffic)

Click OK When Complete

OK

Figure 6.15 Skid Resistance Model Input Screen.

Percent Wet Time

Enter Percent Wet Time
(ranged from 1% to 30%)

1%

Click OK When Complete

OK

Figure 6.16 Percent Wet Time Input Screen.

Typical Accident Rates

Typical Accident Rate (accidents / hr/mi)
(U.S. and State Routes = 199 acc/hr/mi
County Routes = 241 acc/hr/mi)

199

Work Zone Accident Rate (accidents / hr/mi)
(default = 1.7 times the typical accident rate)

338

Click OK When Complete

OK

Figure 6.17 Accident Rate Input Screen.

The Accident Costs input screen is shown in Figure 6.18. The user may input means and coefficients of variation (COV) for the unit costs of a fatality, an injury, and a property damage only (PDO) accident. The mean default values for each severity level are \$2,600,000 per fatality, \$78,333 per injury, and \$2,000 per PDO accident. The mean default values are based on crash cost estimates presented by the Federal Highway Administration. The default COV for each severity level is 10 percent.

The Delay Due to Accidents input screen is shown in Figure 6.19. The user may input the mean and standard deviation describing any or all of the following time intervals. First, the average time required to detect the accident is defined in terms of minutes. The default value for the mean detection time is 5 minutes and the standard deviation is 1 minute. Next, the response time is defined in terms of minutes, and is considered to be a function of the distance to the nearest EMS station and the speed of the response vehicle. The default value is set at 20 minutes, assuming the distance to the nearest EMS station is 20 miles and the response vehicle speed is 60 miles per hour. The default standard deviation for response time is 10 minutes. The next three time intervals describe the average clearance times associated with fatal accidents, injury accidents, and PDO accidents, respectively. The default value for the mean clearance time for a fatal accident is 60 minutes with a standard deviation of 20 minutes. The default value for the mean clearance time for an injury accident is 30 minutes with a standard deviation of 10 minutes. Finally, the default value for the mean clearance time for a PDO accident is 15 minutes with a standard deviation of 2 minutes. A detailed discussion of the establishment of these default values was presented in Section 5.3.2.1.

The next advanced user topic addresses the cost of time associated with delays caused by accidents and/or work zones. The Cost of Time input screen is shown in Figure 6.20. The unit cost of time is separated into costs for passenger cars and trucks. The mean default value for the cost of time for passenger cars is \$12.88 per vehicle-hour and the default COV is 10 percent. The mean default value for the cost of time for trucks is \$21.47 and the default COV is 10 percent. The mean default values were taken from NCHRP Report 133 (Curry and Anderson, 1972), and escalated to present worth costs using the method presented previously in Section 2.5.2.

Accident Costs Based on Severity

	Mean	Coefficient of Variation (%)
Cost of Fatality default = \$2,600,000	<input type="text" value="\$2,600,000"/>	<input type="text" value="10"/>
Cost of Injury default = \$75,000	<input type="text" value="\$75,000"/>	<input type="text" value="10"/>
Cost of PDO Accident default = \$2,000	<input type="text" value="\$2,000"/>	<input type="text" value="10"/>

Note: Default values based on FHWA Crash Cost Estimates

Click OK When Complete

Figure 6.18 Accident Costs Input Screen.

Delay Due To Accidents

	Mean	Standard Deviation
Time to Detect (in minutes) default: mean = 5 min, s = 1 min	<input type="text" value="5"/>	<input type="text" value="1"/>
Response Time (in minutes) default: mean = 20 min, s = 2 min	<input type="text" value="20"/>	<input type="text" value="2"/>
Clearance Time for Fatal Accident (in minutes) default: mean = 60 min, s = 20 min	<input type="text" value="60"/>	<input type="text" value="20"/>
Clearance Time for Injury Accident (in minutes) default: mean = 30 min, s = 10 min	<input type="text" value="30"/>	<input type="text" value="10"/>
Clearance Time for PDO Accident (in minutes) default: mean = 15 min, s = 2 min	<input type="text" value="15"/>	<input type="text" value="2"/>

Click OK When Complete

Figure 6.19 Delay Due to Accidents Input Screen.

	Mean	Coefficient of Variation (%)
Cost of Time: Passenger Cars default = \$12.00/veh-hr	\$12.00	10
Cost of Time: Trucks default = \$21.67/veh-hr	\$21.67	10

Click OK When Complete

OK

Figure 6.20 Cost of Time Input Screen.

	Passenger Car		Truck	
	Mean	COV (%)	Mean	COV (%)
Speed Change Cost: (\$/1,000 stops) default values for 95-0-95: Passenger Car = \$94.41/1,000 stops Truck = \$729.92/1,000 stops	\$94.41	10	\$729.92	10
Idling Cost (\$/veh-hr) default values for idling: Passenger Car = \$0.70/veh-hr Truck = \$0.83/veh-hr	\$0.70	10	\$0.83	10

Click OK When Complete

OK

Excess VOC default values were established by escalating costs presented in NCHRP Report 135.

Figure 6.21 Vehicle Operating Costs Input Screen.

The final advanced user screen is the Vehicle Operating Costs input screen, presented in Figure 6.21. For this research, there are two types of excess vehicle operating costs (VOC) incurred by highway users. The first type of VOC includes costs associated with speed changes, where vehicles are required to come to a stop at the accident scene or work zone and then accelerate up to the initial speed. Speed change costs are separated into costs for passenger cars and costs for trucks. The excess VOC due to speed changes are defined in terms of dollars per 1,000 stops, assuming initial traveling speeds of 55 mph. For passenger cars, the default speed change cost is \$84.41/1,000 stops with a COV of 10 percent. For trucks, the default speed change cost is \$729.92/1,000 stops with a COV of 10 percent. The second type of VOC includes idling costs incurred by users who are stopped due to an accident or a work zone. Similar to speed change costs, idling costs are separated into costs for passenger cars and costs for trucks. The excess VOC due to idling are defined in terms of dollars per vehicle-hour. The default idling cost for passenger cars is \$0.70/vehicle-hour with a COV of 10 percent. The default idling cost for trucks is \$0.83/vehicle-hour with a COV of 10 percent. The mean default values were obtained by escalating unit costs taken from NCHRP Report 133 (Curry and Anderson, 1972) to present worth costs using the method presented in Section 2.5.3.

6.2.3 Performing Additional Analyses

When the “Run LCCA Program” option button is selected, the program performs the economic evaluation and displays the output on an Excel spreadsheet. Upon completion of the analysis, the program provides the user with one final input screen, shown in Figure 6.22. The user is required to select one of two options regarding input values for the next analysis. The default option is set to initialize all input values to their original default values. However, if the user would like use the current input values for the next analysis, or would like to edit the current input values for the next analysis, the user should click on the second option labeled “Edit Current Values.”

In order to run another analysis, the user must click on the “Run LCCA” button in the toolbars area of the Excel spreadsheet. Finally, if the user closes the LCCA program, all input values will be reset to the original default values when the program is opened and executed again.

Figure 6.22 Next Analysis Option Screen.

6.3 MODEL OUTPUT SCREENS

After all input values are successfully entered and the user clicks the “Run LCCA Program” option on the Run LCCA or Edit Input Values Form (Figure 6.11), the economic analysis is executed. The output is presented in two manners. First, a two-page Summary Sheet appears when the analysis is complete. The two-page Summary Sheet is shown in Figure 6.23.

The first page of the Summary Sheet provides initial pavement design data as well as maintenance data. The initial pavement design data includes the layer thicknesses (in inches), the structural number, the probability of failure, the analysis period selected by the user, and the surface aggregate type selected by the user. The layer thicknesses, structural number, and probability of failure are mean values obtained from the risk analysis simulation. The user must use discretion when selecting appropriate layer thicknesses for the actual construction. For example, if rounding results in a surface layer thickness of 3.4 inches, the user must select an

appropriate thickness for the actual construction. In this case, a surface layer thickness of 3.5 inches would most likely be constructed.

The maintenance data provided on the first page of the Summary Sheet includes the PSI threshold and skid resistance threshold for both preservation strategies. In addition, the average required structural overlay thickness (in inches) is presented, as well as the year that the structural overlay is performed. Finally, the average number of surface treatments required for the rehabilitation strategy are provided, as well as the average number of preventive maintenance treatments required over the analysis period for the preventive maintenance strategy. Again, the values may or may not be integer values, since they represent the mean values obtained from the risk analysis simulation.

The second page of the Summary Sheet provides cost data for both preservation strategies. The mean and coefficient of variation (COV) are provided for the total present worth cost, the agency present worth cost, and the user present worth cost for both the rehabilitation and preventive maintenance strategies. Mean present worth costs are provided for the initial cost of construction, the cost of routine maintenance, the structural overlay cost, the cost of surface treatments for the rehabilitation strategy and preventive maintenance treatments for the preventive maintenance strategy, and the salvage value associated with each strategy. Also, mean present worth costs associated with accidents, user delay, and excess vehicle operating costs are provided for both preservation strategies.

When the first page of the Summary Sheet is showing, the user must click the “Next Page” button in the bottom right-hand corner of the screen and the second page of the Summary Sheet appears. When the second page of the Summary Sheet is showing, the user has the option to return to the previous page or close the Summary Sheet screens. In addition, there are two check boxes that the user may select. The user may click on the first check box to print the “Summary Sheet,” or the user may click on the second check box to print the “Details Spreadsheet,” which is described below. The user may choose to select none, one, or both of the check boxes.

The second manner in which output is presented is by the “Details Spreadsheet,” which is shown in Figure 6.24. The Details Spreadsheet is a spreadsheet containing all input values used in the LCCA model, as well as detailed output generated from the model. The first page shows the input values selected by the user, including required user inputs as well as advanced user inputs. If the user does not choose to edit the advanced user input values, the default values used in the analysis are shown.

Summary Sheet - Design and Maintenance Data

INITIAL PAVEMENT DESIGN DATA

Surface Layer Thickness (in inches):	3.5	Probability of Failure (%):	35.9%
Base Layer Thickness (in inches):	7.8	Analysis Period (in years):	30
Subbase Layer Thickness (in inches):	6.0	Surface Aggregate Type:	GRA
Structural Number:	3.22		

MAINTENANCE DATA

	REHABILITATION	PREVENTIVE MAINTENANCE
Maintain PSI Above:	2.0	3.5
Maintain Skid Resistance Above:	32.0	32.0
Required Structural Overlay Thickness (in inches):	2.00	
Year Structural Overlay Is Performed:	15	
Number of Surface Treatments Required:	.566	3.653

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Summary Sheet - Cost Data

COST DATA

	REHABILITATION		PREVENTIVE MAINTENANCE	
	Mean	COV	Mean	COV
TOTAL PW COST:	\$10,640,062	11.0%	\$10,537,069	13.2%
Agency PW Costs:	\$354,961	13.2%	\$307,639	5.5%
Initial Cost of Construction:	\$305,726		\$305,726	
PW Cost of Routine Maintenance:	\$15,194		\$7,679	
PW Cost of Structural Overlay:	\$37,082			
PW Cost of Surface Treatments / Preventive Maint. Treatments:	\$4,620		\$33,262	
PW Salvage Value:	-\$31,452		-\$62,029	
User PW Costs:	\$10,285,931	12.1%	\$10,230,031	13.5%
PW Cost of Accidents:	\$10,222,439		\$10,196,592	
PW Cost of User Delay:	\$90,996		\$25,191	
PW Cost of Excess VOC:	\$12,496		\$0,249	

☐ Print Summary Sheet
 ☐ Print Details Spreadsheet

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Figure 6.23 Summary Sheet for LCCA Output.

***** INPUT *****	Mean	Std Dev	COV	
** Initial Pavement Design **				
ADT = Average Daily Traffic	5,000	500	10.0%	
%T = Percent Trucks	10%	0.01	10.0%	
TF = Truck Factor	0.38	0.038	10.0%	
G=Growth Rate	3.0%	0.003	10.0%	
N = Analysis Period	30			
PSIi = Initial PSI	4.2	0.2814	6.7%	
PSIt = Terminal PSI	2.0			
Mr = Effective Mr	5000	500	10.0%	
a1 = Surface Layer Coeff.	0.44	0.044	10.0%	
a2 = Base Layer Coeff.	0.14	0.014	10.0%	
m2 = Base Drainage Coeff.	1.0	0.1	10.0%	
a3 = Subbase Layer Coeff.	0.11	0.011	10.0%	
m3 = Subbase Drainage Coeff.	1.0	0.1	10.0%	
Surface Aggregate Type	1	(1=SRA, 2=unrestricted)		
Prob. of Failure (init pvmt dgn)	50%			
** Rehabilitation **				
Avg. Overlay Thickness (inches)	2.0			
Prob. of Failure (structural o/l)	50%			
** Constraints **				
PSI Threshold	2.0			
Skid Resistance Threshold	32.0			
** Route Classification **				
1=US/State Rte, 2=County Rte	1			
** Discount Rate **				
Discount Rate (percent)	5.0%			
** Unit Costs **				
Surface / HMA (\$/cy)	\$100	10.00	10.0%	
Base (\$/cy)	\$42	4.20	10.0%	
Subbase (\$/cy)	\$36	3.60	10.0%	
Surface Treatment (\$/lane-mile)	\$10,000	1000.00	10.0%	
** Productivity **				
Routine Maint. (\$/crew-hr)	180	18	10.0%	
Surface Treatment (sq yd/hr)	1500	150	10.0%	
Paving (tons/hr)	125	12.5	10.0%	
** Work Zone Schedule **				
Start Time (0=12am, 23=11pm)	8			
Maximum WZ Hours Per Day	8			
** Skid Resistance Model **				
Slope, Intercept	0	42.5		
** Percent Wet Time **				
Percent Wet Time	15%			
** Accident Rates **				
Typical Accident Rate (acc/hmvm)	199			
Typical Work Zone Accident Rate	338.3			
** Accident Costs **				
Fatality (\$/fatality)	\$2,600,000	260000	10.0%	
Injury (\$/injury)	\$78,333	7833.3	10.0%	
Property Damage Only (PDO)	\$2,000	200	10.0%	
** Accident Response **				
Accident Detection (min)	5	1		
Response (min)	20	2		
Clearance Time - Fatality (min)	60	20		
Clearance Time - Injury (min)	30	10		
Clearance Time - PDO (min)	10	1		
** Cost of Time **				
Autos (\$/veh-hr)	\$12.88	1.288	10.0%	
Trucks (\$/veh-hr)	\$21.47	2.147	10.0%	
** Vehicle Operating Costs **				
Idling: autos (\$/veh-hr)	\$0.70	0.07	10.0%	
Idling: trucks (\$/veh-hr)	\$0.83	0.083	10.0%	
Running Cost: autos	\$84.41	8.441	10.0%	
Running Cost: trucks	\$729.92	72.992	10.0%	
-->(\$/1000 stops, excludes idling)				

Figure 6.24 Details Spreadsheet (page 1).

***** OUTPUT *****	Mean	Std Dev	COV
INITIAL PAVEMENT DESIGN			
Traffic prediction (esals)	659,103	122,098	18.5%
Traffic prediction (log esals)	5.8119	0.0774	1.33%
SN (initial)	3.22	0.3129	9.73%
delta PSI	2.22	0.2976	13.43%
Performance prediction (esals)	1,499,931	1,299,195	86.62%
Performance prediction (log esals)	6.0643	0.3017	4.98%
Probability of failure	38.9%		
Performance prediction (Y)	15		
D1 = surface thickness (in)	3.5	0.3665	10.43%
D2 = base thickness (in)	7.0	0.7400	10.53%
D3 = subbase thickness (in)	6.0	0.6291	10.42%
REHABILITATION			
Future traffic prediction (esals)	1,031,606	204,019	19.78%
Future traffic prediction (log esals)	6.0055	0.0825	1.37%
Probability of failure (for o/l)	40.7%		
Required SN	3.43	0.3442	10.02%
Effective SN	2.7	0.7218	26.71%
Average overlay thickness (in)	2	1.7465	87.19%
Performance			
PSI at start of analysis period	4.22	0.2976	7.06%
PSI immediately before o/l	2.75	1.0049	36.52%
PSI immediately after o/l	4.21	0.2889	6.86%
PSI at end of analysis period	2.53	0.8851	34.92%
Agency Costs			
PW cost of construction	\$328,726.33	19,885.37	6.05%
Cost of structural overlay	\$78,754.58	69,476.16	88.22%
PW cost of overlay	\$37,882.30	33,419.22	88.22%
PW AMC (2*\$/lane-mi)	\$15,183.76	9,650.54	63.56%
Average AMC (2 lane-mi, t=0 to N)	\$1,212.93		
PW cost of surface treatments	\$4,620.23	12,803.41	277.12%
Average cost of surface treatment	\$20,177.83		
Avg # of surface treatments req'd.	0.5660		
PW salvage value	-\$31,451.60	21,569.31	-68.58%
TOTAL PW COST (AGENCY)	\$354,961.01	46,688.84	13.15%
User Costs			
Average accidents per year	5.43	1.56	28.71%
Average accident cost per year	\$340,747.97		
Number of accidents (t=0 to N)	162.97	19.10	11.72%
PW cost of accidents	\$10,222,439.00	1,207,908.53	11.82%
Average delay per year (hours)	364.07		
Average delay cost per year	\$5,032.47		
PW cost of delay	\$50,996.40	51,043.15	100.09%
Average excess VOC per year	\$1,077.22		
PW cost of excess VOC	\$12,495.65	7,954.41	63.66%
TOTAL PW COST (USER)	\$10,285,931.06	1,240,167.45	12.06%
TOTAL PW COST (REHAB)	\$10,640,892.06	1,253,476.73	11.78%
PREVENTIVE MAINTENANCE			
Agency Costs			
PW cost of construction	\$328,726.33	19,885.37	6.05%
PW AMC (2*\$/lane-mi)	\$7,878.78	1,854.70	23.54%
Average AMC (2 lane-mi, t=0 to N)	\$599.46		
PW cost of surface treatments	\$33,262.00	16,204.34	48.72%
Average cost of surface treatment	\$20,096.16		
Avg. # of prev. maint. treatments	3.6530		
PW salvage value	-\$62,028.57	4,887.60	-7.88%
TOTAL PW COST (AGENCY)	\$307,838.53	16,973.54	5.51%
User Costs			
Average accidents per year	5.43	1.56	28.66%
Average accident cost per year	\$339,886.41		
Number of accidents (t=0 to N)	162.88	19.05	11.70%
PW cost of accidents	\$10,196,592.35	1,375,896.95	13.49%
Average delay per year (hours)	157.99		
Average delay cost per year	\$2,181.81		
PW cost of delay	\$25,190.61	9,909.90	39.34%
Average excess VOC per year	\$650.09		
PW cost of excess VOC	\$8,248.00	2,098.88	25.45%
TOTAL PW COST (USER)	\$10,230,030.96	1,384,594.10	13.53%
TOTAL PW COST (PREV MAINT)	\$10,537,869.49	1,387,978.43	13.17%

Figure 6.24 Details Spreadsheet Continued (page 2).

PAVEMENT DESIGN:				STRUCTURAL OVERLAY THICKNESS DISTRIBUTION:			PROBABILITY OF MAINT. ACTION DURING YEAR J:		
	minimum thickness	required thickness					year	surface treatment (rehab)	preventive maint.
most cost-effective layer:	1	3	3.5	0<D<0	0	536			
2nd-most cost-effective:	2	6	7	0<D<5	0.5	367			
3rd-most cost-effective:	3	6	6	5<D<1	1	350	1	0	0
				1<D<1.5	1.5	333	2	0	0
				1.5<D<2	2	298	3	0	0.004
				2<D<2.5	2.5	233	4	0	0.019
				2.5<D<3	3	171	5	0	0.067667
				3<D<3.5	3.5	141	6	0.002	0.102667
				3.5<D<4	4	180	7	0.005	0.131667
				4<D<4.5	4.5	141	8	0.012	0.150667
				4.5<D<5	5	112	9	0.023	0.154333
				5<D<5.5	5.5	74	10	0.036	0.166
				5.5<D<6	6	32	11	0.056667	0.161
				6<D<6.5	6.5	26	12	0.083667	0.171667
				6.5<D<7	7	14	13	0	0.146333
				7<D<7.5	7.5	10	14	0	0.139
				7.5<D<8	8	2	15	0	0.153
				>8	8.5	0	16	0	0.152667
				total		3000	17	0	0.155333
							18	0	0.139333
							19	0	0.132667
							20	0	0.135333
							21	0.001333	0.147333
							22	0.007333	0.136
							23	0.018333	0.130667
							24	0.033333	0.145
							25	0.050667	0.129
							26	0.092333	0.136
							27	0.135333	0.130333
							28	0	0.144333
							29	0	0.138333
							30	0	0.133667

REHABILITATION:				AVG. PVT, CONDITION:		
	min	mean	stddev	COV (%)		
Remaining Life		0.4679	0.2990	63.91		
ESALs to date (Np)	437,279	669,103	122,098	18.52		
ESALs to failure (Nfail)	197,964	1,089,149	1,741,909	92.21		
traffic pred. > perf. pred.	667					

WORK ZONE DURATION:			
	Rehab	Prev Maint	
Aug work zone duration	7.52	4.60	

AVG. PVT, CONDITION:			
	Avg. PSI	Avg. Skid Number	
	mean	COV (%)	mean
Rehabilitation	3.44	21.83	42.5
Preventive Maintenance	3.77	7.74	42.5

Figure 6.24 Details Spreadsheet Continued (page 3).

Figure 6.24 Details Spreadsheet Continued (page 4).

Figure 6.24 Details Spreadsheet Continued (page 5).

The second page of the Details Spreadsheet presents the output generated from the LCCA model. Mean values, standard deviations, and coefficients of variation (COV) obtained from the LCCA simulation are provided for numerous outputs, including pavement design, structural overlay design, maintenance scheduling, and agency and user present worth costs associated with both preservation strategies.

The third page of the Details Spreadsheet provides the user with detailed information regarding pavement design and structural overlay design data. In addition, this page provides the user with the average work zone duration for each strategy as well as the average pavement serviceability and skid resistance for each strategy. A distribution of structural overlay thicknesses is also presented. Finally the probability of performing a surface treatment during a particular year for the rehabilitation strategy and the probability of performing a preventive maintenance treatment during a particular year for the preventive maintenance strategy are presented numerically, and a histogram is also provided to graphically represent these probabilities.

The fourth and fifth pages of the Details Spreadsheet provide the user with histograms representing the total present worth cost distributions, the agency present worth cost distributions, and the user present worth cost distributions for the rehabilitation strategy and the preventive maintenance strategy, respectively. The following section discusses the interpretation of the LCCA model results.

6.4 INTERPRETING LCCA MODEL RESULTS

The LCCA program developed in this research provides a wealth of information to the user. Histograms are included in the output to assist the user in understanding the model results. Since the LCCA model incorporates risk analysis, the model results are presented as probability distributions, rather than discrete values. This probabilistic approach allows the user to determine the likelihood of obtaining various costs associated with each preservation strategy. However, it is up to the user to first establish an acceptable level of risk, and then combine the model output with the amount of risk he or she is willing to take in order to choose the best alternative.

For example, suppose the mean total present worth cost of Alternative 1 is slightly higher than the mean total present worth cost of Alternative 2. In a deterministic analysis, the user is likely to choose Alternative 2 because it has a lower cost than Alternative 1. However, the addition of risk analysis allows the user to consider the distribution of costs. Now suppose Alternative 1, with the higher mean cost, has a very small standard deviation. This means that the

probability of having a significantly higher or lower cost than the mean cost is very small. Furthermore, suppose the standard deviation of Alternative 2 is very large, resulting in a high probability of incurring a much higher or a much lower cost than the mean cost. At this point, the user must decide how much risk is acceptable, and then base his or her decision on the amount of risk he or she is willing to take. Alternative 1 is likely to have a cost very close to the mean value. Alternative 2 may have a significantly lower cost, but there is also the probability that the cost may well exceed the cost of Alternative 1, even though the mean cost for Alternative 2 is less than the mean cost for Alternative 1.

The Details Spreadsheet shown in Figure 6.24 reveals pavement design output data, including expected traffic volumes and layer thicknesses. For the rehabilitation strategy, the average overlay thickness is provided, as well as performance data, such as the PSI immediately before and after the overlay application. Cost data for rehabilitation is presented, including a breakdown of agency cost components and user cost components. Included with the agency costs is the average number of surface treatments required throughout the analysis period. As described previously, surface treatments may be required for the rehabilitation strategy if either the PSI or skid number deteriorates below its threshold value. Included with the user cost components are mean values for the average number of accidents per year, the average delay (in hours) per year, and the average excess vehicle operating costs (VOC) per year. Similar information is provided for the preventive maintenance strategy.

At this point, it is reiterated that the output values presented in the Summary Sheet and Details Spreadsheet (page 2) are mean values obtained from averaging the output from each iteration of the LCCA simulation. Therefore, output values for layer thicknesses, structural overlay thickness, accident predictions, and the number of surface treatments required for the rehabilitation strategy or the required number of preventive maintenance treatments for the preventive maintenance strategy may or may not be integer values. The user must use discretion when using the output to select design thicknesses for pavement construction or to schedule maintenance activities.

On page 2 of the Details Spreadsheet, the user may notice very high coefficients of variation (COV) for some output variables. High COVs associated with pavement design and structural overlay design data may be attributed to the pavement design and structural overlay design procedure used in this analysis. For example, AASHTO recommends a reliability level of 50 percent for low-volume roads, which corresponds to a probability of failure used in this research of 50 percent. This means that there is a 50 percent probability that the pavement will fail prior to the time at which the structural overlay is to be applied. Therefore, it is not

uncommon to find a high degree of variability associated with the structural overlay thickness. Selecting a smaller probability of failure may be one way to decrease COV values for the pavement design and structural overlay design variables.

High COV values are also common for the present worth cost of surface treatments for the rehabilitation alternative. This factor may also be influenced by the pavement design procedure. As mentioned above, there is a 50 percent chance that the pavement will fail before the end of the performance period. There is also a 50 percent probability that the pavement will remain above the terminal PSI level throughout the performance period. Therefore, the probability that a surface treatment is required in order to maintain the PSI level above the PSI threshold is highly variable with each iteration of the simulation. While one iteration may require multiple surface treatments throughout the analysis period, another iteration may require zero surface treatments. Therefore, the total cost of surface treatments for various iterations may range from a cost of zero to a cost comprised of one or more surface treatments.

The cost distribution data presented numerically on page 2 of the Details Spreadsheet (Figure 6.24) is summarized graphically in the histograms presented on pages 4 and 5. The histograms describe the shapes of the total present worth cost, agency present worth cost, and user present worth cost distributions associated with each preservation strategy. The inclusion of the histograms enables the user to visualize both the mean value and the spread of cost data for each alternative, thereby simplifying the interpretation of the probabilistic model output.

Page 3 of the Details Spreadsheet provides the user with the probability that a maintenance action is required during a particular year of the analysis period for each preservation strategy. As defined previously, the term maintenance action may refer to either a surface treatment for the rehabilitation strategy or a preventive maintenance treatment for the preventive maintenance strategy.

For rehabilitation, these data reveal the probability that a surface treatment is required during a particular year of the analysis period. As described previously, a surface treatment is required if the PSI or skid number deteriorates below its corresponding threshold value. Furthermore, a surface treatment is never applied to a pavement within a two-year period prior to the application of the structural overlay or the end of the analysis period. This explains the gap in the exponential increase in the probability that a surface treatment is required during a particular year for the rehabilitation strategy, as shown in Figure 6.24 (page 3). Regarding preventive maintenance, the output reveals the probability that a preventive maintenance treatment is required during a particular year of the analysis period.

The probability that a maintenance action is required during a particular year is calculated by developing a matrix based on the required maintenance actions for each iteration. The rows of the matrix represent the number of iterations in the simulation and the columns represent the number of years in the analysis period. For each iteration, if a maintenance action is required during a particular year, a one is placed in the matrix. Otherwise, a zero is placed in the matrix for that particular year. When the simulation is complete, the numbers in each column of the matrix are added together and the sum is divided by the total number of iterations. This yields the fraction of time (in decimal form) that a maintenance action is required during a particular year.

If the user wishes to estimate a schedule for maintenance activities for a particular preservation strategy, the user should first note the average number of surface treatments or preventive maintenance treatments required over the analysis period, shown on page 2 of the Details Spreadsheet. Then, the years in which maintenance actions may be required can be approximated. In order to derive a maintenance schedule, the user may either refer to the data or the histogram in Figure 6.24, page 3.

The LCCA model developed for this research does not provide a discrete schedule for the timing of maintenance actions. Instead, the model provides the user with the probability that a maintenance action will be required during each year of the analysis period. Then, for each preservation strategy, the user can derive a maintenance schedule for the purpose of maintenance budget planning by combining the probabilistic LCCA model output with engineering judgment.

6.5 CASE STUDY

A case study is included in this chapter to provide guidance on the interpretation of the LCCA model results. The LCCA program developed in this research was executed with all input values shown in Figures 6.2-6.21. The model output was shown in Figures 6.23 and 6.24.

The initial pavement design developed by the model consists of a surface layer thickness of 3.5 inches, a base layer thickness of 7.0 inches, and a subbase layer thickness of 6.0 inches. The initial cost of construction is defined by a probability distribution with a mean of \$328,730 and a COV of 6 percent. This cost is added to both preservation strategies.

For the rehabilitation strategy, a 2-inch overlay is scheduled for year 15. The average overlay thickness of 2 inches has a COV of 87 percent, which is not unusual since the pavement was designed for a probability of failure of 50 percent. The mean cost of the overlay is \$78,760, which corresponds to a present worth cost of \$37,880. The COV of 88 percent reflects the high variability in the required overlay thickness. An average annual routine maintenance cost for the

rehabilitation strategy is approximately \$1,210 per mile of two-lane road. The present worth cost of the annual routine maintenance is \$15,180 with a COV of 64 percent.

The average number of surface treatments required for the rehabilitation strategy is 0.5660 (Figure 6.24, page 2). Therefore, it is likely that either zero or one surface treatment will be required throughout the analysis period. Referring to the histogram in Figure 6.24, page 3, it is likely that one surface treatment will be required toward the end of the analysis period, possibly in year 27. The average cost of a surface treatment is \$20,180, and the estimated present worth cost of the surface treatment is \$4,620 with a COV of 277 percent. The extremely high variability is due to the fact that some iterations of the simulation require one or possibly even two surface treatments (at an average cost of \$20,180 per application), while other iterations require zero surface treatments (at a cost of \$0).

The salvage value is considered a negative agency cost, and is taken as a fraction of the initial cost of the pavement. The salvage value is dependent on the predicted level of serviceability at the end of the analysis period and the initial cost of the pavement. For the rehabilitation strategy, the salvage value is defined by a mean of \$31,450 and a COV of 68 percent.

The total agency present worth cost for the rehabilitation strategy is defined by a mean of \$354,960 with a COV of 13 percent. This cost distribution is presented as a histogram in Figure 6.24, page 4.

The average number of accidents per year is 5.43, with a standard deviation of 1.56 accidents. The average annual accident cost is \$340,750. The present worth cost of accidents over the analysis period is \$10,222,440, with a COV of 12 percent. Similarly, the average delay hours per year is 364 hours, and the average delay cost per year is \$5,030. The present worth cost of delay over the analysis period is approximately \$51,000 with a COV of 100 percent. Delay is caused by work zones or accidents that occur during normal operation. If there is high variability associated with the number of surface treatments required for the rehabilitation strategy, the work zone duration throughout the analysis period will also have high variability. Finally, the average excess vehicle operating costs per year is \$1,080. The present worth cost of excess vehicle operating costs over the analysis period is defined by a mean of \$12,500 with a COV of 63 percent.

The total user present worth cost for the rehabilitation strategy is defined by a mean of \$10,285,930 with a COV of 12 percent. The total present worth cost, including agency and user costs, is \$10,640,890 with a COV of approximately 12 percent. Histograms for the total present

worth cost and user present worth cost for the rehabilitation strategy are shown in Figure 6.24, page 4.

The output for the preventive maintenance strategy is presented in a similar manner as for the rehabilitation strategy. The cost distribution for the initial pavement construction is the same for both strategies. The average annual routine maintenance cost for the preventive maintenance strategy is \$600, and the present worth cost of routine maintenance over the analysis period is defined by a distribution with a mean of \$7,880 and a COV of 24 percent.

The present worth cost distribution for preventive maintenance treatments is defined by a mean \$33,260 and a COV of 48 percent. The average number of preventive maintenance treatments required throughout the analysis period is 3.6530, as shown in Figure 6.24, page 2. Therefore, either three or four preventive maintenance treatment applications may be required. Based on the output, it is likely that four preventive maintenance treatments are required over the analysis period. The histogram in Figure 6.24, page 3 may be used to predict the years in which the treatments may be required. Based on the peaks of the histogram, preventive maintenance treatments may be required in years 10, 17, 24, and 28. Note that this is just one maintenance schedule that may be derived; there are many possible combinations of treatment timings that may be established by the user.

The salvage value for the preventive maintenance treatment is determined by the same procedure used for the rehabilitation strategy. For this example, the present worth cost of the salvage value for the preventive maintenance strategy is \$62,030 and the COV is approximately 8 percent. The total agency present worth cost for the preventive maintenance strategy is defined by a mean of \$307,840 and a COV of 6 percent.

The average number of accidents per year is 5.43, with a standard deviation of 1.56 accidents. The average annual accident cost is \$339,890. The present worth cost of accidents over the analysis period is \$10,196,590 and the COV is 13 percent. Similarly, the average delay hours per year is 160 hours, and the average delay cost per year is \$2,180. The present worth cost of delay over the analysis period is approximately \$25,190 with a COV of 39 percent. Finally, the average excess vehicle operating costs per year is \$650. The present worth cost of excess vehicle operating costs over the analysis period is defined by a mean of \$8,250 with a COV of 25 percent.

The total user present worth cost for the preventive maintenance strategy is defined by a mean of \$10,230,030 and a COV of 13 percent. The total present worth cost, including agency and user costs, is \$10,537,870 and a COV of approximately 13 percent. Histograms for the total

present worth cost and user present worth cost for the preventive maintenance strategy are shown in Figure 6.24, page 5.

Figure 6.24, page 3 provides the user with information such as the average work zone duration for each preservation strategy, as well as the distributions describing the average serviceability and skid resistance over the analysis period for each strategy. For example, the average work zone duration for the rehabilitation strategy is 7.52 hours, while the average work zone duration for the preventive maintenance strategy is 4.60 hours.

For the rehabilitation strategy, the average PSI is 3.44 with a COV of 22 percent, and the average skid number is 42.5 with a COV of 0 percent. For the preventive maintenance strategy, the average PSI is 3.77 with a COV of 8 percent, and the average skid number is 42.5 with a COV of 0 percent. The discrete value for the average skid number reflects the skid resistance prediction model for skid resistant aggregate, which has a slope of zero and an intercept of 42.5. If unrestricted aggregate is selected for the surface aggregate type, the skid resistance would deteriorate with cumulative vehicle passes, and the average skid number would be defined by a probability distribution rather than a discrete value.

At this point, it is up to the decision-maker to determine which preservation strategy is best. In this case study, the preventive maintenance strategy has a lower total present worth cost than the rehabilitation strategy. However, the variability in total present worth cost is slightly greater for the preventive maintenance strategy than for the rehabilitation strategy. In addition to cost, the decision-maker may consider the pavement serviceability and/or skid resistance in the decision-making process. In this case study, the average serviceability for the preventive maintenance strategy is maintained at a higher level of PSI than for the preventive maintenance strategy. Accident experience and user delay are additional factors that the decision-maker must consider. In this example, the average number of accidents per year is the same for both alternatives. However, user delay hours for the preventive maintenance strategy are significantly lower than for the rehabilitation strategy.

It is likely that the preventive maintenance strategy would be selected as the better alternative in this case study. However, the decision process depends on the wants and needs of the highway agency, and the allowable degree of risk the highway agency is willing to take. This risk-based approach allows the analyst to quantify all possible outcomes associated with each preservation strategy, but the analyst must combine this probabilistic output with his or her judgment to make the best decision.

6.6 SUMMARY OF LCCA MODEL EXECUTION

This chapter presented a detailed description of the LCCA model execution. Each input screen was described and illustrated, including the required user inputs as well as the optional advanced user inputs. The model output was also described and illustrated. Furthermore, this chapter provides guidance to the user regarding the interpretation of the probabilistic model output. A case study was also included.

The incorporation of risk analysis into the LCCA model requires that the user define each input variable by a probability distribution rather than a discrete value. This allows the user to consider the variability associated with each input parameter that is included in the economic evaluation. Probability distributions are also used to present the output of this unique LCCA model. The probabilistic results of this LCCA allow the user to assess the risk associated with each preservation strategy, thereby enhancing the decision-making process.

CHAPTER 7

MODEL VALIDATION AND SENSITIVITY ANALYSIS

This chapter addresses the LCCA model validation and sensitivity analysis. Section 7.1 focuses on the component model validation. Since there were no existing LCCA models that included similar input variables as well as the incorporation of variability into the model found during the literature search, the results for each of the model components were validated individually. Section 7.2 discusses aggregate model validation. Next, Section 7.3 describes the procedure used to perform the sensitivity analysis on the LCCA output, and then presents the conclusions derived from the results of the sensitivity analysis. Finally, a chapter summary is presented in Section 7.4.

7.1 COMPONENT MODEL VALIDATION

Upon completion of the LCCA model development, model validation must be performed. For this research, the validation process was established using general definitions and guidelines developed by the National Aeronautics and Space Administration's (NASA) Independent Verification and Validation facility (IV&V).

NASA's IV&V facility was instituted to:

- identify software risks that could impact the safety and success of NASA missions,
- identify missing, incomplete, and errorful requirements, and
- identify programmatic and technical risks that could impact the program schedule, cost, and quality (NASA, 2000).

According to NASA's IV&V facility, verification is "the process of determining whether or not the products of a given phase of the software development cycle fulfill the established requirements". Validation is "the evaluation of the software at the end of the development lifecycle to ensure that the product not only complies with standard safety requirements and the specific criteria set forth by the customer, but performs exactly as expected" (NASA, 2000).

Validation of the entire LCCA model derived in this research is not possible, because no other acceptable model was found that incorporates all of the input variables described in Chapter 6 into a single risk-based LCCA model such as this. Therefore, the model validation was achieved by first breaking down the LCCA model into various components and then performing validation of the model components on an individual basis. The components that were considered in the model validation include the initial pavement design, the structural overlay

design, performance models, accident prediction, user delay estimation, and various cost components.

7.1.1 Initial Pavement Design Validation

The initial pavement design model validation was performed using the AASHTO pavement design equation from the 1993 AASHTO Design Guide, which is a widely accepted procedure for pavement design. The performance of the LCCA model developed in this research was assessed by comparing the model output to the results of an example problem solved by hand using the AASHTO equation with recommended values for the reliability and overall standard deviation. The LCCA model was executed, and the structural number for the initial pavement was recorded. Using the same traffic, roadbed soil resilient modulus, initial PSI and terminal PSI values, in addition to a reliability level of 50 percent and a standard deviation of 0.45, the pavement design was derived by hand following the procedure presented in the AASHTO Design Guide (1993). This process was repeated three times, and the two corresponding structural number outputs differed by values in the hundredths each of the three times. The results showed that the initial pavement design procedure in the LCCA model performed as expected.

7.1.2 Structural Overlay Design Validation

A similar procedure was used to validate the structural overlay design. Three iterations of the model were executed, and the overlay thickness was recorded after each iteration. This process was accomplished using the debugging option within VBA (Visual Basic for Applications) for Microsoft® Excel. Following the “Remaining Life” method presented in the AASHTO Design Guide (1993) and using the same input values that the LCCA program selected, the required overlay thickness was calculated by hand. The model output and the hand-calculated output differed by required thickness values of less than five-tenths of an inch all three times. Therefore, it was accepted that the structural overlay design procedure in the LCCA model performed as expected.

7.1.3 Performance Models Validation

Validation of the performance models was complex, as there are only a limited number of existing models in the literature. For the immediate effects of a structural overlay, existing literature showed that the PSI immediately after a structural overlay is applied tends to return to the initial PSI of the original pavement (deSolminihaç and Hudson, 1995). This concept was incorporated into the LCCA model, and a series of three simulations proved that the PSI

immediately after the overlay returned to the initial PSI value plus or minus one-hundredth. For preventive maintenance, one model was found in the literature describing the immediate effects of a chip seal on the pavement condition, based on the condition of the existing pavement at the time of application (Al-Mansour and Sinha, 1994). This model was incorporated into the LCCA model developed for this research. A single iteration of the LCCA program was executed and the PSI immediately after each preventive maintenance treatment application was recorded. Calculations by hand yielded the same results as the calculations incorporated into the program.

Long-term pavement performance models were also considered in the validation process. Since the AASHTO pavement design procedure is a widely accepted method for pavement design, the long-term pavement performance model for initial pavements and pavements with structural overlays (the AASHTO equation solved for the PSI at time t) was deemed acceptable. However, validation of the preventive maintenance modeling technique was not as straightforward because no existing models describing the long-term effects of preventive maintenance on pavement condition were found in the literature. Since a unique modeling procedure was developed for the purposes of this research, validation could only be achieved by confirming that the calculations for the PSI over time were performed correctly by the LCCA program. This was accomplished by comparing the PSI for a particular year of a single iteration determined from the model to a PSI value calculated by hand using the same input values and modeling equations. The comparison proved that the LCCA model performed as expected, as the LCCA program output yielded the same values as the calculations performed by hand.

7.1.4 Accident Prediction Validation

The next step of the model validation is to check the validity of the accident prediction. Upon first glance, the average number of predicted accidents per year for a one-mile section of road appears to be high. However, the LCCA model output for accident experience was compared to the 1998 West Virginia Accident Data presented by the West Virginia Division of Highways, and the overall results were very similar.

First, the total number of vehicles that drove on West Virginia roads in 1998 was estimated by dividing the total number of vehicle-miles driven (approximately 17 billion vehicle-miles) by the total miles of paved roads in West Virginia (approximately 38,000 miles). Then, dividing the total number of vehicles by 365 yields an average ADT of approximately 1,200 vehicles per day (vpd) for all roads in West Virginia. Based on the fact that there were 47,460 accidents during 1998, and approximately 38,000 miles of paved roads, it is expected that there would be, on the average, roughly 1.25 accidents per mile during that year. Note that this yields

an average accident rate of 285 accidents per one hundred million vehicle miles (hmv), which is slightly greater than the typical accident rate default values, but significantly less than the work zone accident rate default values used for this analysis.

When the LCCA program is initialized to an ADT of 1,200 vpd, a growth rate of 1 percent (the smallest acceptable growth rate for this program), and a typical accident rate of 241 accidents per hmv (assuming a county route), the average number of accidents per year for a mile section of roadway associated with either preservation strategy ranged from 1.30 to 1.45. This average was close to the 1.25 accidents per mile averaged during the year 1998, based on the general accident data provided by West Virginia. The method used for validating accident prediction could be improved by using traffic data and accident data specifically collected from two-lane, low volume U.S., state, and county routes. However, since these data are not available, the validation procedure described above was used to show that the accident prediction component of the LCCA program performed as expected.

7.1.5 User Delay Estimation Validation

A unique method for estimating user delay at accidents and work zones was developed for this LCCA program. The method for estimating user delay involves incorporating equations obtained from the Signalized Intersection chapter of the Highway Capacity Manual (1998) into the LCCA model, as described in Chapter 5. Since actual data for user delay caused by accidents and work zones are not available for two-lane, low volume roads, the validation procedure simply involves verifying that the equations are programmed correctly and that the calculations yield precise results.

For each year of the analysis period, delay caused by accidents and work zones are estimated. The debugging option of VBA in Excel was used in order to obtain the estimated delay hours for a particular year of a single iteration. This model output was compared to two values. First, a hand-calculation was performed using the same equations that were programmed into the LCCA model. All input values and other variables (such as green time and cycle length) that were used for the LCCA model were used in the hand calculations. The model output was compared to the hand-calculated delay estimate, and the difference between the two estimates was within one-tenth of an hour. This confirmed that the equations were programmed correctly.

Next, the Highway Capacity Software, based on equations presented in the Highway Capacity Manual (1998) was used to generate delay estimates at signalized intersections. The same green times and cycle lengths defined in Chapter 5 of this dissertation were input into the Highway Capacity Software, and the output was the delay per vehicle (in seconds per vehicle).

Converting the delay time to hours per vehicle and then multiplying the delay hours per vehicle times the demand for the condition under analysis (either the demand during normal operation for accidents or the work zone demand for work zone conditions), yields the total delay hours for a particular year. The output generated from the Highway Capacity Software was compared to the output of the LCCA model, and the results differed by less than one delay hour. This confirmed that the results from the LCCA program are precise.

7.1.6 Cost Components Validation

The final step of the validation procedure for this LCCA involved the validation of cost estimates obtained from the LCCA program. The default values for the unit costs of materials, preventive maintenance treatments, routine maintenance, accidents, user delay and excess vehicle operating costs (VOC) were obtained from the literature. Therefore, the unit cost default values were used in the validation process.

Hand calculations were performed in order to validate that the program was calculating each cost correctly. For agency costs, the precision of the initial cost of construction was confirmed by comparing the mean cost obtained from the program output to a hand-calculated cost obtained using the same layer thicknesses and unit costs of materials that were used in the program. The structural overlay cost was validated in the same manner, comparing the mean output cost obtained from the LCCA program to a hand-calculated cost obtained using the same average overlay thickness and unit cost of surface material that were used in the program.

The final agency cost components that were validated include the cost of surface treatments for the rehabilitation strategy and the cost of preventive maintenance treatments for the preventive maintenance strategy. The mean cost of surface treatments for the rehabilitation strategy obtained from the LCCA program was compared to a hand-calculated cost obtained using the same unit cost of a surface treatment and average number of surface treatments that were used in the program. In a similar manner, the mean cost of preventive maintenance treatments obtained from the LCCA program was compared to a hand-calculated cost obtained using the same unit cost of a preventive maintenance treatment and the average number of treatment applications that were required in the program.

For user costs, the precision of the accident costs, user delay costs, and vehicle operating costs were confirmed using the technique described above. The program output for the average accident cost per year was compared to a hand-calculated cost obtained using the severity breakdown programmed into the LCCA model, the unit costs of accidents based on the severity level, and the average number of accidents per year (estimated by the LCCA program). The

program output for the average delay cost per year was compared to a hand-calculated cost obtained by using the same values for the ADT and percent trucks, the unit cost of time for passenger cars and trucks, and the average delay per year that were used in the program. Finally, program output for excess VOC per year was compared to a hand-calculated value obtained using the same ADT and percent trucks, the unit VOC for passenger cars and trucks, and the average delay hours that were used in the program.

Based on the validation procedure described in this section, the LCCA program performs all agency and user cost calculations as expected. An inherent assumption of this type of validation is that the component models integrated into this LCCA are correct. There is certainly room to criticize some of the component models. Models were pulled from different sources with different levels of sophistication. However, it is beyond the scope of this research to develop new component models. Now that the component models have been integrated into a comprehensive model, a sensitivity analysis can be performed to identify the parameters that have the greatest influence on the decision variables. This will give guidance to developing research recommendations.

7.2 AGGREGATE MODEL VALIDATION CONCEPTS

The risk analysis model developed in this research is a unique aggregation of component models, exercised with the Monte Carlo Simulation method to produce a distribution of the total present worth of costs for low volume road paving projects. Whenever a unique computer program is developed, the validity of the program should be established before the results of the model are implemented. Generally, validation is a process for comparing the results of a new process to either an existing process or a primary database. The risk analysis model developed during this research is unique, so comparison to an existing program is not possible. Therefore, the only option is to compare the output of the risk analysis model to a primary database. However, this type of validation is very problematical. The output of the risk analysis model is the total present worth of costs for a low-volume road paving project. Total present worth of costs is not a measurable quantity since it is dependent on several estimated variables, such as the discount rate, the cost of fatal accidents, and the number of equivalent single axle loads applied during a pavement's life.

Since validation of the final output of the model is not possible, the only option is to validate interim predictions within the model. Interim predictions are available at two levels within the risk analysis model, the output of the component models, and the aggregation of the component models into interim values.

At the component model level, there are two issues, the validity of the component models, and the reliability of the code for executing the models. The validity of the component models was not examined during this research. These models are empirical in nature and there is always room for improvement. However, the scope of this project was to build on the state-of-the-art, rather than trying to recreate it. The validity of the component models was not at issue during this research. The reliability of the coding of these models into the risk assessment model was thoroughly evaluated following procedures prescribed by NASA. This process demonstrated that the computer program properly performs the calculations.

Several interim values in the risk analysis model are candidates for a validation process. The total present worth of costs consists of the agency and user costs. The agency costs consist of the cost of new pavement construction, rehabilitation, preventive maintenance treatments and routine maintenance. The user costs consist of the costs of accidents during normal operation, wet weather accidents, work zone accidents, and delay and speed change costs resulting from work zones and accidents. Validating cost estimates is problematical. Costs are determined by multiplying a unit price by an activity, event, or consumable item. Many unit prices are not measurable. Examples include the unit price of fatal accidents, and the value of highway user's time. Since these unit prices are not measurable, the costs associated with these variables cannot be validated. Even when the unit prices are measurable, the application of the measured unit price may not be meaningful in the risk analysis context. For example, conceptually the agency cost estimates for the construction and preservation of a road are available in highway department files. However, tracking these costs over a 30 year analysis period would not provide meaningful information for a current risk analysis. The cost of constructing a pavement 30 years ago, even if adjusted to constant dollars, would not be meaningful due to the differences in materials and technology used for modern pavement construction. A meaningful analysis would use current unit prices to capture agency costs. Due to the problems with determining meaningful unit prices, the validation process should not focus on either agency or user costs. The validation process should focus on verifying the prediction of the activities, events and consumable items used in the estimate of costs.

7.2.1 Validation of Agency Items

The risk analysis model produces uses estimates of several agency items, including:

- ◇ New pavement design
- ◇ Timing of rehabilitation
- ◇ Timing of surface treatment if needed with the rehabilitation strategy

- ◇ Timing of preventive maintenance
- ◇ Need for routine maintenance in years when other treatments are not applied.

In addition, pavement serviceability and skid resistance are estimated for each year of the analysis period. These are used as inputs for the models that predict user costs.

Validation of the agency activity estimates would require a primary data base that captures the interactions between pavement design, construction, rehabilitation, preventive maintenance and routine maintenance activities and the related data on traffic loads and environmental conditions. There are two potential database sources for this validation, existing state highway agency pavement management system databases and research databases, such as the Long Term Pavement Performance (LTPP) study.

Pavement management systems have been used by state highway agencies since the late 1970s. These systems were developed to assist highway department officials with making decisions at the network level. Decisions made at this level are oriented toward the selection of projects, timing, and treatment types to a sufficient level of accuracy to identify and support budget requests. The data needed for this level of decision is not as specific as is needed for the validation of the predictions of the risk analysis model. The other major issue with the use of pavement management system databases is the fact that the information in these databases reflects the policies and decisions of the highway agency. For example, two agencies could use similar management systems but use different threshold levels for making programming decisions. Consequently, the performance of the pavements would appear different as a result of this policy decision, rather than as a function of the engineering characteristics of the pavements. Any attempt to use this type of information for calibration of the risk analysis model would require constraining operation of the model to mimic the policies of the agency. The full capability of the model could not be evaluated.

The other option for validation of the agency items in the risk analysis model is to use a research quality database, such as is being developed in the LTPP study. Unfortunately, despite the millions of dollars being invested in this study, it is not a suitable database for validation of the risk analysis model. New pavement design, rehabilitation and preventive maintenance are being addressed in separate studies within the LTPP study. Validation of the risk analysis model requires an integrated approach to investigating the interactions between pavement design, rehabilitation and preventive maintenance. Furthermore, even if the data elements were suitable for calibration of the risk analysis model, data being collected is fraught with quality issues (Evans and Eltahan, 2000, FHWA, 2000, FHWA/LTPP, 2000).

Designing a research program for validating the risk analysis model is a very complex task. This would essentially involve developing data to verify each of the component models used in the risk analysis model. Hence, as a minimum, the design of an experiment for validating the risk analysis program would be similar in concept, but more complex, than the design of the LTPP study. A large number of sections would be needed to capture all the interaction effects. The observation period must be sufficiently long to capture the serviceability and skid resistance performance of the pavements. Furthermore, the essence of the risk analysis program is the treatment of the variability of the input variables, so the database must be large enough to quantify the distribution of these variables. The variables that would compose the experimental design are essentially the input to the risk analysis program, as described previously.

7.2.2 Validation of User Items

The user items predicted by the risk analysis model are the delays and speed change cycles due to work zones and accidents, and the accident predictions as a function of skid resistance. The effects of work zones on the traffic stream could be studied. However, all studies trying to relate accidents to roadway characteristics have inherent restrictions that inhibit the development of reliable data sets.

The effects of work zones on delays and speed change cycles could be experimentally observed and quantified as a function of the type of work zone and the duration of the activity. This would require observing traffic speeds and operations on highways both during the work activity and when the work zone is not present. Factors that should be included in the observations would include the traffic characteristics, work zone traffic control methods, road design features, such as grade and curvature and environmental conditions. Developing a sufficient database to capture accident rate differentials associated with work zones would require an extensive study due to the relatively limited occurrence of accidents. Similarly, developing a database to capture the relationship between accidents and skid resistance requires an extensive and long-term study. The literature is rich in such studies, but the resulting correlations are poor. Attempts to study traffic accidents are always hindered by the fact that whenever a highway agency suspects there is an assignable cause between accidents and a roadway feature, it is incumbent on the agency to take steps to eliminate the perceived hazard. Highway agencies simply cannot leave a pavement with poor skid resistance in a hazardous condition until the researchers observe a sufficient number of accidents to establish statistically valid results. As a result, researchers are left to data mine highway department records in an attempt to glean trends from the data. Consequently, predictive models of accidents have poor reliability. Any attempt

to design an experiment to capture the relationship between skid resistance and accident rates, especially for low volume roads, would face the same inhibiting factors that have thwarted the efforts of previous researchers.

7.2.3 Summary of Aggregate Model Validation

While the need to validate the risk analysis model is recognized and discussed, this dissertation offers little toward the resolution of this issue. Proper validation of this model would require an experimental program that eclipses the largest research projects ever attempted in pavement engineering. The AASHO Road Test only addressed the new pavement design portion of the model. The LTPP study embraces many of the elements of the model, but in a piecemeal and unsuitable fashion. The World Bank has invested millions of dollars in researching the relationships between user costs and roadway characteristics. None of these studies have addressed the most perplexing, and potentially the most significant, issue of the relationship between pavement characteristics and accidents. Each of these studies was initiated with the development of an experimental plan formulated by teams of engineers, statisticians and analysts.

7.3 SENSITIVITY ANALYSIS

After the LCCA model was developed and validated, a sensitivity analysis was performed. The purpose of conducting a sensitivity analysis is to determine the significance of each input parameter in the LCCA by identifying the effects of parameter variability on model results. This informs the user as to which components within each preservation strategy have the greatest influence on the present worth life-cycle cost. By evaluating the effects of various factors on the total present worth cost, the user can assess the risk associated with each strategy.

7.3.1 Sensitivity Analysis Procedure

Because of the large number of input parameters used in this LCCA, the sensitivity analysis was focused on examining the effects of individual parameters on the model output. Seventy-four parameters were analyzed, including the means and coefficients of variation (COV) for all required user inputs as well as the advanced user inputs identified in Chapter 6. A mid-range value was established for each parameter, where the mid-range was typically the default value. The mid-range value was then multiplied by 0.85, 1.0, and 1.15, so that a total of three LCCA simulations were required for each parameter. The common range of +/- 15% of the parameter mid-range was chosen for this analysis to allow for normalization when comparing results from one particular parameter to another in order to determine the degree of significance

of each parameter. If a linear relationship does in fact exist between the input variable and the LCCA model output (present worth cost), the range of input values (in this case +/- 15%) is arbitrary since the slope of the line will remain constant for any range of input values.

After the LCCA model was executed three times for each parameter under consideration (at 0.85 times the mid-range, the mid-range, and 1.15 times the mid-range), a linear regression analysis was performed on the present worth cost distributions. The present worth cost distributions considered in the sensitivity analysis included the total present worth cost, the agency present worth cost, and the user present worth cost for each preservation strategy.

A linear regression model was used to approximate the relationship between the response variable Y (present worth cost), and the single explanatory variable x (a particular LCCA model input), by a linear function. The linear regression model is written as (Hogg and Ledolter, 1992):

$$Y_i = \beta_0 + \beta_1 x_i + \varepsilon_i \quad \text{for } i = 1, 2, \dots, n \quad (7.1)$$

β_0 and β_1 are the coefficients in the linear relationship. β_0 defines the intercept and β_1 defines the slope. Therefore, a change of one unit in the explanatory variable x translates into a change of β_1 units in the response variable. The random variables ε_i are errors that represent the scatter around the linear relationship. It is assumed that the errors are mutually independent and normally distributed with a mean of zero and a variance of σ^2 (Hogg and Ledolter, 1992).

After the regression equation was estimated for a particular input variable, a hypothesis test was performed on the regression coefficients. For this research, the hypothesis test of interest was the following:

$$H_0: \beta_1 = 0$$

$$H_1: \beta_1 \neq 0$$

The null hypothesis, H_0 , assumes that β_1 , or the slope, is equal to zero. If H_0 is not rejected, it is concluded that the explanatory variable x is not important in explaining the variability in the response variable Y (Hogg and Ledolter, 1992). If the null hypothesis is rejected, the alternative H_1 implies that x does have a significant linear association with Y .

The test statistic used for this hypothesis test is the *t-statistic*, which is calculated by dividing the least-squares estimate β_1 by the standard error for β_1 as shown in the equation below:

$$t_{\hat{\beta}_1} = \frac{\hat{\beta}_1}{s(\hat{\beta}_1)} \quad (7.2)$$

At a significance level of α , the null hypothesis is rejected if the inequality shown in Equation 7.3 is true.

$$\left| t_{\hat{\beta}_1} \right| \geq t(\alpha / 2; n - 2) \quad (7.3)$$

Otherwise, there is not enough evidence to reject the null hypothesis. If the null hypothesis is not rejected, the input variable is considered to have an insignificant effect on the LCCA model output. If the null hypothesis is rejected and the alternative is accepted, the input variable is considered to have a statistically significant effect on the LCCA model output.

After the completion of the hypothesis tests, the significant input variables were identified and used for further analysis to determine the degree to which each variable influenced the output distribution. As mentioned previously, the output distributions considered in the sensitivity analysis included the total present worth cost, the agency present worth cost, and the user present worth cost associated with the rehabilitation strategy and the preventive maintenance strategy, respectively.

Each input variable that was statistically significant with respect to a particular output distribution was normalized, or divided by that variable's mid-range input value. For example, if ADT was statistically significant to the total present worth cost of rehabilitation, each of the three ADT input values was divided by the mid-range ADT value (5,000 vehicles per day). Next, a graph was constructed where the three variable input values were plotted on the x-axis and their corresponding output distribution means were plotted on the y-axis. The normalization allowed the input values for different variables to be plotted on a common scale. The slopes corresponding to each input variable were then compared to one another, and the input variables with the steeper slopes were considered to have a greater effect on the output distribution. The results of the sensitivity analysis are discussed in the following section.

7.3.2 Sensitivity Analysis Results

Seventy-four input variables were individually analyzed to determine the effects of each variable on the LCCA results. The output distributions considered for this analysis included the total present worth cost, the present worth agency cost, and the present worth user cost for both

preservation strategies. The statistically significant variables associated with each of the six output distributions are identified in Tables 7.1-7.6 and their corresponding output means (in present worth dollars) are plotted in Figures 7.1-7.6.

7.3.2.1 Total Present Worth Cost – Rehabilitation Strategy

For the total present worth cost of rehabilitation, the variables that were identified as statistically significant are shown in Table 7.1. Figure 7.1 shows a graph of the total present worth cost for the rehabilitation strategy for each of the statistically significant input variables. The input variables with the steeper slopes are considered to have a greater effect on the total present worth cost. The lines intersect at the mid-range point, where all input variables are set at their mid-range or default values.

The analysis period and route classification are associated with the steepest slope in Figure 7.1. Obviously, the longer the time period that the highway agency must maintain the pavement, the greater the total present worth cost. Route classification also has a high degree of significance due to its correlation with accidents. West Virginia Accident Data (1990-1999) reported higher accident rates on county routes than U.S. and state routes, which may be due to the fact that county routes are often designed to lesser geometric design standards than U.S. and state routes. Since the cost of accidents is extremely high, an increase in accident experience has a significant effect on the total present worth cost for both rehabilitation and preventive maintenance. The default option for route classification is a U.S. or state route, but the user may instead select the county route option for the analysis. Selecting a county route automatically increases the accident rate used in the analysis, thereby increasing the probability of accidents occurring, resulting in an increase in total present worth cost.

Two additional input variables with relatively steep slopes in Figure 7.1 are the ADT mean and the normal accident rate. With respect to ADT, higher traffic volumes require a thicker initial pavement design and may also require more frequent maintenance actions. In addition, higher traffic volumes result in a greater number of accidents, which will greatly increase the total present worth cost of rehabilitation. The normal accident rate also has a significant effect on the total present worth cost of rehabilitation. Again, as accident experience increases, the present worth costs rise considerably.

The last observation noted from Figure 7.1 is that the surface layer coefficient mean has an indirect relationship with the total present worth cost of rehabilitation. An increase in the surface layer coefficient represents an increase in the structural capacity of the surface layer material, which means that a thinner layer of material is required to provide the same structural capacity. The results of the sensitivity analysis indicate that increasing the value of the layer

coefficient, or making the surface material more durable, results in a statistically significant decrease in the total present worth cost. However, this conclusion can be somewhat misleading, as the quality of the material should be reflected in the cost of the material. There is usually a positive correlation between costs and quality. This correlation was not considered in the sensitivity analysis, hence the potential for misleading results.

Table 7.1 Statistically Significant Input Variables for Total Present Worth Cost for Rehabilitation Strategy.

INPUT VARIABLE	INPUT VALUE	TOTAL PRESENT WORTH COST	T-STATISTIC
ADT Mean (vpd)	4250	10,186,480	62.8
	5000	11,972,800	
	5750	13,663,200	
Analysis Period (years)	25	9,239,080	30.5
	30	11,920,060	
	34	14,324,630	
Cost of Injury Mean (\$)	66583.05	11,028,550	141.4
	78333.00	11,921,450	
	90082.95	12,836,480	
Cost of Fatality Mean (\$)	2,210,000	11,200,630	4384.0
	2,600,000	11,911,290	
	2,990,000	12,622,510	
Growth Rate Mean (%)	2.55	11,113,620	29.8
	3.00	11,935,750	
	3.45	12,859,220	
Normal Accident Rate (accidents per 100 mvm)	1.6915E-06	10,182,100	419.9
	1.9900E-06	11,902,200	
	2.2885E-06	13,608,170	
Route Classification	1 = U.S. / State	11,880,490	98.8
	2 = County	14,313,180	
Surface Layer Coefficient Mean	0.374	11,930,420	-18.5
	0.440	11,905,810	
	0.506	11,876,110

Figure 7.1 Effect of Significant Input Variables on Total Present Worth Cost for Rehabilitation Strategy.

7.3.2.2 Total Present Worth Cost – Preventive Maintenance Strategy

The input variables identified as statistically significant with respect to the total present worth cost for the preventive maintenance strategy are shown in Table 7.2. A plot of the input variables versus the total present worth cost for the preventive maintenance strategy is shown in Figure 7.2.

Although there is a slight difference between the statistically significant input variables for the total present worth cost of preventive maintenance and the rehabilitation strategy, the variables with the greatest effect on the total present worth cost are the same for both strategies. The steepest slope in Figure 7.2 corresponds to the analysis period and the route classification. Other variables that have a significant effect on the output distribution are the ADT mean and the normal accident rate.

Table 7.2 Statistically Significant Input Variables for Total Present Worth Cost for Preventive Maintenance Strategy.

INPUT VARIABLE	INPUT VALUE	TOTAL PRESENT WORTH COST	T-STATISTIC
ADT Mean (vpd)	4250	10,065,980	87.4
	5000	11,835,070	
	5750	13,535,460	
ADT COV (%)	8.5	11,760,470	27.9
	10.0	11,805,760	
	11.5	11,857,040	
Analysis Period (years)	25	9,092,000	26.3
	30	11,770,090	
	34	14,215,870	
Cost of Fatality Mean (\$)	2,210,000	11,083,920	188.1
	2,600,000	11,770,180	
	2,990,000	12,469,200	
Cost of Injury Mean (\$)	66,583.05	10,902,670	4607.0
	78,333.00	11,819,190	
	90,082.95	12,735,010	
Growth Rate Mean (%)	2.55	11,020,600	19.0
	3.00	11,789,440	
	3.45	12,712,120	
Normal Accident Rate (accidents per 100 mvm)	1.6915E-06	10,073,250	113.3
	1.9900E-06	11,762,920	
	2.2885E-06	13,505,050	
Route Classification	1 = U.S. / State	11,713,200	61.3
	2 = County	14,186,260

Figure 7.2 Effect of Significant Input Variables on Total Present Worth Cost for Preventive Maintenance Strategy.

7.3.2.3 Agency Present Worth Cost – Rehabilitation Strategy

Only two input variables were identified as statistically significant with respect to the total agency present worth cost for the rehabilitation strategy, as listed in Table 7.3. A plot of the input variables versus the total agency present worth cost for the rehabilitation strategy is shown in Figure 7.3.

The input variable with the greatest effect on the agency present worth cost of rehabilitation is the unit of cost materials mean (\$/cubic yard) for the surface layer, or the unit cost of hot-mix asphalt (HMA) with skid-resistant aggregate (SRA is the default option for the surface aggregate type). An increase in the unit cost of HMA increases the initial cost of construction as well as the cost of the structural overlay. The surface layer coefficient COV was also identified as a statistically significant input variable that directly effects the total agency present worth cost of rehabilitation to a much lesser degree.

Table 7.3 Statistically Significant Input Variables for Agency Present Worth Cost for Rehabilitation Strategy.

INPUT VARIABLE	INPUT VALUE	TOTAL PRESENT WORTH COST	T-STATISTIC
Surface Layer Coefficient COV (%)	8.5	371,300	20.2
	10.0	387,260	
	11.5	400,710	
Unit Cost of Materials, Surface (HMA), Mean (\$/cy)	85	330,310	12.9
	100	373,160	
	115	429,280

Figure 7.3 Effect of Significant Input Variables on Agency Present Worth Cost for Rehabilitation Strategy.

7.3.2.4 Agency Present Worth Cost – Preventive Maintenance Strategy

The input variables that were identified as statistically significant with respect to the total agency present worth cost for the preventive maintenance strategy are listed in Table 7.4. Again, only two input variables were identified as statistically significant. An increase in the unit cost of surface materials COV had a slightly greater direct effect on the agency present worth cost than an increase in the ADT COV. A plot of the two input variables versus the total agency present worth cost for the preventive maintenance strategy is shown in Figure 7.4.

Table 7.4 Statistically Significant Input Variables for Agency Present Worth Cost for Preventive Maintenance Strategy.

INPUT VARIABLE	INPUT VALUE	TOTAL PRESENT WORTH COST	T-STATISTIC
ADT COV (%)	8.5	312,130	31.7
	10.0	321,260	
	11.5	329,430	
Unit Cost of Materials, Surface, COV (%)	85	304,340	34.6
	100	324,000	
	115	341,780

Figure 7.4 Effect of Significant Input Variables on Agency Present Worth Cost for Preventive Maintenance Strategy.

7.3.2.5 User Present Worth Cost – Rehabilitation Strategy

The input variables identified as statistically significant with respect to the total user present worth cost for the rehabilitation strategy are listed in Table 7.5. A plot of the normalized input values versus the user present worth cost is shown in Figure 7.5. The table and figure shown below appear to be very similar to Table 7.1 and Figure 7.1, respectively. This is due to the fact that user costs account for the majority of the total cost for both the rehabilitation and preventive maintenance strategies.

The input variables with the greatest effect on user costs for the rehabilitation strategy are the analysis period and route classification. Route classification has a high degree of significance on the user present worth cost due to its correlation with accidents. The justification for this assessment was presented in Section 7.2.2.1. Other input variables that showed a considerable effect on the output include the ADT mean and the normal accident rate, followed by the cost of injury mean, the growth rate mean, and the cost of fatality mean.

Table 7.5 Statistically Significant Input Variables for User Present Worth Cost for Rehabilitation Strategy.

INPUT VARIABLE	INPUT VALUE	TOTAL PRESENT WORTH COST	T-STATISTIC
ADT Mean (vpd)	4250	9,847,270	81.5
	5000	11,601,770	
	5750	13,283,300	
Analysis Period (years)	25	8,874,210	29.3
	30	11,538,940	
	34	13,940,330	
Cost of Fatality Mean (\$)	2,210,000	10,841,570	134.4
	2,600,000	11,535,930	
	2,990,000	12,248,410	
Cost of Injury Mean (\$)	66583.05	10,599,340	150.2
	78333.00	11,546,230	
	90082.95	12,471,540	
Cost of PDO Mean (\$)	1700	11,507,030	52.4
	2000	11,526,630	
	2300	11,547,560	
Growth Rate Mean (%)	2.55	10,745,350	32.2
	3.00	11,568,090	
	3.45	12,484,340	
Normal Accident Rate (accidents per 100 mvm)	1.6915E-06	9,819,220	668.5
	1.9900E-06	11,522,200	
	2.2885E-06	13,234,020	
Route Classification	1 = U.S. / State	11,524,230	74.8
	2 = County	13,931,370

Figure 7.5 Effects of Significant Input Variables on User Present Worth Cost for Rehabilitation Strategy.

7.3.2.6 User Present Worth Cost – Preventive Maintenance Strategy

The input variables that were found to be statistically significant with respect to the total user present worth cost for the preventive maintenance strategy are identified in Table 7.6. Many of the input variables listed in Table 7.6 are also variables that were statistically significant to the total present worth cost for the preventive maintenance strategy (identified in Table 7.2). The input variables that have the greatest effect on user costs are the analysis period and route classification. Other variables that considerably effect the user present worth cost for the preventive maintenance strategy include the ADT mean and the normal accident rate, as well as cost of injury mean, growth rate mean, and the cost of fatality mean. Figure 7.6 shows a graph of the total user present worth cost for the preventive maintenance strategy for each of the statistically significant input variables.

Table 7.6 Statistically Significant Input Variables for User Present Worth Cost for Preventive Maintenance Strategy.

INPUT VARIABLE	INPUT VALUE	TOTAL PRESENT WORTH COST	T-STATISTIC
ADT Mean (vpd)	4250	9,778,710	104.9
	5000	11,520,400	
	5750	13,205,540	
ADT COV (%)	8.5	11,448,340	19.7
	10.0	11,484,500	
	11.5	11,527,610	
Analysis Period (years)	25	8,799,580	25.5
	30	11,444,780	
	34	13,870,980	
Cost of Fatality Mean (\$)	2,210,000	10,779,800	79.4
	2,600,000	11,450,700	
	2,990,000	12,151,490	
Cost of Injury Mean (\$)	66583.05	10,538,960	100.1
	78333.00	11,494,940	
	90082.95	12,418,430	
Growth Rate Mean (%)	2.55	10,703,680	21.0
	3.00	11,476,930	
	3.45	12,389,040	
Normal Accident Rate (accidents per 100 mvm)	1.6915E-06	9,759,420	96.8
	1.9900E-06	11,439,420	
	2.2885E-06	13,180,610	
Route Classification	1 = U.S. / State	11,455,140	55.0
	2 = County	13,859,000

Figure 7.6 Effects of Significant Input Variables on User Present Worth Cost for Preventive Maintenance Strategy.

7.3.3 Comments on Sensitivity Analysis Results

The significant input parameters affecting the total present worth cost and user present worth cost were all associated with accidents. This is not surprising, because accident costs overwhelm the total and user present worth costs for both preservation strategies.

Significant input parameters for agency present worth costs were somewhat unexpected. Only ADT and/or unit cost of the surface layer were significant for either strategy. Conventional wisdom indicates that ESALs (based on ADT, percent trucks, truck factor, and growth rate) should have a significant effect on the agency present worth cost. Pavement designs should be increased to accommodate higher traffic loads. With this in mind, a sensitivity analysis was performed to determine the effect of ESALs, or the effect of the interaction of ADT, percent trucks, truck factor, and growth rate on agency present worth cost. By varying combinations of traffic parameters, agency present worth costs for a range of ESALs was obtained, starting with the minimum number of ESALs accepted for this LCCA program and ending with the maximum

number of ESALs. The output is plotted in Figure 7.7. A curvilinear relationship exists between ESALs and agency present worth cost. For the sensitivity analysis, when values were input into the LCCA program, the portion of the plot that was analyzed (the range in ESALs) did not show a significant effect on the agency present worth cost. But when the ESALs were analyzed from the minimum to the maximum number of allowable ESALs, a logarithmic relationship became apparent. Thus, the interaction of ADT, percent trucks, truck factor, and growth rate (ESALs) may have a significant effect on the output (agency present worth cost), depending on which portion of the agency present worth cost versus ESALs curve is under consideration.

For minimum ESALs, the pavement design is constrained by the minimum allowable layer thicknesses. For the rehabilitation strategy, any combination of traffic parameters resulting in less than 260,000 ESALs over the analysis period results in a pavement design consisting of minimum allowable layer thicknesses (because a 1.5, 2.0, or 2.5-inch structural overlay is considered in the design process). Since the majority of agency cost comes from the initial cost of construction, the agency present worth cost is constrained by the minimum pavement design, rather than the traffic loadings.

Figure 7.7 Trends in ESALs versus Agency Present Worth Cost.

7.4 SUMMARY OF MODEL VALIDATION AND SENSITIVITY ANALYSIS

This chapter addressed the model validation and the sensitivity analysis. Validation of the LCCA model developed for this research was performed by investigating the output of various model components. The components considered in the validation process included the initial pavement design, the structural overlay design, performance prediction models for both preservation strategies, accident prediction, user delay estimation, and various cost calculations. In cases where data were available, such as accident experience, the existing data were compared to the LCCA model output. However, in most cases, validation was achieved by comparing the LCCA program output with calculations performed by hand using the same input and/or default values and the same equations that were programmed into the LCCA model. It was concluded that all model components investigated during the model validation process performed as expected.

A sensitivity analysis was performed to determine the significance of each input parameter in the LCCA by identifying the effects of parameter variability on model results. The results from the sensitivity analysis provide the user with information as to which components within each preservation strategy have the greatest influence on the present worth life-cycle cost.

The parameters that were identified as having the greatest influence on the total present worth cost for both preservation strategies included the route classification, the analysis period, the normal accident rate, the ADT mean, the growth rate mean, the mean cost of a fatality, and the mean cost of an injury. The significance of route classification is due to the fact that accident rates vary based on the type of roadway. For example, in West Virginia, U.S. and state routes tend to have lower accident rates than county routes (West Virginia Accident Data, 1999). Therefore, changing the route classification from a U.S. or state route to a county route results in a higher accident prediction, thereby increasing the cost associated with accidents. Since accident costs are extremely high in comparison to agency costs, an increase in the number of accidents will greatly increase the total present worth cost of either preservation strategy. Agency cost considered in this analysis was limited to paving costs. The costs of upgrading geometry from county to US highway standards were not considered in this analysis.

The results of the sensitivity analysis may be used to enhance the LCCA output. If a particular parameter is identified as greatly influencing the LCCA results, the user should focus on providing the best, most accurate estimate possible for that parameter. This may require establishing a research project for collecting appropriate data so that a more accurate estimate of the parameter can be made. Recommendations for enhancing model output based on the results of this sensitivity analysis are provided in Chapter 8.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

Recently there has been a shift in the focus of federal highway funding from the construction of new pavements toward the preservation of existing pavements. Therefore, highway agencies have directed their attention toward finding preservation strategies that yield the greatest value from existing pavements. The research presented herein involves the development of a risk-based model for evaluating the life-cycle costs of various flexible pavement designs and preservation alternatives. The model is a project-level, probabilistic life cycle cost analysis (LCCA) that considers the inherent uncertainty associated with input variables. Additional features of the model include the incorporation of functional aspects (structural capacity and pavement condition) and safety (skid resistance) into the design, the inclusion of rehabilitation and preventive maintenance as preservation strategy alternatives, and the inclusion of both agency and user costs in the present worth cost analysis.

8.1 SUMMARY AND CONCLUSIONS

A detailed literature search was conducted, and no risk-based LCCA models that include all of the same features as the model developed in this research were found. Therefore, the literature review presented in Chapter 2 focuses on the individual model components that were integrated into this unique LCCA. Models for the initial pavement design, the structural overlay design, pavement performance prediction, skid resistance prediction, accident prediction, and user delay estimation were selected from the existing literature and incorporated into this LCCA. In addition, construction costs, accident rates, clearance times and costs, user delay costs, and vehicle operating costs presented in the literature were used to establish default values for the LCCA. The existing models and parameter values necessary for the development of this LCCA were incorporated into the program, even in cases when the components were not as robust as would be desired. For the models or parameters that do not exist or that lack appropriate validation, research programs should be established to develop or refine the integrated models or parameter values.

Chapter 3 presented the initial pavement design model. The pavement design procedure used for this LCCA was derived in a unique manner combining a general form of the AASHTO flexible pavement design equation with a risk-based approach that captures the effects of variability within the design process. In the AASHTO pavement design procedure, the user must

select a reliability level, and then assume an overall standard deviation that accounts for the variability associated with each of the input variables used in the traffic prediction and the performance prediction. AASHTO recommends an overall standard deviation of 0.45 for flexible pavements, regardless of the accuracy or precision of each of the discrete input values.

The incorporation of risk analysis into the pavement design model allows the user to define the variability associated with each input parameter, thereby eliminating the need for assuming a value for the overall standard deviation. With this approach, the variability associated with the traffic prediction and the performance prediction is separated into two distributions. The feature of this probabilistic approach is that inherent variability is captured in the pavement design process.

The generation of preservation strategies was described in Chapter 4. The two types of preservation strategies included in this LCCA were preventive maintenance and rehabilitation. The preventive maintenance strategy consists of preventive maintenance treatments as well as routine maintenance activities. The rehabilitation strategy consists of a structural overlay at some point in time throughout the analysis period, routine maintenance activities, and surface treatments, if required.

Pavement serviceability and skid resistance directly influence the scheduling of maintenance activities for both preservation strategies. For the preventive maintenance strategy, a preventive maintenance treatment is scheduled when the PSI or skid number of the pavement deteriorates below the PSI threshold or the skid resistance threshold. Similarly, for the rehabilitation strategy, a surface treatment is required when the PSI or skid number deteriorates below its corresponding threshold value. For each iteration of the LCCA program, a schedule for maintenance activities is derived for each preservation strategy. When the simulation is complete, the probability that a preventive maintenance treatment for the preventive maintenance strategy or a surface treatment for the rehabilitation strategy is required for each year of the analysis period is calculated. Thus, the LCCA program output does not provide the user with a discrete maintenance schedule for each preservation strategy. Instead, the LCCA output provides the user with the likelihood that a preventive maintenance treatment or a surface treatment is required in a particular year. From this probabilistic output, the user can predict a likely maintenance schedule for each preservation strategy.

In order to model the pavement performance over time, both the immediate effects and long-term effects of rehabilitation and preventive maintenance must be established. The model presented by AASHTO was incorporated into this LCCA program for predicting pavement performance over time for the rehabilitation strategy. However, the literature search revealed that

there are limited data describing the effects of preventive maintenance over time. Thus, a pavement performance model for describing the long-term effects of preventive maintenance was developed for this research.

Chapter 5 presented the life-cycle cost models. The LCCA developed for this research considers both agency costs and user costs. Agency costs include costs associated with the initial cost of construction, routine maintenance, rehabilitation, and preventive maintenance. User costs considered in this analysis include costs due to accidents, user delay, and excess vehicle operating costs. Future costs are discounted to a net present value, allowing for the comparison of the two preservation strategy alternatives.

The incorporation of risk analysis captures variability in the cost calculations. This allows the user to consider the variability of various cost components and thereby assess the risk associated with each alternative. The present worth costs obtained from the LCCA model are defined by probability distributions that describe the likelihood of all possible outcomes. The probabilistic nature of this LCCA model exposes uncertainty that may be hidden in a deterministic model, which may be very significant information to the analyst during the decision-making process.

The LCCA model execution was described in great detail in Chapter 6. The LCCA developed in this research was programmed in Visual Basic for Applications, and is executed as a macro in Microsoft® Excel 97. The various required user input screens and advanced user input screens, along with all default values, were presented. In addition, a discussion was presented regarding the interpretation of LCCA model results. It is critical that the user understands the probabilistic model results, and uses the benefits of risk analysis to enhance the decision-making process.

A case study was also presented in Chapter 6. The case study presents a sample run of the LCCA program, including all input values and LCCA model output. The case study should be used as a guide for interpreting the LCCA results. However, it is the analyst's responsibility to determine the amount of risk he or she is willing to take. The decision-making process should be based on this factor in combination with the probabilistic model results.

Chapter 7 presented the model validation and sensitivity analysis. Validation of the LCCA model developed for this research was performed by investigating the output of various model components. In cases where data were available, the existing data were compared to the LCCA model output. However, in most cases, validation was achieved by comparing the LCCA program output with hand calculations using the same input and/or default values. Based on this validation process, it was concluded that all model components performed as expected.

A sensitivity analysis was conducted to determine the significance of each input parameter on the LCCA model output. The parameters that were identified as having the greatest influence on the total present worth cost for both preservation strategies are:

- route classification,
- analysis period,
- normal accident rate,
- ADT mean,
- growth rate mean,
- mean cost of a fatality, and
- mean cost of an injury.

As described in Chapter 7, route classification has a direct relationship with the accident rates selected for the analysis.

Since accident experience has the most significant effect on the total present worth cost distribution for either preservation strategy, the user should input the most accurate accident rates possible. Furthermore, the user should also focus on entering accurate traffic data, in particular, the average daily traffic and growth rate.

8.2 RECOMMENDATIONS FOR FUTURE WORK

This research presented the development of an integrated risk-based life-cycle cost analysis for various pavement preservation strategies. Throughout the model development, validation process, and sensitivity analysis, several issues were discovered that require further development and refinement before this research is implemented into practice.

Recommendations for future work are presented below.

- Improve the user-program interface:
 - Integrate user input screens where only a few inputs are required. This will reduce the number of input screens that the user must view.
 - Provide more user-friendly options such as an initial screen that allows the user to select the input screens he or she wishes to view or edit, and a screen informing the user how much run-time is remaining.
 - Provide an option that allows the user to permanently save the current input values as default values for all future model executions.
- Add an option for analyzing existing pavements. To enhance the performance modeling procedure, non-destructive testing may be performed on the existing pavement and the measurements may be incorporated into the analysis. The effective

structural number determined from non-destructive testing is much more accurate than the effective structural number derived from AASHTO's remaining life method.

- Incorporate the uncertainty of each model component into the LCCA program. Regression models are deterministic, but unless the correlation coefficient is equal to one ($R^2 = 1$), these models do not perfectly describe the variability in the data used for developing the model. Hence, the standard error term of the regression model would be a source of variation that should be captured in the risk analysis.
- Expand the scope of the LCCA model:
 - Consider the inclusion of multi-lane, high traffic volume roads in the analysis.
 - Consider traffic control costs in the agency cost calculations.
- Establish a research program to validate the models for the long-term effects and interactions of pavement design, rehabilitation, preventive maintenance, and routine maintenance.
- Collect and analyze data to verify the accuracy of the model used to describe the immediate effects of preventive maintenance.
- Verify the accident prediction methodology. Collect traffic and accident data on low-volume U.S., state, and county routes to examine accident experience, including those accidents that occur during wet weather or work zone conditions. Since accidents generate the greatest present worth cost in this LCCA, attention should be focused on deriving the most accurate accident rates and prediction models as possible.
- Verify the user delay estimation procedure. Collect user delay data at work zones on two-lane, low traffic volume roads and compare the data to the user delay estimation model results.
- Verify accident detection, response time, and clearance time default values. Collect data related to these parameters from accidents that occur on low traffic volume, two-lane rural roads.
- Develop a fuzzy logic method for interpreting risk. Base the decision process on the amount of risk the highway agency is willing to take and the probabilistic LCCA results.
- Develop a brief training course for analysts. Probabilistic analysis methods may be unfamiliar to many practicing engineers. Successful implementation of a risk-based procedure for pavement LCCA may require further education in order for the users to

fully understand the benefits of a probabilistic analysis method relative to a deterministic method.

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VITA

Jennifer A. Reigle was born in Aberdeen, Maryland on May 11, 1972. She was raised in Newport, Pennsylvania, and graduated from Newport Junior/Senior High School in June 1990. In May 1994, Jennifer received her Bachelor's degree in Mathematics from West Virginia University in Morgantown, West Virginia. She then focused her studies on Operations Research and Transportation Engineering, and obtained her Master's degree in Engineering from West Virginia University in December 1996. Her Master's research involved the development of a universal distress index for pavements using fuzzy logic. Jennifer presented her research at the 1996 and 1997 Transportation Research Board Annual Meetings in Washington, D.C. and also published her research results in Transportation Research Record 1592. In addition, she participated as a guest speaker and panelist in the U.S. Department of Transportation's National Transportation Education Conference in Knoxville, Tennessee in October 1996.

Jennifer remained in Morgantown and pursued her Ph.D. in Civil Engineering with an emphasis in Transportation Engineering and Pavement Design. Throughout her years as a Ph.D. student, she was employed as a teaching assistant and a graduate research assistant, and was actively involved with the Asphalt Technology Laboratory at West Virginia University. She also fulfilled the first step to becoming a Professional Engineer by passing the Fundamentals of Engineering exam. Currently, Jennifer is a candidate for the Doctor of Philosophy degree in Civil Engineering at West Virginia University, and plans to graduate in December 2000.

Jennifer is employed as a Project Engineer at CGH Pavement Engineering, Inc. in Mechanicsburg, Pennsylvania, where she currently resides.