EVALUATION OF PAVEMENT MANAGEMENT DATA AND ANALYSIS OF TREATMENT EFFECTIVENESS USING MULTI-LEVEL TREATMENT TRANSITION MATRICES

By

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ABSTRACT

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Since the early 1970's, several State Highway Agencies (SHAs) have developed elaborate pavement condition and distress data collection systems. Advancements in the data collection techniques precipitated improvements in the data usage. Such improvements include better modeling of the time-series pavement condition and distress data to assess pavement performance and to predict future conditions. Available time-series pavement condition and distress data were obtained from the Colorado Department of Transportation (CDOT), the Louisiana Department of Transportation and Development (LADOTD), the Michigan Department of Transportation (MDOT), the Washington State Department of Transportation (WSDOT), and the Minnesota Road Research project (MnROAD). The data for five pavement condition and distress types and six pavement treatment types were modeled with the appropriate mathematical functions and used to; 1) assess the pavement conditions and rates of deterioration before and after the treatment and the treatment benefits; 2) develop methodologies for estimating the pavement conditions and distresses of the passing lanes using the driving lane condition and distress data and the traffic distribution factors; and 3) determine whether or not the data support the analyses of the cost effectiveness of various pavement treatments.

It is shown that: 1) the existing data support the data modeling and the consequent prediction of future pavement conditions, distresses, and rates of deterioration. 2) the conditions and distresses of the pavements in the passing lanes could be accurately predicted using the

newly developed methodologies, the historical condition and distress data of the driving lane, and traffic distribution. 3) the details of the existing cost data did not support cost effective analyses of pavement treatments; rather treatment effectiveness was analyzed and discussed using one newly developed algorithm and two existing ones.

In addition, treatment transition matrices (T^2Ms) were developed to conveniently display the results of the analyses before and after treatment and the treatment benefits in a matrix format. The data in the T^2Ms were also used to perform statistical analysis to determine whether or not the pavement condition states after treatment are associated to those before treatment.

Finally, step-by-step guidelines and procedures for the implementation of the findings of this study were developed and are included in Chapter 5 of this dissertation.

TO THOSE WHO BELIEVE IN ME AND FOR WHOM I HOLD CLOSE TO MY HEART

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In memory of Matthew J. Brundage

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LIST OF ACRONYMS

4R – Resurfacing, Restoration, Rehabilitation, and Recycling

AAA – American Automobile Association

AASHO – American Association of State Highway Officials

AASHTO – American Association of State Highway and Transportation Officials

ARFCOM – Australian Road Research Board’s Road Fuel Consumption

ASTM – American Society for Testing and Materials

AT – After Treatment

BMP – Beginning Mile Point

BPR – Bureau of Public Roads

BT – Before Treatment

CDOT – Colorado Department of Transportation

COBA – Cost Benefit Analysis

COL – Center of Lane

CS – Condition State

DI – Distress Index

DOT – Department of Transportation

DP – Distress Point

DRR – Deterioration Rate Reduction

DSL – Design Service Life

EI – Effectiveness Index

EMP – Ending Mile Point

ESAL – Equivalent Single Axle Load

EUAC – Equivalent Uniform Annual Cost

FHWA – Federal Highway Administration

FWD – Falling Weight Deflectometer

GIS – Geographical Information System

GPR – Ground Penetrating Radar

GPS – Global Positioning System

HA – Highway Authority

HDM-III or IV – Highway Design and Maintenance Standards

HMA – Hot-Mix Asphalt

IRI – International Roughness Index

ISTEA – Intermodal Surface Transportation Efficiency Act

LADOTD – Louisiana Department of Transportation and Development

LC – Longitudinal Crack

LCCA – Life Cycle Cost Analysis

LDF – Lane Distribution Factor

LOS – Level of Service

LRRB – Minnesota Local Road Research Board

LTPP – Long Term Pavement Performance

LWP – Left Wheel Path

MCA – Markov Chain Algorithm

MDOT – Michigan Department of Transportation

M-E PDG – Mechanistic-Empirical Pavement Design Guide

Mn/DOT – Minnesota Department of Transportation

MnROAD – Minnesota Road Research test facility

MPO – Metropolitan Planning Organization

MR – Resilient Modulus

MSU – Michigan State University

MTB – Modified Total Benefit

NAASRA – National Association of Australian State Road Authorities

NCHRP – National Cooperative Highway Research Program

NCPP - National Center for Pavement Preservation

NDT – Non-Destructive Deflection Test

NHI – National Highway Institute

NIMPAC – National Association of Australian Road Authorities Improved Model for Project Assessment and Costing

NPW – Net Present Worth

PC – Pavement Condition

PCC – Portland cement Concrete

PCR – Pavement Condition Rating

PJ – Performance Jump

PMS – Pavement Management System

PRA – Public Roads Administration

PSI – Pavement Serviceability Index

PSL – Pavement Service Life

QA – Quality Assurance

QC – Quality Control

RD – Rut Depth

RIP-DATA – Road Inventory Program Data Assurance Testing Application

RL – Remaining Life

RSL – Remaining Service Life

RWD – Rolling Wheel Deflectometer

RWP – Right Wheel Path

SA – Surface Age

SAFETEA-LU - Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users

SE – Standard Error

SHA – State highway agency

SHRP – Strategic Highway Research Program

SL – Segment Length

SLE – Service Life Extension

SN – Structural Number

SRID – State Route Identification

T^2M – Treatment Transition Matrix

TB – Total Benefit

TC – Transverse Crack

TEA-21 – Transportation Equity Act for the 21st Century

Th – Threshold Value

TL – Treatment Life

TRDF – Texas Research and Development Foundation

VCR – Volume-Capacity Ratio

VDOT – Virginia Department of Transportation

VETO – Vejstandard och Transportomkostninger

VOC – Vehicle Operating Cost

VOT – Value of Time

WSDOT – Washington State Department of Transportation

CHAPTER 1

INTRODUCTION & RESEARCH PLAN

1.1 Introduction

Since the early 1970's, several State Highway Agencies (SHAs) have developed elaborate pavement condition and distress data collection systems. The systems have evolved from walking along the road and recording the data on a specially designed sheet of paper, to wind-shield survey, and to automated systems (Flintsch et al. 2004, McGhee 2004). With advancements in data collection have come improvements in the data usage. Such improvements include modeling the time-series pavement condition and distress data for assessing pavement performance and for predicting future conditions. The main objective of assessing past and future pavement performance is the selection of the most cost-effective pavement treatment type, time, and project boundaries. To this end, the goal of this research study is to maximize the use of pavement condition and distress data and to further improve the benefits of pavement condition data collection.

1.2 Pavement Treatment Selection Background

Some SHAs in the USA select pavement preservation and rehabilitation activities based on their benefits and costs. Various pavement treatments are applied to improve the existing surface conditions and reduce distresses and their causes. The identification of the appropriate alternative pavement treatment types that would improve or reduce the surface conditions and distresses and their causes should be based on:

1. The type, severity, and extent of the pavement condition or distress (Labi et al. 2005).
2. Results of the forensic investigation of the pavement section. For each pavement project, the selected or the preferred pavement treatment types should correct most, if not all, the

pavement conditions and distresses and their causes (Baladi et al. 1999, Hicks et al. 2000, Peshkin et al. 2004).

The alternative pavement treatment types that were selected to correct or reduce the pavement conditions and distresses and their causes could be applied at varying times or conditions resulting in different costs and pavement performances (treatment benefits). In general, the benefits and costs of each selected treatment type are functions of the pavement conditions, distresses, and rates of deterioration at the treatment application time (Schuler & Schmidt 2009). Such time should be determined through cost-benefit analyses to maximize benefits and minimize the costs (Hand et al. 1999, Labi & Sinha 2003). Such analyses may yield diverse results for different SHAs because of the substantially different state-of-the-practice used in the pavement management system (PMS) data collection and analysis. Each SHA uses different procedures, terminology, and practice to collect and store the pavement conditions, distresses, costs, and other data. These non-standardized procedures and practices have led to many differences between SHAs including (Baladi et al. 2009):

- The continuity of the pavement condition and distress data, some states such as Colorado, Louisiana, Michigan, and Washington collect data on a continuous basis whereas others, such as Arizona and Mississippi, collect data based on sampling procedures.
- The frequency of data collection, some SHAs collect pavement condition and distress data every year, while others every other year, and still others every three or four years.
- Although most SHAs use linear location reference system, the make-up of the system varies substantially from one SHA to another and often within the same SHA. The errors associated with each system in identifying and locating the same pavement sections for each data collection cycle vary significantly.

- The type of data and the terminology used to label the data are not universal.
- Construction, rehabilitation, and preventive maintenance cost data are included and easily accessible in some SHA and very hard to impossible to retrieve in others.
- The accessibility of the basic inventory data (layer thicknesses and material types) is highly variable.
- For most pavement condition and distress types and severity levels, the assigned condition points, the condition indices, and the priority factors are not universal.

To account for these differences, the treatment selection guidelines to be developed in this study will be general in nature and their implementation will be progressive (step-by-step). Recommendations will be provided for data collection and analyses, while the step-by-step implementation procedure would allow the SHA to gradually adopt the guidelines as the required data are collected or transferred to the standard format.

1.3 Research Scope and Objectives

The scope of this research study is to determine the optimum set of pavement related data items that could be used to optimize and maximize the benefits of pavement management data collection and to determine the benefits of pavement treatments.

The specific objectives of this research study are:

- Determine the optimum set of data needed to comprehensively and systematically arrive at cost-effective pavement management decisions.
- Determine the impact of the current state-of-the-practice regarding the quality and accuracy of the time-series pavement condition and distress data.
- Review the advantages and shortcomings of existing methodologies and algorithms to express the benefits of pavement treatments, and recommend the most reliable methods.

- Analyze the state-of-the-practice to establish, for some treatment types, treatment transition matrices (T^2M) expressing the probability that the treatment causes gain, loss, or has no effect on the service life of the treated pavement segments. The matrices will be based on both the before treatment (BT) and the after treatment (AT) pavement conditions, distresses, and rates of deterioration.
- Provide implementation steps to be used by SHAs to estimate the effectiveness of pavement treatments based on the BT conditions, distresses, and rates of deterioration of planned pavement projects.
- Determine the most cost-effective pavement treatment type, time, and project boundaries based on the pavement conditions, distresses, and rates of deterioration.
- Develop methodology to estimate the pavement conditions and distresses of multiple-lane facilities based on the time-series conditions and distresses of the driving lane.
- Develop universal guidelines that can be implemented by SHAs to reliably determine the most cost-effective:
  a) Pavement treatment strategy at the network level within a given set of constraints (budget and political constraints).
  b) Treatment type, time, and project boundaries at the project level.

To accomplish the above objectives, a comprehensive research plan was drawn and is presented in the next section.

1.4 Research Plan

The general approach of this research consists of the four tasks shown in Figure 1.1. The tasks are detailed in the next few subsections.

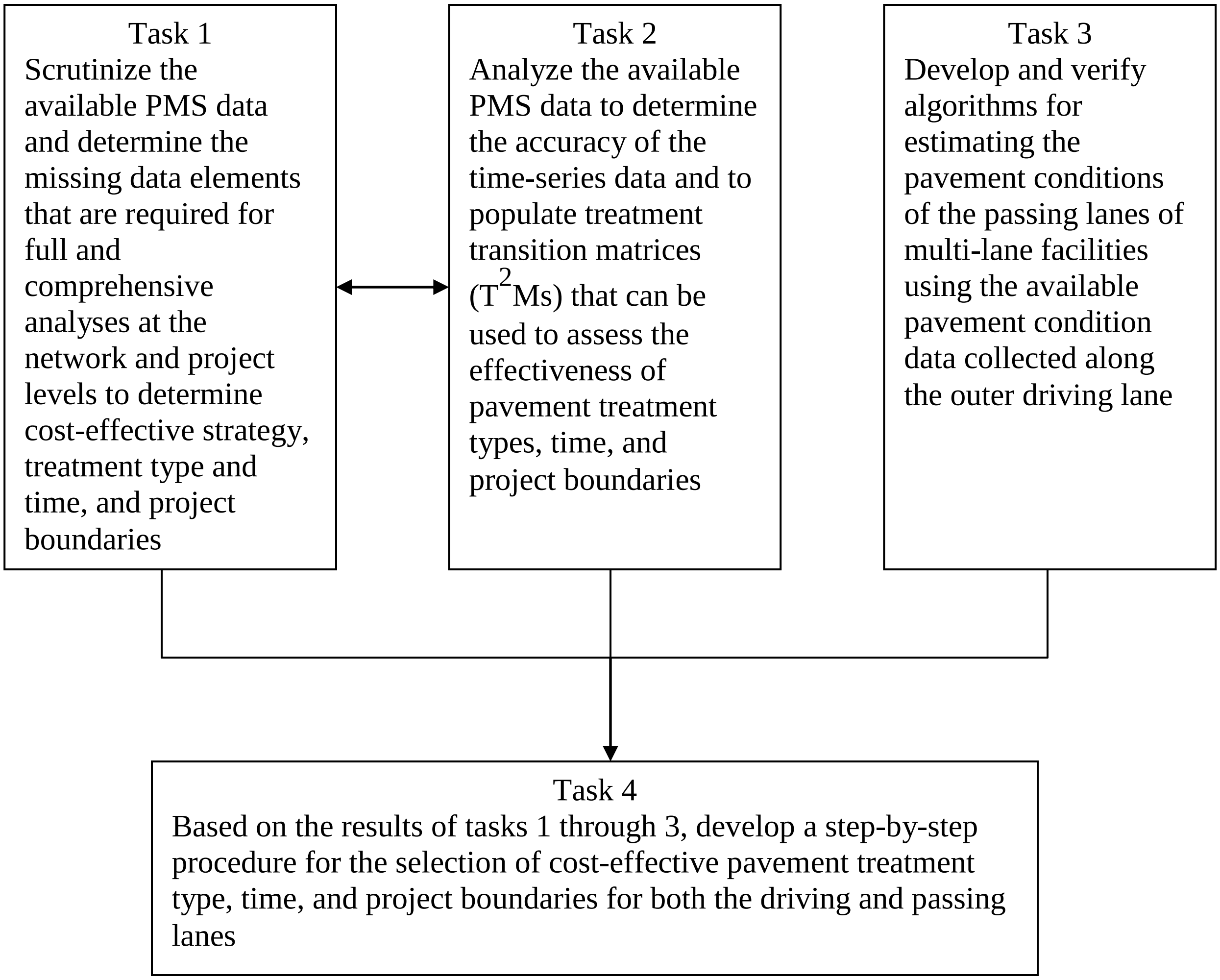


Figure 1.1 Flow chart and the four tasks of the research plan

1.4.1 Task 1 – Scrutinize the Available PMS Data and Determine the Missing Data Elements that are Required for Full and Comprehensive Analyses at the Network and Project Levels to Arrive at Cost-Effective Strategy, Treatment Type and Time, and Project Boundaries

Most SHAs obtain substantial amounts of information regarding the pavement network and the actions of the SHA. However, the mass availability of that information is sometimes limited and often varies from one SHA to the next. The data collected for this research were obtained from the PMS units of the Colorado Department of Transportation (CDOT), the

Louisiana Department of Transportation and Development (LADOTD), the Michigan Department of Transportation (MDOT), the Washington State Department of Transportation (WSDOT), the Minnesota Road Research (MnROAD) test facility, and the Long Term Pavement Performance (LTPP). The databases will be scrutinized for their comprehensiveness. It is envisioned that the following objectives will be achieved in this task:

- Identify the various data elements available in the databases.
- Identify any missing data elements in the databases.
- Populate a list of all of the required data elements for cost-effective management of pavement networks.
- Make recommendations for a comprehensive database.

1.4.2 Task 2 – Analyze the Available PMS Data to Determine the Accuracy of the Time-Series Data and to Populate Treatment Transition Matrices (T^2Ms) that Can be Used to Assess the Effectiveness of Pavement Treatment Types, Time, and Project Boundaries

The available time-series data will be analyzed to evaluate the accuracy of the collected pavement condition data and the location reference system. The data will also be analyzed to determine the effectiveness of pavement treatment type, timing, and project boundaries, and to populate T^2Ms. It is envisioned that the following objectives will be achieved in this task:

- Determine the accuracy of the pavement condition and distress survey segment boundaries and its affect on the remaining service life (RSL).
- Evaluate the various expressions of pavement treatment benefits.
- Determine the effectiveness of pavement treatment type, timing, and project boundaries for several pavement treatments.

- Populate T^2Ms for various pavement treatments.
- Compare different states-of-the-practices through their T^2Ms.
- Estimate the future conditions of a pavement network due to application of the treatment strategies.
- Make recommendations for collecting accurate time-series pavement condition and distress data and determining the effectiveness of pavement treatment type and timing.

1.4.3 Task 3 – Develop and Verify Algorithms for Estimating the Pavement Conditions of the Passing Lanes of Multi-Lane Facilities Using the Available Pavement Condition Data Collected Along the Outer Driving Lane

Most Interstate Highways in the United States consist of two lanes in each direction (driving and passing lanes) and a small percent have more than two lanes. In the past, most SHAs have designed pavement treatments based on the conditions of the driving lane and simply extended that treatment to the passing lane(s). Recently, many SHAs started treating the driving and passing lanes at different times. However, the state-of-the-practice of almost all SHAs is to collect data in the driving lane only, and therefore the available data must be analyzed to estimate the conditions and distresses of the passing lanes. The MnROAD pavement condition and distress data for the driving and passing lanes will be utilized to address this need. It is envisioned that the following objectives will be achieved in this task:

- Develop and verify algorithms to estimate the pavement conditions and distresses of the passing lane(s) based on the measured driving lane data.
- Make recommendations to estimate the pavement conditions and distresses of the passing lane(s) without requiring significant additional data collection.

1.4.4 Task 4 – Based on the Results of Tasks 1 through 3, Develop a Step-By-Step Procedure for the Selection of Cost-Effective Pavement Treatment Type, Time, and Project Boundaries for Both the Driving and Passing Lanes

In this task, a step-by-step procedure for the selection of cost-effective pavement treatment type, time, and project boundaries will be developed based on the results of the analyses in Tasks 2 and 3. The procedure will serve as a set of guidelines applicable to any SHA to incorporate the findings and recommendations of this study. It is envisioned that the following objectives will be achieved in this task:

- Address the process of selecting cost-effective pavement treatment type, time, and project boundaries to better manage pavement networks.
- Provide methodology to implement the tools, analyses, and evaluations of this research.
- Make recommendations based on the findings of this research.

1.5 Dissertation Layout

This dissertation consists of five chapters and two appendices as listed below.

Chapter 1 – Introduction & Research Plan

Chapter 2 – Literature Review

Chapter 3 – Data Mining

Chapter 4 – Data Analysis & Discussion

Chapter 5 – Implementation Procedures

Chapter 6 – Summary, Conclusions, & Recommendations

Appendix A – PMS Data

Appendix B – Analyzed Data

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The highway systems in the United States and in most of the industrialized world represent the single largest investment ever made in the history of these nations. Modern highway systems are a necessity that a nation must utilize in order to thrive or advance. In the developing world, the lack of modern and efficient transportation systems to move people and goods represents a major obstacle in their development. Without efficient and modern transportation systems, farm products spoil in fields and industrial goods remain in the factories (Baladi et al. 1992).

Unfortunately, as is the case with any natural or manufactured products, highway systems do not last forever. They deteriorate and disintegrate at an accelerated rate unless they are properly and continually maintained, rehabilitated, redesigned, and reconstructed. An annual flow of money is required to keep highway pavements operating in safe and acceptable conditions. This annual money requirement, however small, is not available to the highway administrators. Lack of funds, coupled with a high public demand to repair the highway systems, makes the administrators job a difficult one. The lack of appropriate funds is further compounded by several other factors including (RTAC 1977, FHWA 1978, Haas & Hudson 1982, CTR 1984, ERES 1987, Proceedings 1987, Baladi et al. 1992):

1. The variability of the engineering and physical properties of the materials that make up the highway systems.
2. The number of available rehabilitation and preservation alternatives to repair and restore the highway systems and their associated costs.

3. The number of bridges, highway miles, and the different highway classes and pavement types that are in need of repair.
4. The increasing traffic volumes and loads.
5. The dwindling supply of funding and natural resources such as aggregates, sand, and asphalt binders.

Given the above factors, the transportation infrastructure problem cannot be solved efficiently unless all the alternatives are analyzed, engineered, and compared to provide effective solutions. The engineering of the solution requires that accurate and up to date highway system data be collected and maintained to analyze and compare the various available alternatives. The required data could be divided into several categories including:

1. Inventory data including the as built pavement layer thicknesses, the location reference system, and environmental data.
2. All previous pavement maintenance, preservation, rehabilitation, and re-construction data and their associated costs. The data include: location, date and type of action, materials, and all special provisions.
3. Traffic volume and load data.
4. Pavement condition and distress data including ride quality, skid resistance, rut depth, and pavement surface conditions.
5. Drainage data including longitudinal and cross-drains, ditches, drainage outlets, and the hydraulic conductivity of the pavement materials.
6. The engineering properties and strength of all pavement materials and the roadbed soil and their components.
7. Feedback data concerning the pavement performance and behavior of past rehabilitation,

preservation, and maintenance activities.

8. Results of all forensic investigations that were conducted to determine the causes of pavement conditions and distresses and accidents.
9. Cost data including material and labor costs.
10. Results of all research studies and analyses that were conducted relative to the operation, safety, performance, behavior, and longevity of the highway systems.

For most State Highway Agencies (SHAs), the majority, if not all, of the data categories listed above are routinely collected by different groups, divisions, or offices of the agencies. However, because of different constraints (turf problems and the lack of a common reference location system) the data are not shared or are not made easily accessible to every person within the agency. Hence, it is crucially important that SHAs establish comprehensive, effective, and transparent cooperation and intra communication processes. These processes would assist the SHAs to establish systematic, unified, and cost-effective approaches to preserve the highway systems. The collection of these processes coupled with the actions taken and the decisions made by each person within the SHA make the pavement management system (PMS). For the benefits of the reader, the various terminologies used in this literature review and the remaining chapters of this dissertation are presented below.

2.2 Terminologies

Unfortunately, no universal definitions of pavement management terms have been accepted. Various definitions can be found throughout the literature and are being used by local, state, and federal highway authorities. For clarification, the definitions of pavement related terms used throughout this document are divided into five categories and are presented below.

2.2.1 Definitions of Pavement Structure, Type, and Design Terms

The following terms are defined after (Baladi et al. 1992, AASHTO 1993).

- **Pavement Structure** - The pavement structure is a combination of subbase, base, and surface layers placed on a subgrade to support the traffic load and distribute it to the roadbed soil. Hence, the term pavement structure does not include roadbed soils. The term pavement, on the other hand, consists of the following components:
    - **Subgrade** – The top surface of a roadbed upon which the pavement structure and shoulders are constructed. Hence, the term subgrade includes no material.
    - **Roadbed Soil** - The roadbed soil is the graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder. Roadbed material includes all material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.
    - **Subbase -** The subbase is the layer or layers of specified or selected material of designed thickness placed on the subgrade to support the base layer (or in the case of rigid pavements, the Portland cement concrete (PCC) slab).
    - **Base -** The base is the layer or layers of specified or selected material of designed thickness placed on the subbase or the subgrade to support a surface layer.
    - **Surface Layer** – The surface layer consists of one or more courses of a pavement structure designed to accommodate the traffic load, the top course of which resists skidding, traffic abrasion, and the disintegrating effects of climate. In flexible pavements, the top course is typically called the wearing course.
- **Rigid Pavement** – A pavement structure that distributes the traffic loads to the subgrade, where the surface layer consists of PCC slabs of relatively high bending resistance.

- **Flexible Pavement -** A pavement structure which maintains contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.
- **Composite Pavement -** A pavement structure composed of an asphalt concrete wearing surface and PCC slab; a PCC overlay on a flexible pavement is also referred to as a composite pavement.
- **Design Life -** The design life or more accurately, the design service life (DSL) is an estimate of the number of years of service (after construction or rehabilitation) for the pavement to accumulate the predetermined threshold value of distress points or condition. This estimate is typically a function of the design procedure and the condition prediction models used by the SHA.
- **Pavement Service Life or Performance Period -** The pavement service life (PSL) or performance period is the number of years a pavement performed within the specified limits of serviceability levels or conditions. Hence, the PSL could be shorter or longer than the DSL.
- **Pavement Life -** Pavement life is the actual number of years in service, starting from the year of construction to the year of reconstruction. Hence, the pavement life is the sum of all PSL periods between the original construction and total reconstruction or rehabilitation. For a newly constructed or rehabilitated pavement section, the pavement life is assumed to be equal to the design life. The actual pavement life could be shorter (the threshold value of condition or distress is reached in a shorter time period than the expected design life) or longer (the threshold value of condition or distress is reached in a longer time period than the expected design life) than the design life.

2.2.2 Definitions of Pavement Condition, Distress, and Performance Terms

The following terms are defined after (Baladi et al. 1992).

- **Pavement Distress** – Pavement distress is the physical manifestations of defects in a pavement.
- **Severity Level** – The severity level describes distinct categories of the progression of the distress type. Most distress types can take on a variety of severity levels. The most commonly used severity levels are low, medium, and high.
- **Extent** - The extent is a measure of the quantity or amount of each pavement distress type and its severity level.
- **Pavement Serviceability -** Pavement serviceability is a measure of the ability, at time of observation, of a pavement to serve traffic (automobiles and trucks) that uses the facility.
- **Pavement Condition** – The pavement condition is a quantitative representation of quality or distress in pavement at a given point in time.
- **Pavement Performance -** Pavement performance is the assessment of how well the pavement serves the user over time.
- **Threshold Value** – A threshold value indicates the pavement condition, distress index value, or the level of distress at which the pavement is rendered in need of repair.

2.2.3 Definitions of Pavement Performance Modeling Terms

The following terms are defined after (AASHTO 1990, Dawson et al. 2011).

- **Model Form** – The term model form implies the functional algorithm, such as exponential, power, logistic, etc…, that can be used to express the pavement performance over time.
- **Pavement Condition Modeling** – Pavement condition modeling involves the use of a specific model form (exponential, power, logistic, etc…) to model a given set of time-series data, such as pavement condition over time. The model form and its parameters could be used for the prediction of future pavement conditions. Pavement modeling could be

accomplished at the network level based on average data or at the project level based on detailed condition data.

- **Network Level -** Network level is the level at which the pavement conditions and/or performance are summarized to support key administrative decisions that affect programs for road networks or systems.
- **Project Level -** Project level is the level at which the pavement conditions and/or performance are detailed to support design or technical/engineering management decisions for specific projects along pavement sections.
- **Before Treatment (BT) –** BT refers to the pavement condition and distress data or the time between the pavement treatment in question and the previous treatment, reconstruction, or original construction.
- **After Treatment (AT) –** AT refers to the pavement condition and distress data or the time immediately after application of the treatment to the next treatment or reconstruction.

2.2.4 Definitions of Pavement Management Terms

- **Management** – Management is the act of managing, conducting or supervising, directing, and controlling the affairs of an organization or system. To manage is to directly handle and manipulate or maneuver toward desired objectives. To conduct or to supervise is to take responsibility for the acts and achievements of a group or groups. To direct is to constantly guide and regulate to achieve smooth operation. To control is to regulate and restrain to maintain course within the bounds of the objectives (Webster 2010).
- **System -** A system is a regularly interacting or interdependent group of items forming a unified whole. A system implies an efficient and fully developed or carefully formulated method(s) often emphasizing the idea of rational orderliness (Webster 2010).

- **Pavement Management** – Pavement management in its broadest sense encompasses all the activities involved in the planning, financing, designing, constructing, maintaining, evaluating, and rehabilitating the pavement portion of a public works program (Baladi et al. 1992).
- **Pavement Management System (PMS)** – Pavement management system (PMS) is a set of tools or methods that assist decision-makers in finding the optimum strategies for providing, evaluating, and maintaining pavements in a serviceable condition over a given period of time (AASHTO 1993).

  Other definitions of PMS include:

  - The pavement functions considered in an integrated, coordinated manner (Haas 1978).
  - The process of coordinating and controlling a comprehensive set of activities in order to maintain pavements, so as to make the best possible use of resources available, i.e. maximize the benefit for society (OECD 1987).
  - An established, documented procedure treating the pavement management activities in a systematic, coordinated manner (AASHTO 1985).
- **Managing Pavements** – Managing pavement consists of the various acts of planning, designing, constructing, maintaining, evaluating, and rehabilitating pavements (after Baladi et al. 1992).
- **Optimum –** The term optimum implies the maximization of an output or outputs relative to one or more objective functions (AASHTO 1990). For example, two of the objective functions of the optimization of pavement treatment strategy (an output) are minimizing users and agency costs and maximizing users and agency benefits (such as providing smoother pavement and maximum longevity).

- **Pavement Surface Age –** Pavement surface age is defined as the age in years of the pavement surface since construction or the last preservation action (Baladi et al. 1992).
- **Remaining Service Life (RSL)** – RSL is the estimated number of years, from any given year (usually from the last condition survey year), to the date when the conditions of the pavement section reach a pre-specified threshold value. As stated in Equation 2.1 (Baladi et al. 1992).

$$RSL = \{t(PC = Th) - SA\} < DSL \qquad \text{Equation 2.1}$$

Where, $t(PC = Th)$ is the time at which the pavement condition reaches the threshold value;

SA is the pavement surface age in years;

DSL is the pavement design service life in years

The RSL of a given pavement network can be calculated as the weighted average RSL of the "n" pavement segments within the network using Equation 2.2. It should be noted that any pavement segment that falls below the threshold value has a zero RSL. In general, no negative RSL should be assigned to any pavement regardless of its condition. For a newly designed and constructed or rehabilitated pavement segment, its RSL is equal to the design life (Baladi et al. 1992).

$$RSL_{(network)} = \frac{\sum_{i=1}^{n}(RSL_i)(SL_i)}{\sum_{i=1}^{n} SL_i}$$

Equation 2.2

Where, i is the "i^{th}" pavement segment;

n is the total number of pavement segments in the network;

RSL is the remaining service life;

SL is the segment length

- **Service Life Extension (SLE)** – SLE is the gain in service life resulting from a pavement treatment, as shown in Figure 2.1 (Dawson et al. 2011).

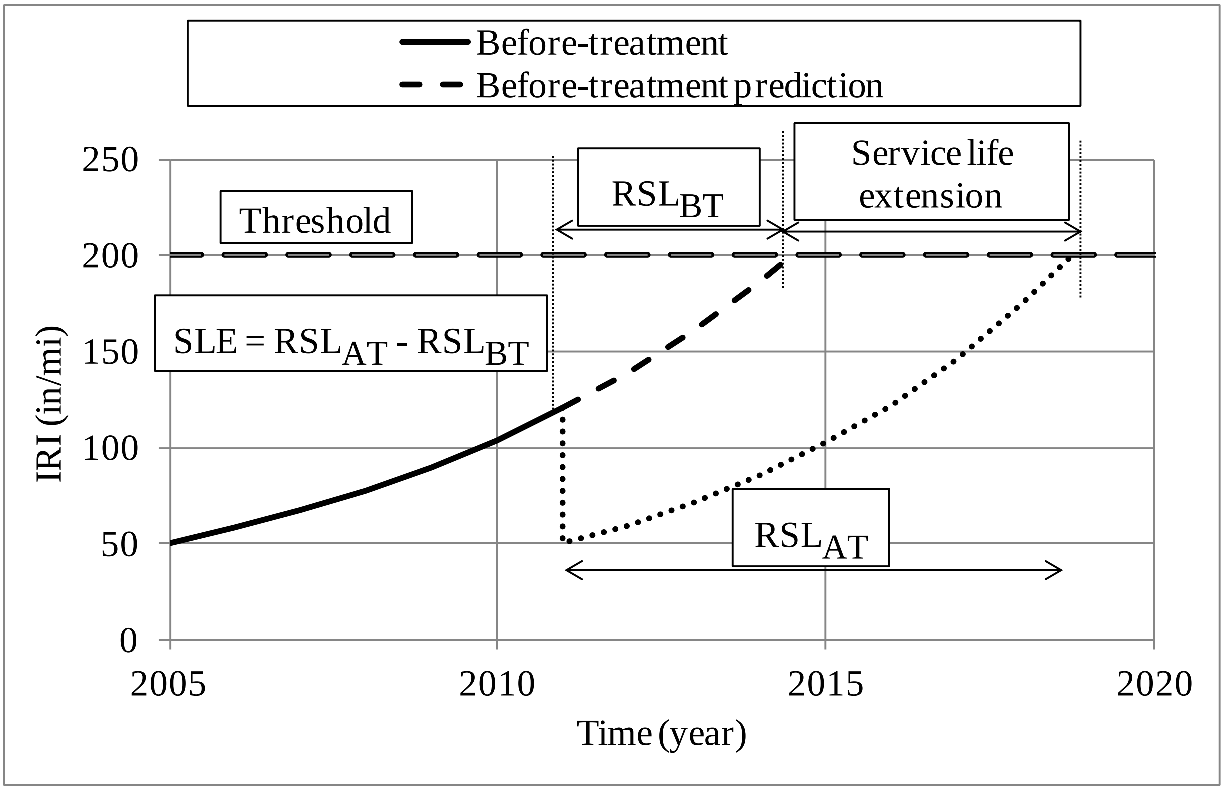


Figure 2.1 Schematic of the definition of SLE

- **Treatment Life (TL)** – TL is the time between the treatment date and the date when the pavement conditions or distresses reach the lesser of the threshold value or the BT pavement condition or distress, as shown in Figure 2.2. The TL is bound by the treatment life limit (TLL) (the average reported life of the treatment, see Table 2.11). In the case of worse pavement condition or distress AT, the TL is taken as the negative of the time for the BT conditions or distresses to reach the AT conditions, as shown in Figure 2.3 (Dawson et al. 2011).

- **Agency Costs** – Agency costs include all costs incurred directly by the agency over the life of the project (Walls & Smith 1998).

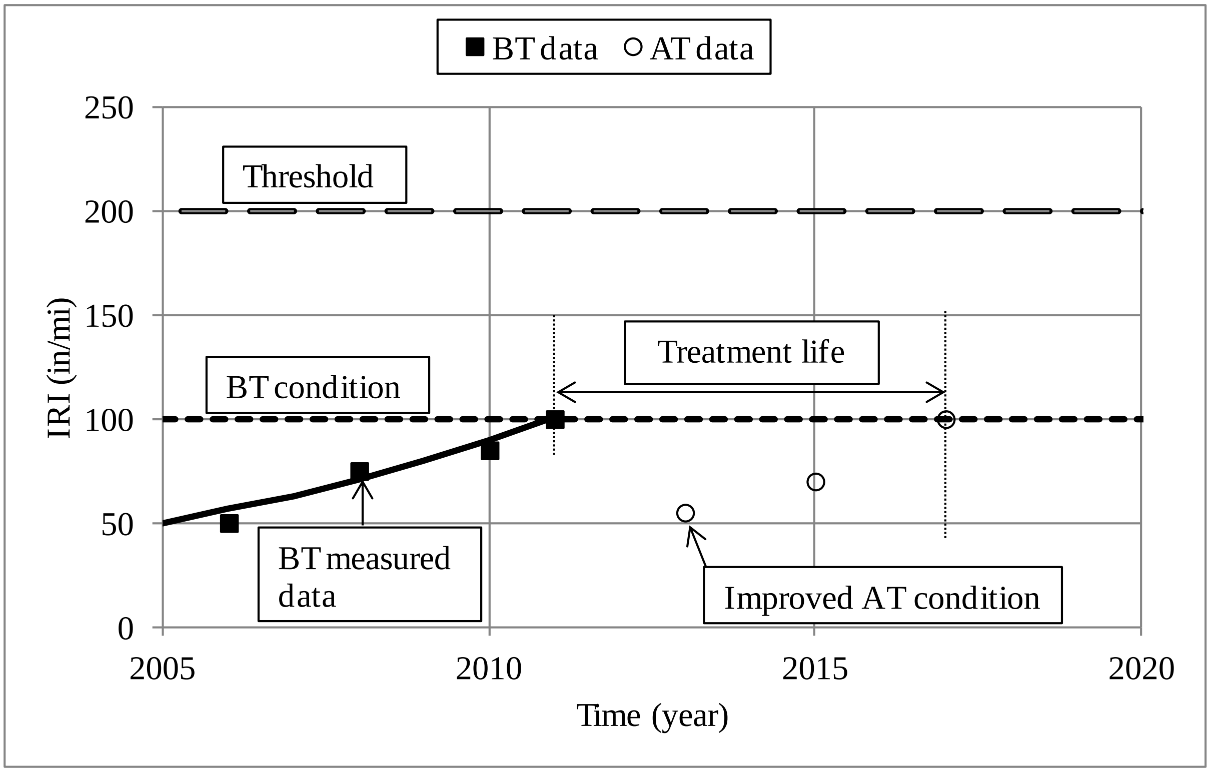


Figure 2.2 Schematic of the definition of TL

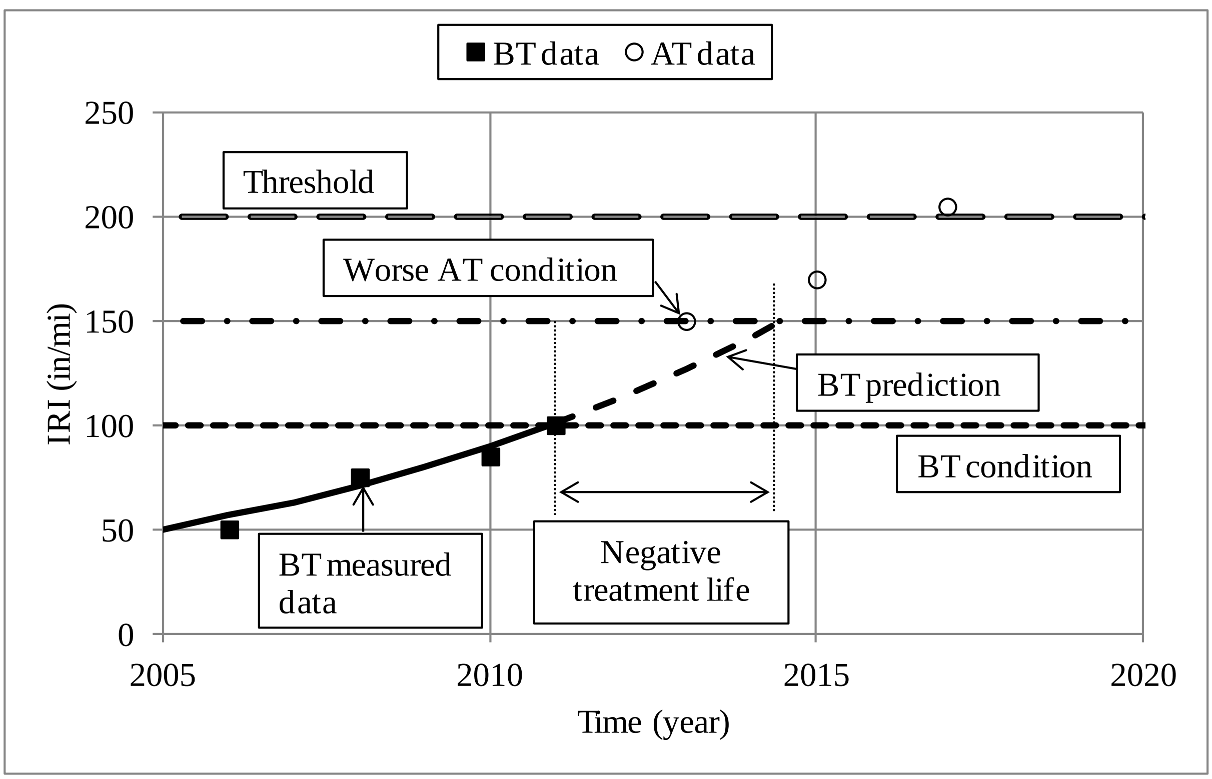


Figure 2.3 Schematic of the definition of negative TL

- **Agency Benefits** – Agency benefits are the increase in the longevity of the pavement segment or network (after Dawson et al. 2011).
- **User Costs** – User costs include delay, vehicle operating, and crash cost incurred by the users of a facility (Walls & Smith 1998).
- **User Benefits** – User benefits include reduction in user cost while providing the specified level of service (after Walls & Smith 1998).

2.2.5 Definitions of Pavement Preservation Terms

The Federal Highway Administration (FHWA) defines the following pavement preservation terms (Geiger 2005):

- **Pavement Preservation** – Pavement preservation is a program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extend pavement life, improve safety, and meet motorist expectations.
- **Routine Maintenance** – Routine maintenance consists of works that are planned and performed on a routine basis to maintain and preserve the condition of the highway system or to respond to specific conditions and events that restore the highway system to an adequate level of service.
    - **Corrective Maintenance –** Corrective maintenance activities are performed in response to the development of a deficiency or deficiencies that negatively impact the safe, efficient operations of the facility and future integrity of the pavement segment. Corrective maintenance activities are generally reactive, not proactive, and performed to restore a pavement to an acceptable level of service due to unforeseen conditions.

    - **Catastrophic Maintenance –** Catastrophic maintenance describes work activities generally necessary to return a roadway facility back to a minimum level of service while a permanent restoration is being designed and scheduled.
- **Preventive Maintenance** – Preventive maintenance is a planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without significantly increasing the structural capacity.
- **Pavement Rehabilitation** – Pavement rehabilitation consists of structural enhancements that extend the service life of an existing pavement and/or improve its load carrying capacity. Rehabilitation techniques include restoration treatments and structural overlays.
- **Pavement Reconstruction** – Pavement reconstruction is the replacement of the entire existing pavement structure by the placing of an equivalent or higher capacity pavement structure.
- **Thin Treatment** – Thin pavement treatments include non-structural hot-mix asphalt (HMA) overlay and mill and fill actions of less than 2.5-inch thickness (Baladi et al. 2011).
- **Thick Treatment** – Thick pavement treatments include structural HMA overlay and mill and fill actions of 2.5-inch thickness or greater (Baladi et al. 2011).

2.3 Historical Development of Roads

Early civilizations did not recognize the need for transportation routes or trails because the populations consisted of individual hunters who traveled (with no specific destination) following their prey. Around 30,000 B.C. families started to settle in small communities. When trades between people of two settlements emerged, narrow transportation trails (2 to 3 feet wide) connecting the settlements were established. After the invention of the wheel around 4,500 B.C., these dirt trails were widened to allow the passage of a vehicle (Yoder 1959, Carey & Irick 1960, Hazard 1977, Oglesby & Hicks 1982, Wright & Paquette 1987, and Baladi et al. 1992). The roots of the extensive highway systems of modern civilizations originate in the antedate of recorded history. The first hard surface road was built in the Mesopotamia around 3,500 B.C. The ancient civilizations came to realize the importance of roads and transportation systems (Carey & Irick 1960). The Egyptians built great roads to aid in the construction of the pyramids, and Babylon paved its streets. Later, Egypt was connected to Babylon and Rome via a stone surfaced road constructed in three layers. Remnants of this road can be found today throughout Syria, Israel, Iraq, and the rest of the Middle East countries (Baladi et al. 1992).

2.3.1 Ancient Engineered Roads

The Roman Empire constructed the first engineered highway system to aid in its conquest of the ancient world and to move goods and trades. The Roman road system consisted of three layers built in an excavated trench. After excavation, the bottom of the trench (the dredge line or the roadbed soil) was compacted by slave and animal labor. The first layer (subbase) consisted of compacted, hand-broken stones. The second layer (base course) was made of compacted small stones that were stabilized with mortar. The wearing course was made of massive stone blocks that were set and embedded in mortar. The Roman roads were geometrically designed to accommodate horses, two narrow vehicles (chariots), or one large piece of military equipment, traveling at a speed of about 5 to 10 mile per hour (mph). Structurally, the three layer system, of the roads, allowed spreading of the applied load to the roadbed soil and enhanced vertical drainage from the surface to the roadbed soil (most roads were located in dry freeze and dry non-freeze areas). This pavement design and construction technique became standard throughout Europe and the rest of the Roman

Empire until the late 18th century (Oglesby & Hicks 1982, Wright & Paquette 1987, Baladi et al. 1992).

2.3.2 European Development

After the split of the Roman Empire, road building virtually ceased and in some instances regressed, for a period of about 1,000 years. During this period, the Arab Empire rose to power and conquered portions of the Roman Empire such as today Jordan, Iraq, and Syria. The Arab Empire produced great advances in building architectures, sciences, and arts. Road building, however, was not a high priority item. The main reason is that the empire inherited the well developed Roman road system. The art of road building was revived in Europe during the eighteenth century. In France, Pierre Tresaguet (1716-1796) advocated a modified road construction method using smaller surface stones to replace the massive stone blocks used by the Romans. He was the first road builder to acknowledge the importance of drainage and routine maintenance upon pavement performance. During the reign of Napoleon (1800-1814), high priority road construction for military purposes was established, creating a great system of French roads. In England, Macadam (1756-1836) was the first road builder to design roads (which bear his name) based on two principles; drainage and compacted subgrade to support the applied loads (Oglesby & Hicks 1982, Wright & Paquette 1987, and Baladi et al. 1992). Furthermore, England's road improvements introduced the concepts of the right of way and the rights of eminent domain. Individual property owners, however, were still responsible for routine maintenance activities, such as draining and clearing the roads from vegetation and debris (Wright & Paquette 1987, Baladi et al. 1992).

2.3.3 Early American Roads

Transportation systems in the early history of the United States were limited to waterways since most settlements were located in bay areas or adjacent to rivers. The relatively few inland settlements were connected to shore or rivers by cleared pathways through the forests. After the Revolutionary War and the aftermath of the Whiskey Rebellion, the first turnpike toll road was built in Pennsylvania between Philadelphia and Lancaster. Shortly after this opening, numerous other turnpikes were financed and built by the private sector. In 1806, the Congress authorized the first toll free road (Old National Pike) to connect Cumberland, Maryland, to Wheeling, West Virginia. This toll free road was later extended to St. Louis, Missouri. At about the same time, the country witnessed the development of the railroads. This development practically halted the extension of existing turnpikes and the development of newer ones (Oglesby & Hicks 1982, Wright & Paquette 1987, Baladi et al. 1992).

In the late 19th century, a strong popular demand led to state financed aid for highway construction. New Jersey was the first state to enact a highway aid law. Shortly after, other states enacted similar legislation (Baladi et al. 1992).

2.3.4 Modern U.S. Highway Systems and Programs

The demand for improved road surfaces, particularly in the cities, to control noise and dust brought several new types of road surfaces, such as brick, asphalt, and concrete surfaces. The first brick surfaced road was built in 1871 in Charleston, West Virginia. In 1867, Pennsylvania Avenue in Washington, D.C. became the first asphalt surfaced road. Michigan took the lead in introducing rigid pavement by building the first concrete road in Wayne County. Beginning in 1904, the great industrial revolution produced motor vehicles in considerable numbers. This caused a sharp increase in the demand for highway and rural road improvements, especially those connecting farms to cities (farms to market roads). Federal and state governments came to realize that road financing and construction was a matter for their concern. Thus, additional State Aid Laws were enacted. By the year 1920, most states had established a form of Highway authority (HA) responsible for construction and maintenance of major state roads (CRES 1984, ERES 1987, Baladi et al. 1992, Nostrand 1992).

At the federal level, the creation of the Office of Road Inquiry of the Department of Agriculture in 1903, inaugurated the involvement of the Federal Government in highway activities on a continuing basis. In 1918, the office became the Bureau of Public Roads (BPR) of the Department of Agriculture. The Great Depression and President Roosevelt's "New Deal" pushed for the creation of massive public works projects; one focus was on the creation of highways which crossed the entire nation. Under the reorganization of the Federal Government of 1939, the BPR became the Public Roads Administration (PRA), which was then transferred to the Federal Works Agency. During this period study and discussion of the funding, organization, spatial layout, and many other topics regarding the national highway system took place. Each state, local, and federal HA had a separate set of ideas, traditions, and practices. There was a need for a centralized authority to set guidelines on highway design and management. Another reorganization of the government in 1949 renamed the PRA to become the BPR again and transferred the bureau to the Department of Commerce. In 1967, the BPR was again transferred to the newly formed Department of Transportation (DOT). Finally, in 1970, the BPR was again renamed to its present designation "the Federal Highway Administration (FHWA)" (Baladi et al. 1992, Nostrand 1992).

In the same time frame, the Federal Aid Highway Act of 1916 marked the beginning of the modern era of Federal Aid for highway activities by authorizing the expenditure of $75 million for rural highway improvements. In 1934, the Congress authorized the expenditure by the states of up to 1.5 percent of the Federal Aid Funds for highway planning and surveying. The 1938 Federal Aid Highway Act directed the chief of the BPR to study the feasibility of a six route network based on the statewide highway planning surveys. The 1944 Act provided $500 million per year for 3 years for improvement of the highway system and it established the Federal Aid Secondary System (Baladi et al. 1992).

The most important time period in the development of the national highway system was during President Eisenhower's administration in the 1950's. He recognized the military advantage of a highway system which could transport troops and supplies across the nation. In 1919, the first military transcontinental motor convoy took 62 days to travel from Washington D.C. to San Francisco; the young Dwight Eisenhower was on that convoy. During World War II, the effectiveness of the German Autobahn was seen by the allies and recognized as an important transportation system during wartime. Consequently, after the Korean War, President Eisenhower highlighted the importance of transcontinental highway system by stating "Together, the united forces of our communication and transportation systems are dynamic elements in the very name we bear – United States. Without them, we would be a mere alliance of many separate parts." Thus, the national highway system became the main objective of his administration and the modern funding procedure of gas taxes and federal/state cost sharing was developed during this time (Nostrand 1992). The 1952 Federal Aid Highway Act specifically authorized the expenditure of funds for the Interstate Highway System and directed the Secretary of Commerce to conduct studies of highway financing. Finally, in 1955, the President's Advisory Committee on the National Highway Program (the Clay Committee) reported the need for modernization of the Interstate System. This and other efforts led to the Federal Aid Highway Act of 1956, which is the most important act in the development of the current U.S. highway system. Prior to this act, Federal Aid appropriation for highways came from the general budget. In the 1956 act, proceeds from highway user taxes were earmarked and put into the newly created Highway Trust Fund. The 1956 act authorized the design and construction of the approximately 41,000-mile National Interstate and

Defense Highway System to be completed within a period of 15 years and made funds available to the states on a 90-10 matching basis. The act did not include matching funds for the maintenance and rehabilitation of the Interstate Systems. The Interstate and Defense Highway System was later extended to about 42,500 miles (CRES 1984, Baladi et al. 1992).

The Federal Aid Highway Act of 1976 initiated the 3R program (resurfacing, restoration, and rehabilitation), which later was expanded to include recycling (4R). The Surface Transportation and Uniform Relocation Assistance Act of 1987 represented another milestone in the development of the nation's transportation network. The act, under different titles, authorized funds for construction and preservation of highways, for highway safety, and for mass transportation programs. It also expanded and improved the relocation assistance program. Under title I, Section 128, the act established and authorized funding for the Strategic Highway Research Program (SHRP). Section 131 authorized the National Highway Institute (NHI) to set aside funds for the education and training of state and local highway authority employees (CRES 1984, Baladi et al. 1992).

The Federal Aid Highway Act of 1991 (Intermodal Surface Transportation Efficiency Act, ISTEA) transferred the center of attention of national transportation policy to increasing efficiency. States and Metropolitan Planning Organizations (MPOs) were given the ability to use state and local funds for the promotion of alternative transportation projects. Focus was placed on community involvement and promoting bicycling and walking. This act was enhanced by the 1998 act (Transportation Equity Act for the 21st Century, TEA-21) (Axelson et al. 1999). Then in 2005 the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) provided funding and framework to maintain and grow the transportation infrastructure. The act focused on challenges such as improving safety, reducing traffic congestion, improving efficiency in freight movement, increasing intermodal connectivity, protecting the environment, and providing a base for future challenges (FHWA 2005).

2.4 Pavement Management System – State-of-the-Practice

The National state-of-the-practice and the International state-of-the-practice in pavement management systems are reviewed in the next few sections.

2.4.1 Pavement Management vs. Managing Pavement

Managing pavement consists of the various acts of planning, designing, constructing, maintaining, evaluating, and rehabilitating pavements. While pavement management encompasses all the activities involved in the planning, financing, designing, constructing, maintaining, evaluating, and rehabilitating the pavement portion of a public works program. In other words, pavement management is the effective directing of the various actions involved in managing pavements. Further, the establishment of a documented procedure regarding all of the pavement management activities is referred to as a pavement management system (PMS). Pavement management and PMSs are comprised of the following elements which serve to assist decision-makers in managing the pavements in the most cost-effective manor (Baladi et al. 1992).

1. Pavement surveys related to serviceability and condition
2. Database containing all information related to pavements
3. Analysis scheme
4. Decision criteria
5. Implementation procedures

2.4.2 Historical PMS Perspective

When the first SHA was created, it consisted of few people who worked together to manage the roads within the given set of constraints. Over time, the responsibilities of the agency grew exponentially and so its size. Various offices, bureaus, or divisions (such as planning, design, construction, materials, and so forth) were then created within the agency to handle certain responsibilities. Turf problems developed between the various divisions that resulted in duplication of work and worsened communication. Hence, the management of the road systems became fragmented and activities were disconnected. Further, comprehensive management and rehabilitation of the Interstate system were not included in the earlier Federal Aid Highway Acts. Consequently, SHAs did not spend their dollars on maintaining or rehabilitating the Interstate network. In the late 1950's and early 1960's SHAs across the country were busy building highways at an unprecedented rate. Unfortunately, this was accomplished without detailed thought process regarding the management of the system. The philosophy at that time can be summarized as follows: "let the system deteriorate to reconstruction level so that federal matching funds will become available." Indeed, the concept of highway infrastructure management based on recorded condition data did not exist in the 1950's and early 1960's (Baladi et al 1992).

Transportation infrastructure asset management has evolved from undocumented procedures to elaborate and extensive data collection systems. The first recorded condition data were collected in the mid 1940's in the State of Michigan. Engineers walked along the road and recorded pavement conditions on a strip map. From the condition data, the "remaining life" of each pavement segment was calculated. Unfortunately, the data were kept on papers in the office where they were collected and were not made available to other offices within the SHA. The first pavement management related procedure was developed and implemented at the American Association of State Highway Officials (AASHO) Road Test. The collected condition data were later used to develop empirical equations for the design of asphalt and concrete pavements. The equations were later modified and became the foundations of the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (Carey & Irick 1960, Hutchinson & Haas 1969, and Yoder & Witczak 1975, Baladi 1979, Baladi et al. 1992).

In the 1970's, the need for the development of PMSs became the talk of most SHAs. The National Cooperative Highway Research Program (NCHRP) responded to this need by sponsoring project 20-7 "Pavement Management System Development." In 1980, the FHWA issued a Federal Highway Program Manual "FHPM 6-1-1-12" entitled "Pavement Management" encouraging SHAs to develop PMSs, which was later upgraded by FHPM 6-2-4-1 entitled "Pavement Design Policy". In 1982, the FHWA developed Demonstration Project No. 61, which was taken around the country to more than 40 states. In 1982, the NHI of the FHWA sponsored a 6-week course on pavement management (CTR 1984). The course was offered at the University of Texas, Austin. In 1988, the NHI sponsored a 4-week course on pavement condition, design, rehabilitation, and management. The materials for the latter course were developed and the 4-week course was offered six times at Michigan State University (Baladi et al. 1992, Baladi et al.1992, and Baladi et al. 2002). In 1988, the FHWA sponsored a short course on advanced PMSs, which was offered eight times around the country. In short, the activities at the federal level were numerous and aimed at encouraging the SHAs to establish their own PMSs (Baladi et al. 1992, Nostrand 1992).

In the 1980's some engineers advocated the development of various management systems such as maintenance management system, construction management system, pavement management system, bridge management system, road side management system, safety management system, and so forth. Although the data in such systems are helpful, they were developed independently. Consequently, the systems were disconnected and did not communicate with each other. Hence, data in the maintenance or construction divisions were not easily accessible to others. Even within one management system such as the PMS, some advocated the collection of two sets of condition data, one at the network level management and the other at the project level. This precipitated, in some cases, a two tier system that is disconnected and not compatible. The network level data were kept in computer data files or a database, while the project level data (the expensive and detailed data) were kept in the project file. Finally, in the late 1980's and early 1990's, some advocated the development and implementation of asset management system that houses the management of all components of the transportation infrastructure (Baladi et al. 1992).

The goals and objectives of the acts of managing pavements and the functions of the PMS depend on the SHA. In general, the goals of a successful PMS can be grouped into the three basic categories summarized below; program development, improving PMS operation, and feedback (Baladi et al. 1992).

1. Program development - One of the basic functions of a PMS is to aid the SHA in developing its annual and multi-year programs. This can be attained by:
    - Monitoring (keeping records) conditions of the entire pavement network prior to and after the applications of preservation and rehabilitation treatments.
    - Analyzing the monitoring data and comparing the results to the acceptable conditions.
    - Providing suitable information for economic (cost) and engineering analyses of all reasonable preservation and rehabilitation alternatives.
    - Estimating accurate funding levels required to keep or to improve the health of the network.
    - Demonstrating the effects of budget limitations on pavement conditions and performance.

2. Improved PMS operation - This goal can be attained if the PMS is structured to achieve the following objectives:
   - Establishing cost-effective management of the substantial transportation investment.
   - Enhancing communication between the different offices of the SHA.
   - Monitoring and updating, on a regular basis, the environmental, traffic load and volume, costs, and condition data.
   - Obtaining and analyzing all information necessary to support a sound decision-making process.
3. Feedback - One of the goals of an effective PMS is to establish a feedback system. The objectives of this goal are:
   - Providing and updating accurate information concerning the various activities (maintenance, construction, design, etc.) of the SHA.
   - Checking the accuracy and quality of engineering practices.
   - Improving design, maintenance, construction, and rehabilitation methods and practices to achieve cost-effective pavement performance.
   - Comparing predicted and measured pavement performance to improve the existing pavement performance prediction models.
   - Verifying the accuracy of traffic forecasting models.
   - Updating and modifying existing policies, specifications, and standards that would improve the cost-effectiveness of pavement preservation.

In addition, it has been said that a PMS "Cannot directly consider unquantifiable factors such as political factors; nor does it alone make decisions". This is true if and only if the PMS is limited to a computer program. In reality, a PMS encompasses all the people (managers, engineers, and technicians) of a SHA as well as a few computer programs. Thus, the people can handle the shortcoming of the computer programs. A PMS separated from the people is not truly a PMS. Nevertheless, the above objectives unfold around the central question of how to keep or improve the health of the network in a cost-effective manner. This can be achieved if the activities presented in the next subsections are comprehensively and systematically implemented as parts of the PMS (Baladi et al. 1992).

2.4.3 PMS State-of-the-Practice

Since the early 1970's, several SHAs have developed elaborate pavement condition data collection systems. The systems have evolved from walking along the road and recording the data on a specially designed sheet of paper, to wind-shield survey, and to automated videotaping system (Flintsch et al. 2004, McGhee 2004). Today however, huge differences between the SHA pavement management practices and their PMS can be found. These include (Baladi et al. 2009):

- The continuity of the pavement condition data, some states such as Colorado, Louisiana, Michigan, and Washington collect pavement condition data on a continuous basis whereas others, such as Arizona and Mississippi, collect data based on a sampling procedure.
- The frequency of data collection, some agencies collect pavement condition data every year, while others every other year, and still others every three or four years.
- Although all agencies use linear location reference system, the make-up of the system varies substantially from one agency to another and often within the same agency.
- The type of data and the terminology used to label the data are not universal.
- The type of data storage varies from data files to relational database.
- The use of the PMS data within the agency varies from intensive to not at all.

- The relationships between the collected condition data and the material and structural design processes are in different degrees of development.
- Construction, rehabilitation, and preventive maintenance cost data are included and easily accessible in some agencies and very hard to impossible to retrieve in others.
- The accessibility of the basic inventory data (layer thicknesses and material types) is highly variable.
- The condition indices used by the agencies to rate the various pavement segments are based on different scales and different algorithms.
- For most condition or distress types and severity levels, the assigned distress points and priority factors are not universal.
- The definitions of low, medium, and high severity distress vary from one agency to another.
- The distress levels at which the pavement is in need of repair (the threshold value) are not universal and they vary substantially from one SHA to another.
- The institutional issues regarding PMS development and implementation are highly variable.
- The location of the PMS office within the highway agency varies from planning, to design, to safety, to research, and to administration.
- Various data regarding the pavement network are collected by different offices within the agency. In most cases, the data collected by one office are not compatible to the data collected by other offices or are not accessible.
- Various management systems (maintenance, pavement, and road side management systems to name a few) were established without any common denominator between the systems.

Due to the variation in the way SHAs conduct their PMS, large differences exist between them relative to the degrees of development of their PMS and its implementation. Further, differences in the size and performance of their road networks can be found. These are summarized in Table 2.1 (Hartgen et al. 2009). For each state, the following data are provided in the table:

- The total number of miles under each state's jurisdiction.
- The total number of lane-miles under each state's jurisdiction.
- The ratio of lane-miles to miles, which is indicative of the weighted average number of lanes.
- The percent of each state's urban, rural, and rural arterial road networks considered by the state to be in poor condition.

The data in Table 2.1 indicate that, for the roads within the state's jurisdiction:

1. The number of road miles varies from only 939 miles in Hawaii to 79,975 miles in Texas. The national total road mileage under the jurisdiction of all SHA is 777,741 miles.
2. The number of lane-miles ranges from 2,442 in Hawaii to 192,345 in Texas. The national total lane-miles under the jurisdiction of all SHAs is 1,849,382 lane-miles.
3. The ratio of lane-miles to miles (indicating the weighted average number of lanes in the network), of all roads, varies from 2.06 in West Virginia to 3.65 in New Jersey, with a national average of 2.38 lanes.
4. The percent of each road class considered by the state to be in poor condition varies from one road type to another as follows:
    - Urban Interstate roads – 0.0 to 25.0 percent with a national average of 5.86 percent.
    - Rural Interstate roads – 0.0 to 16.32 percent, with a national average of 1.93 percent.
    - Rural arterial roads - 0.0 to 16.44 percent, with a national average of 0.64 percent.

Table 2.1 Summary of state road inventory and conditions (Hartgen et al. 2009)

State	Inventory			Poor condition (percent of road class)		
	Miles	Lane-miles	Lane-miles/ miles	Urban	Rural	Rural arterial
AK	5,651	11,698	2.07	1.47	6.03	16.44
AL	10,936	28,098	2.57	4.35	2.76	0.05
AR	16,439	37,025	2.25	7.14	3.91	0.45
AZ	6,785	18,752	2.76	0	0.31	0.43
CA	15,269	50,732	3.32	24.72	16.32	1.08
CO	9,092	22,912	2.52	5.22	2.48	0.91
CT	3,717	9,789	2.63	4.64	0	0.61
DE	5,309	11,642	2.19	5	N/A	0
FL	12,062	42,080	3.49	0.14	0	0
GA	17,914	47,273	2.64	0	0	0
HI	939	2,442	2.6	25	0	2.73
IA	8,887	22,974	2.59	9.21	1.59	1.6
ID	4,959	12,093	2.44	14.44	0.58	0.12
IL	16,058	41,977	2.61	4.8	0	1.03
IN	11,188	28,358	2.53	2.8	0	0.06
KS	10,369	23,997	2.31	0.51	0	0.07
KY	27,547	61,386	2.23	0.48	0	0.04
LA	16,681	38,458	2.31	7.26	7.37	2.24
MA	2,833	8,655	3.06	0.42	0	0
MD	5,150	14,675	2.85	8.3	0	0.23
ME	8,519	18,111	2.13	1.49	0	2.16
MI	9,673	27,503	2.84	7.41	2.3	0.12
MN	11,881	29,180	2.46	1.98	2.12	0.19
MO	33,685	75,471	2.24	2.4	0	0.32
MS	10,957	27,395	2.5	8.7	1.01	0.48
MT	10,785	24,469	2.27	3.28	0.35	0.04
NC	79,288	169,612	2.14	2.82	3.14	0.51
ND	7,384	16,987	2.3	0	0	0.82
NE	9,955	22,486	2.26	7.94	0	0.56
NH	3,990	8,857	2.22	9.09	0.67	1.93
NJ	2,327	8,504	3.65	17.76	6.15	0.79
NM	11,983	29,301	2.45	0	0	0.11
NV	5,383	13,058	2.43	0.84	0	0.14
NY	14,969	38,055	2.54	10.76	7.69	1.5
OH	19,266	48,970	2.54	1.77	0.55	0.2
OK	12,284	30,057	2.45	10.84	1.02	2.55
OR	7,536	18,266	2.42	0	0	0.4
PA	39,871	88,445	2.22	2.08	0.47	0.47
RI	1,105	2,908	2.63	0	0	10.2

Table 2.1 (Cont'd)

State	Inventory			Poor condition (percent of road class)		
	Miles	Lane-miles	Lane-miles/ miles	Urban	Rural	Rural arterial
SC	41,437	89,861	2.17	0.76	0.17	0.16
SD	7,843	18,071	2.3	5.26	0	1.38
TN	13,886	36,420	2.62	1.2	0.29	0.32
TX	79,975	192,345	2.41	5.07	0.43	0.07
UT	5,831	15,188	2.6	1.89	0.97	0.61
VA	57,727	124,891	2.16	2.92	0	0.07
VT	2,633	6,043	2.3	17.07	2.87	1.56
WA	7,044	18,392	2.61	3.72	2.36	0.66
WI	11,769	29,417	2.5	5.34	3.14	0.28
WV	34,217	70,491	2.06	3.39	1.6	0.86
WY	6,753	15,612	2.31	5.15	1.35	0.05
Avg.	777,741	1,849,382	2.49	5.34	1.93	1.15

Over the last few years period, the staff of the National Center for Pavement Preservation (NCPP), which is located on the campus of Michigan State University, conducted survey of 43 SHAs regarding their PMS practices. The survey data that are related to this study were obtained from the NCPP office and are summarized in Figures 2.4 through 2.13 below. A summary of the findings is enumerated below (Galehouse 2010).

1. The terminology used by the SHAs to label pavement condition and distress data vary substantially from one agency to another.
2. For the same pavement type, some pavement condition data are collected by almost all SHAs. These include transverse and longitudinal cracks, and longitudinal and transverse profiles. Although other pavement condition or distress types (such as faulting, alligator cracking, and lane-shoulder drop-off) are also collected, they are not universal.
3. The types of pavement condition data collected by most SHAs are based on the most common condition or distress types found along the various pavement types. The methods

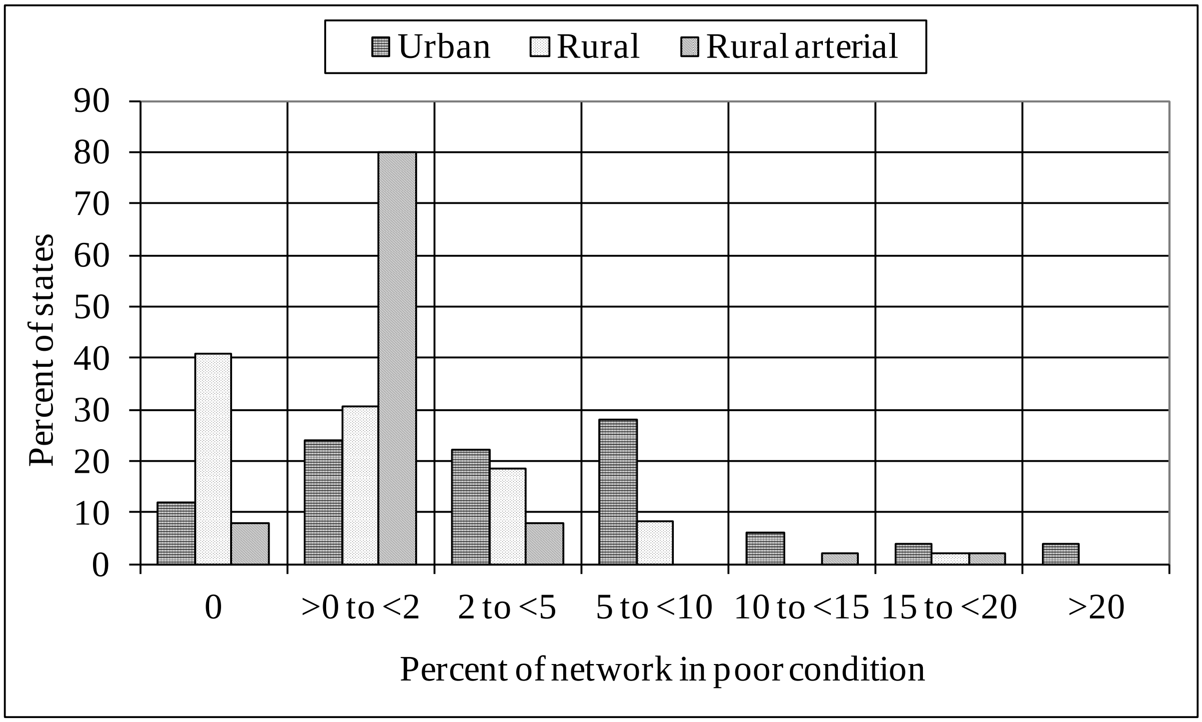


Figure 2.4 Percent of states versus the percent of network in poor condition

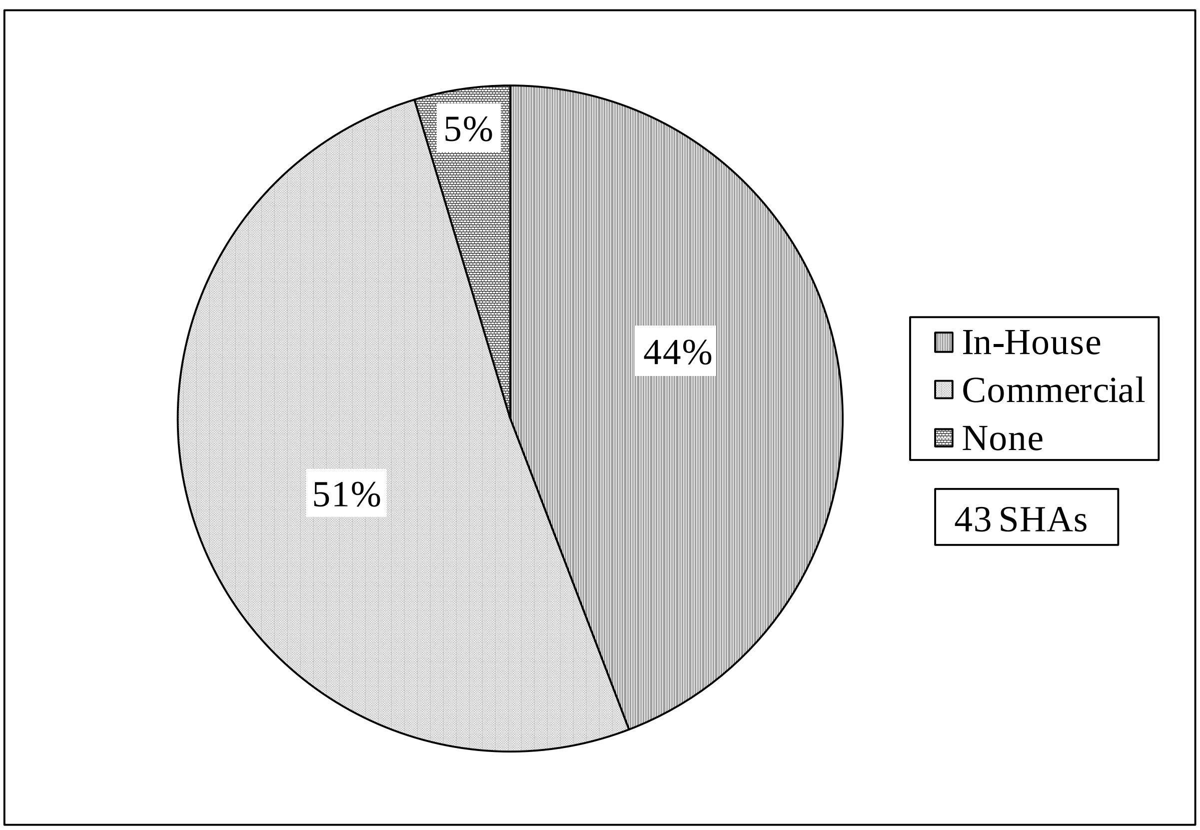


Figure 2.5 Percent of SHAs using in-house or commercial PMS computer program

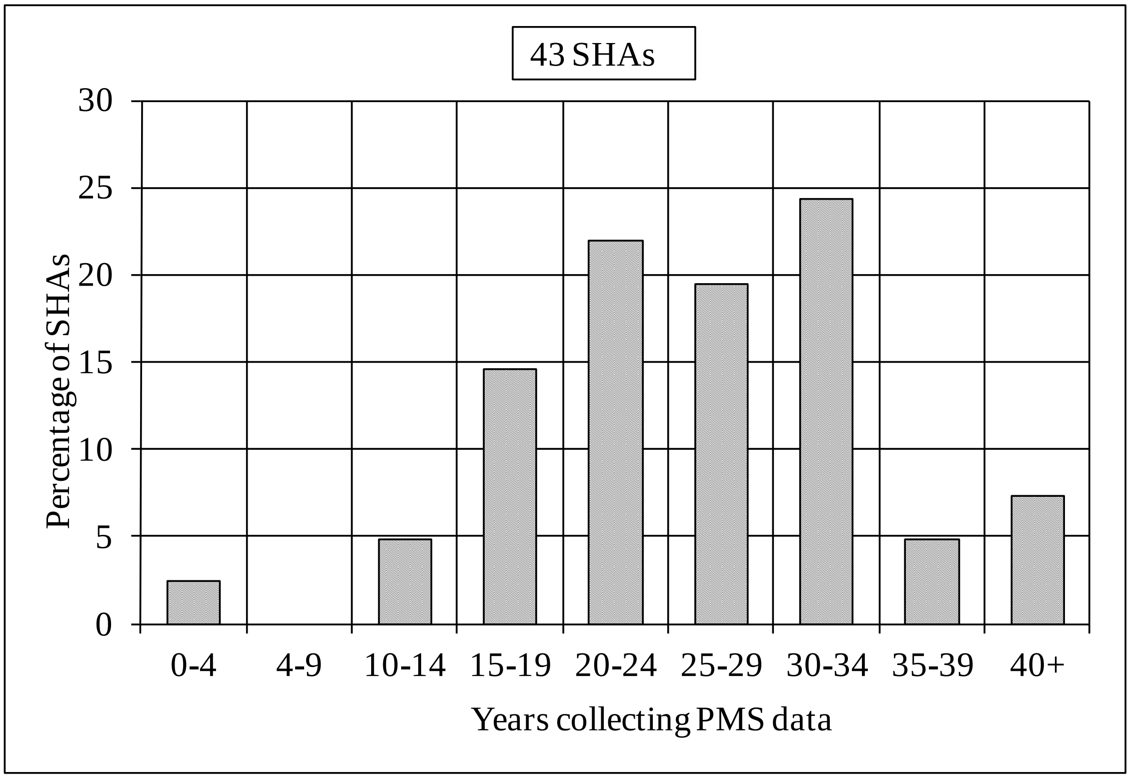


Figure 2.6 Percent of SHAs versus the number of years the PMS data have been collected

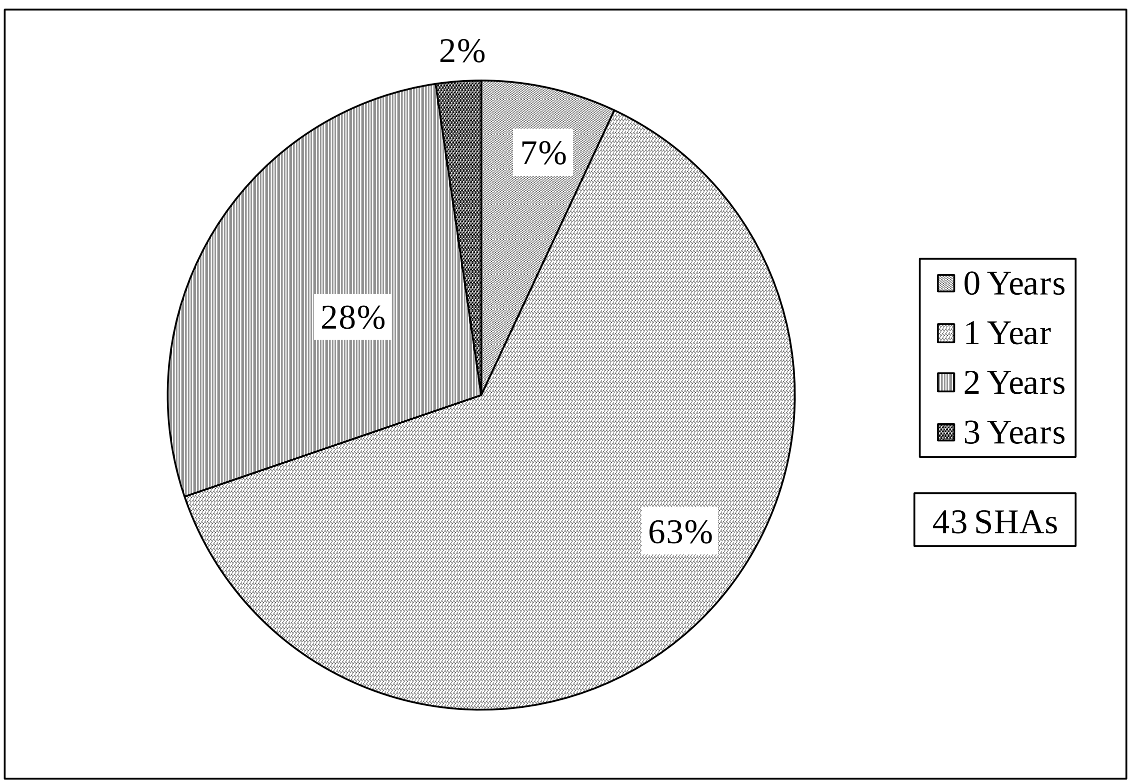


Figure 2.7 Percent of SHAs using the indicated PMS data collection frequency

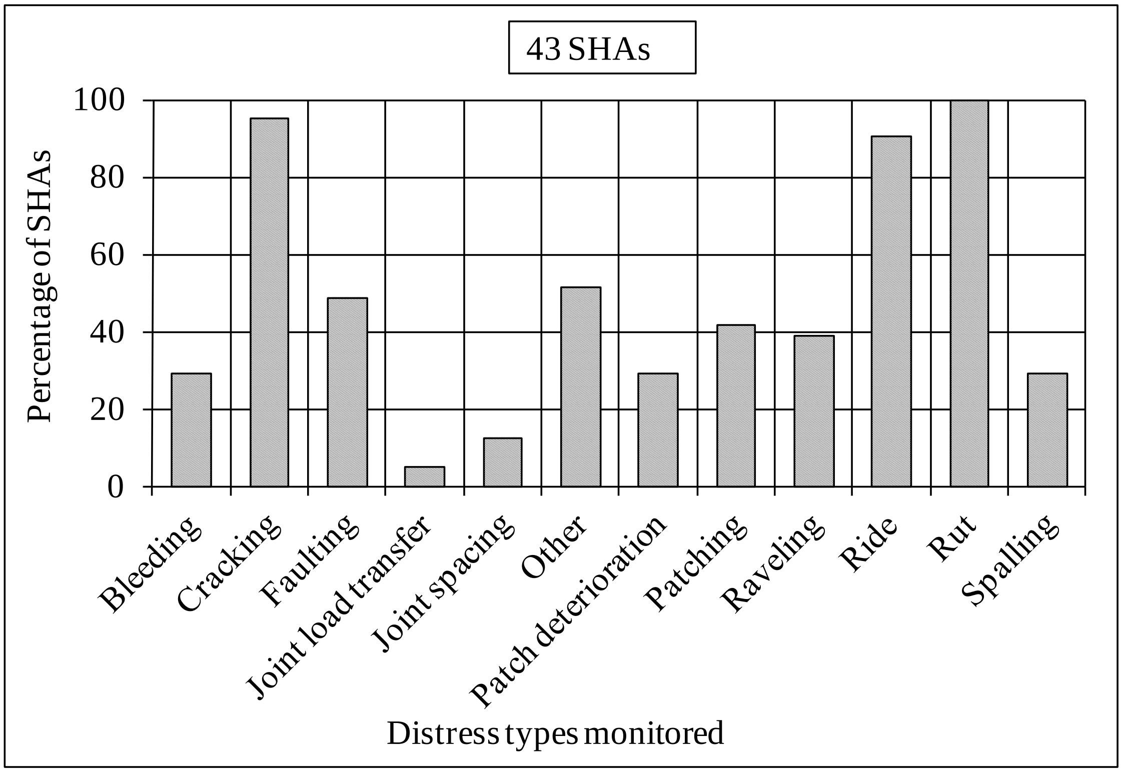


Figure 2.8 Percent of SHAs collecting the specified pavement condition data

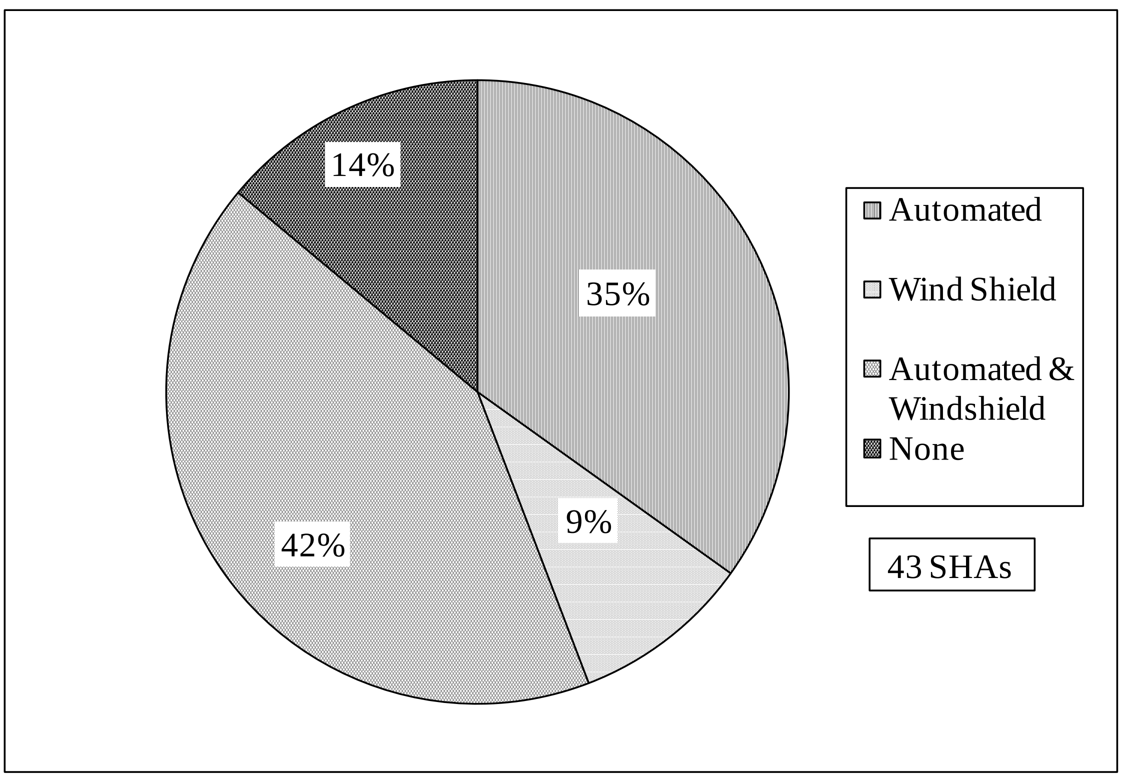


Figure 2.9 Percent of SHAs using the stated method of pavement condition data collection

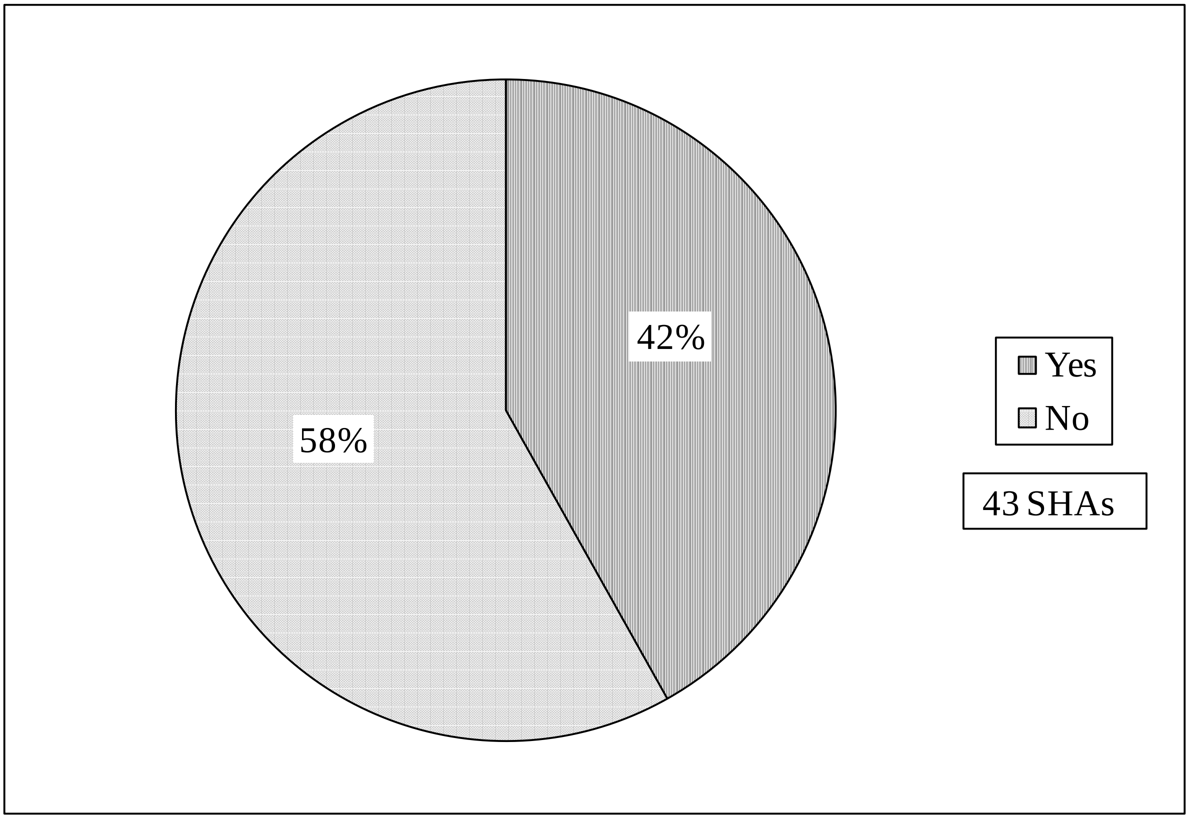


Figure 2.10 Percent of SHAs calculating the pavement rate of deterioration

used by SHAs to collect pavement condition and distress data vary from manual (wind-shield survey), to semi-automated (videotaping and digitizing the data by reviewing the videotapes), and to automated (computer identification/recognition of the images of the pavement condition and distress).

4. The method and the frequency of data collection vary from one agency to another. The frequency of pavement condition data collection (manual or imaging) varies from one to three years. Whereas sensor-measured features such as roughness, rut depth, and joint faulting data are typically collected every year. This variability is mainly related to the relative difficulties (cost, resources, and technology) of data collection and processing.
5. (Huang 2004). The optimum pavement monitoring frequency should be based on the cost and the associated risk for missing the optimum timing for treatment (Format 2005).

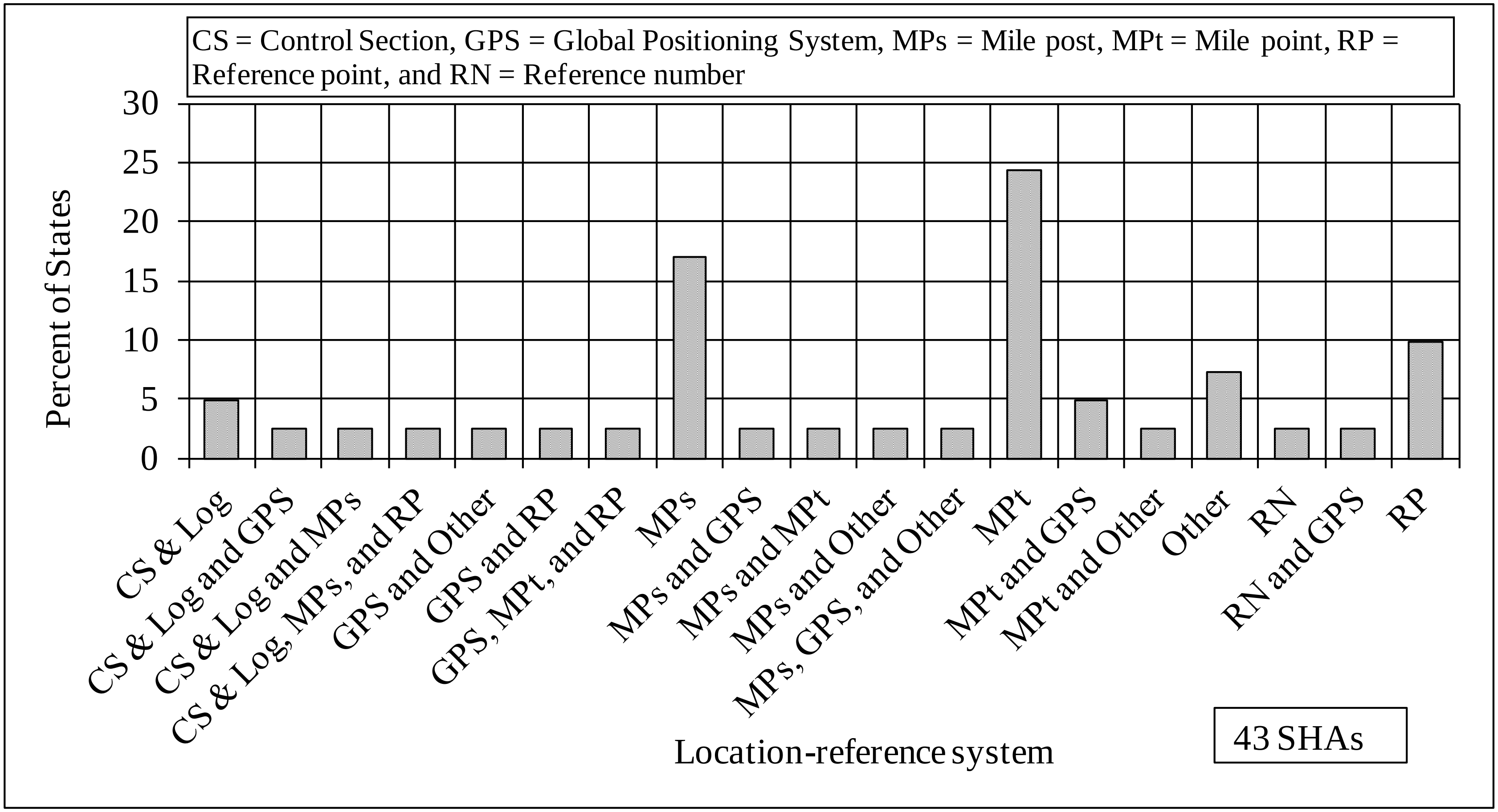


Figure 2.11 Percent of SHAs using the stated location reference system

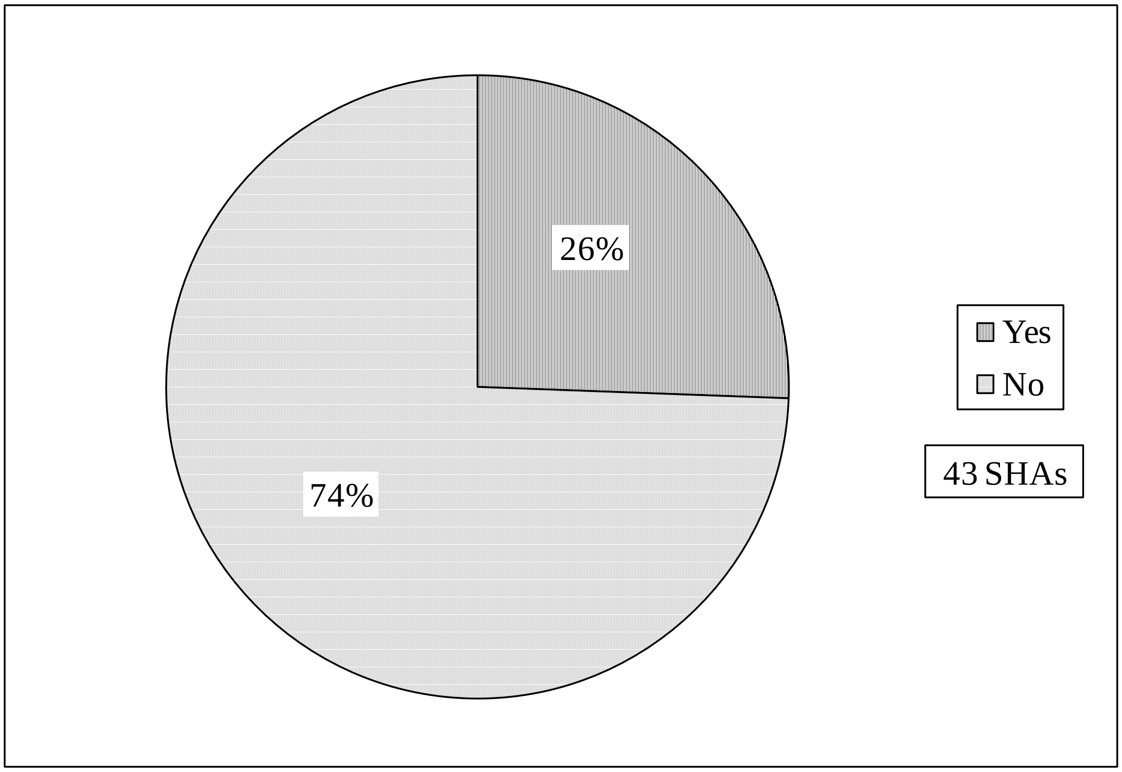


Figure 2.12 Percent of SHAs calculating the pavement life extension

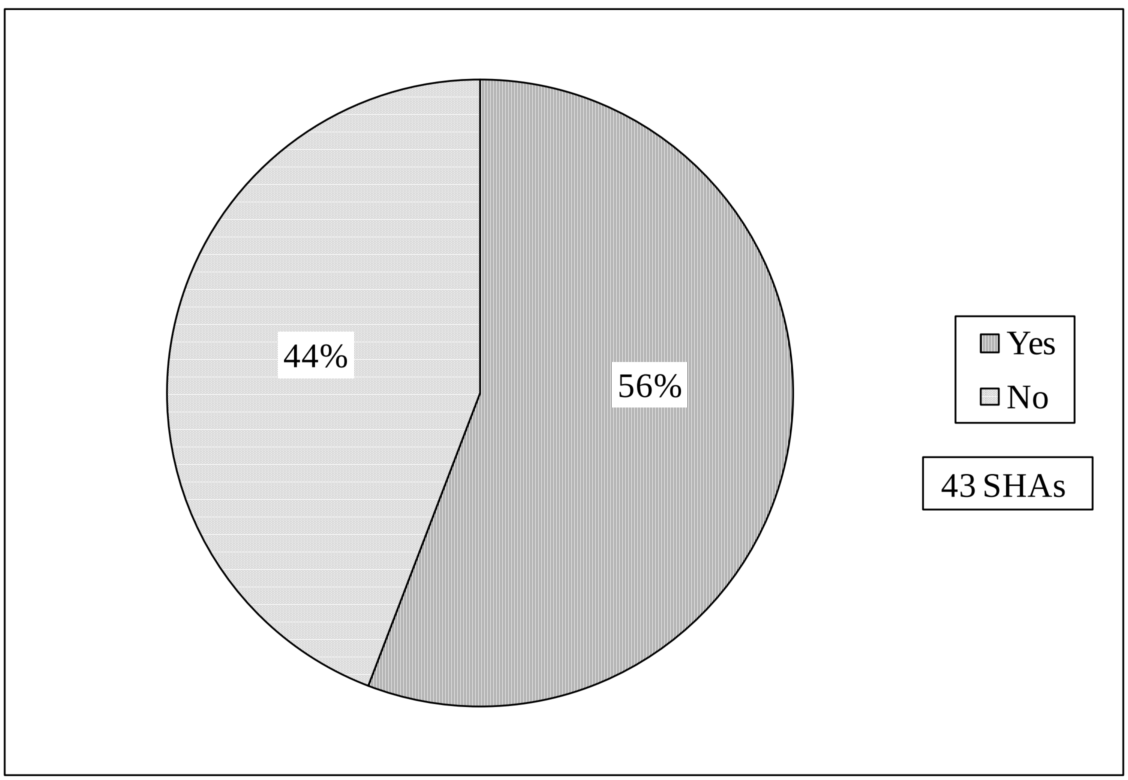


Figure 2.13 Percent of SHAs capable of forecasting pavement condition

6. Some SHAs collect data on a continuous basis and catalog the data for every 0.1 mile along the pavement network. Others use sampling procedure where the conditions and distresses of a short segment (100 to 500 ft) of the pavement are considered to represent the pavement conditions and distresses along one or more miles.
7. For multiple lane roads, almost all SHAs collect the pavement condition and distress data along the outer lane (the traffic lane).
8. All SHAs do not collect pavement condition and distress data on ramps on a regular basis.
9. All SHAs do not collect pavement condition and distress data on shoulders on a regular basis.
10. All SHAs reference their pavement condition and distress data using some sort of one or more linear location reference systems. Some of the reference systems are not physically identifiable along the pavement, while others are. SHA interest in Geographical Information System (GIS) and Global Positioning System (GPS) has increased recently. All SHAs that use GPS technology also use a linear reference system; the data are typically archived by mile post designation. Since most, if not all, highway personnel are trained to use their agency linear referencing systems, its combined use with GPS or GIS is likely to continue for the foreseeable future (McGhee 2004).

To this end, the PMS state-of-the-practice is affected by the set of condition and distress data and the accuracy/quality control of the data. These two topics are summarized later.

2.4.4 Pavement Condition and Distress Data

Most SHAs collect the pavement condition and distress data by type (e.g., transverse cracks, rut depth, and so forth), severity level, and/or extent. The extent of a given distress type is expressed by count, length, area, or percent depending on the SHA. Whereas, the distress severity levels are generally expressed in the descriptive terms low, medium, or high.

Monitoring pavement surface condition and distress over time is an essential part of pavement management and is required for the development and/or calibration of pavement performance models. The time-series condition and distress data could be used to accomplish the goals and objectives of a SHA. In particular, the data could be used to (Baladi et al. 2009):

1. Assess the pavement surface conditions and distresses at the time of survey.
2. Determine uniform pavement sections, with certain ranges of pavement surface conditions.
3. Determine the pavement rate of deterioration.
4. Calculate the RSL of each pavement survey segment and the distribution of RSL along the pavement network.
5. Determine the needs for pavement preservation and rehabilitation.
6. Assess, if possible, the causes of pavement conditions and distresses and select the most cost-effective preservation or rehabilitation actions.
7. Estimate the dollar value of the pavement network asset.
8. Estimate the short- and long-term budgetary needs.
9. Communicate with legislators and the community at large.
10. Provide better information to managers and decision-makers.
11. Reflect on the state-of-the-practice from project inception to project construction.

2.4.5 Pavement Condition and Distress Data Collection - Procedures

In general, SHAs collect pavement condition and distress data using one or more of three procedures, manual, semi-automated, and fully automated. Manual data collection procedure entails walking or driving along the pavement and observing and recording the pavement condition and distress types and their severity and extent. The procedure is conducted in different ways as stated below (FHWA 1995).

1. Walking on the shoulder and surveying, measuring, and recording 100 percent of the pavement surface distress types, severities, and extents. Some SHAs map the conditions whereas others do not.
2. Walking on the shoulder and surveying short pavement segment (sampled areas) of the pavement surface along which all distress types, severities, and extents are measured or estimated, recorded, and assumed to represent the pavement conditions and distresses along longer pavement sections.
3. Driving on the shoulder at creep speed and estimating and recording the pavement distress types, severities, and extents. Periodic shoulder stops are made at selected areas to measure the distress and check the estimates.
4. Driving on the outside lane and estimating some condition or distress types, severities, and extents. Periodic shoulder stops are made to verify the estimates.
5. Driving survey during which the pavement condition is assigned a general descriptive term such as excellent, very good, good, fair, poor, or very poor or sufficiency rating number without identifying individual condition or distress types and their severities and extents. The survey is done at the posted speed limit and is mainly affected by the pavement roughness.

Various significant issues are associated with the manual pavement condition survey. These include:

- The safety of the survey crew.
- The subjectivity of the surveyors and the variability between them.
- The effects of time of day, training, experience, fatigue, and weather, on the surveyors.

Given the issues associated with the manual pavement condition and distress survey, the state-of-the-practice has shifted toward semi-automated condition and distress data collection procedures (FHWA 1995, Freeman & Ragsdale 2003). The procedures consist of obtaining continuous digital or analog images on high resolution film or video. At some later time, trained observers view the images and identify and digitize the type, severity, and extent of the pavement surface conditions and distresses (FHWA 1995). The issue of this procedure is the subjectivity of the digitization process and the variability between the reviewers.

The fully automated procedure consists of obtaining continuous digital images of the pavement surface using light or laser technology. The images are examined by computers to identify and digitize the type, severity, and extent of the individual conditions and distresses in real-time or at a later time. This method would likely produce the most consistent and accurate data at a reasonable cost. Unfortunately, none of the SHAs have reported on the costs and benefits of the automated and semi-automated data collection techniques (Freeman & Ragsdale 2003, McGee 2004). Nevertheless, human elements would likely be necessary to perform quality control of the automated data as the technology improves regarding the accuracy and reliability of the data and the speed of data collection and acquisition (FORMAT 2003).

2.4.6 Pavement Data Collection – State-of-the-Practice

The evaluation of the pavement condition and distress data is an essential step in the application of suitable pavement maintenance and preservation practices (Cafisco 2002). The practices of data collection and analyses are limited by the technical, practical, and economical constraints within the SHAs. The National Cooperative Highway Research Program (NCHRP) Synthesis of Highway Practice 222 states that some of the practical constraints can be attributed to the size and variability of the condition of the pavement network. The accuracy of the data is directly related to the data collection frequency, the selected representative samples, and the sample size (Zimmerman 1995). Sampling of pavement condition and distress data requires less time and money than continuous data collection, and is meant to support accurate pavement condition and distress predictions without excessive amounts of data (Robertson et al. 2004). In the NCHRP 2004 Synthesis of Highway Practice 334, 42 states, the District of Columbia, 2 FHWA offices, 10 Canadian provinces and territories, and Transport Canada (airfields) were surveyed regarding data collection (McGhee 2004). It was found that:

1. Most agencies use semi-automated means for pavement condition data collection along the entire outer traffic lane every other year.
2. For pavement cracking:
    a. Nine agencies reported that they survey 100% of the lane to be evaluated.
    b. Three agencies collect cracking data on a varying length basis.
    c. Five agencies sample 10% to 30% of the roadway using a random sampling technique.
    d. Others reported that they videotape 100% of the survey lane but, for each one-mile, they digitize the data along 50 to 1,000 foot segments.
3. For pavement roughness:
    a. Many agencies collect the data along the entire surveying lane and they report the data for each 0.1-mile interval.
    b. The Canadian provinces report the roughness data at 50-meter to 100-meter intervals.
    c. In the District of Columbia, the reporting intervals are measured by one city block.
    d. The State of Arizona uses a reporting interval of 1 mile.

In another report, the ministry of transportation of Quebec, Canada, reported the use of sampling of 30-meter pavement segment for every 100-meter pavement segment, or thirty percent sample size (Tremblay et al. 2004).

In addition, the information provided in Figures 2.4 through 2.13 indicate that the data collection and logging practices vary greatly from one SHA to another. The minimum reported survey length is 0.001 miles while the longest is 1 mile. The survey length is typically decided by two main criteria, the sampling procedure and the data storage and analysis capacity of the SHA. The agencies who practice data sampling typically store the data along a longer segment length. For example, a SHA who samples 30%, collect data for 1,581 feet (30% of a mile) and store the data along the represented mile. On the other hand, agencies who practice continuous data collection typically store their data along shorter segments, such as 0.1 mile (Dean et al. 2011).

2.4.7 Pavement Condition and Distress Data Collection - Sampling

Some SHAs collect pavement condition and distress data on a continuous basis and log the data over short pavement segments (0.1 mile, 0.5 mile, or 1 mile), whereas others use sampling techniques to reduce the time and costs of data collection. It is typically assumed that the pavement condition of a pavement surveying length (e.g., one mile, two miles, or longer) can be represented by the conditions of the pavement along a shorter segment (such as 100, 200, or 500 ft) along the survey segment. Two sampling techniques are typically utilized; randomly selected short segments or fixed short segments.

Some SHAs (such as the Colorado Department of Transportation, CDOT, the Louisiana Department of Transportation and Development, LADOTD, the Michigan Department of Transportation, MDOT, and the Washington State Department of Transportation, WSDOT, use semi-automated technology (sensors and images) to collect pavement condition and distress data. The sensor (longitudinal and transverse profiles) and image data are collected and digitized on continuous basis and the data are stored for 0.1 mile long pavement segments. Although continuous data collection procedure provides the SHAs with the most accurate and detailed data, it may not be the most cost effective procedure. Sampling will undoubtedly reduce the immediate cost of data collection at the expense of data accuracy. Data accuracy, on the other hand, affects the quality of the decisions regarding the selection of the treatment type, time, and project boundaries. The costs of lower quality decisions due to sampling could be much higher than the savings incurred by sampling. Hence, the decision to sample or not to sample must be based on the ratio of the cost of lower quality decisions and the savings incurred by sampling. The lower is this ratio the more cost-effective is the data collection technique. One study suggests that 70% sampling (data collected from 70% of the pavement surface) will nearly eliminate statistically significant error. While in another study, a methodology was developed for the selection of sample sizes based on error and costs to agencies and users (Mishalani & Gong 2007, Ong et al. 2008).

The common intent of sampling is to properly assess the current pavement condition and distress and reduce the time and cost of the data collection. The proper assessment of the data collection must be judged based on the effects of sampling on the accuracy of the various decisions regarding the selection of treatment type, time, and project boundaries. It has been shown that 10% sampling yields about ± 50% error in sensor based data and leads to deviations in project boundary selection of up to 3 miles. The error decreases with increased sampling size, but was found to be ± 20% at 60% sampling for sensor based data. The errors associated with image based data were found to be more than 100% even with 60% sampling. It should be noted that the error associated with sampling substantially decreases for more uniform pavement sections. The cost of inaccurate decisions could be several folds higher than the savings incurred by sampling due to the errors in project identification and treatment selection (Dean et al. 2010, Dean et al. 2011).

Dean et al. 2012 conducted analyses to determine the effects of data reporting and survey lengths on the accuracy of the pavement management decisions and on the data analysis. Time-series data collected and stored for each 0.1 mile long pavement segment were used to calculate the conditions and distresses along longer survey lengths (0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, and 1 mile), thus making it possible to study the impact of data reporting intervals on the accuracy of data modeling. It was found that the pavement survey length does not affect the capability of the data to be modeled using mathematical functions. They added that data reported along longer pavement segments lack the detail regarding the variability of the pavement condition and distress along the reporting segments. Given the low cost of data storage, they recommended reporting intervals of 0.1 mile.

2.4.8 Pavement Condition and Distress Data Collection - Frequency

According to Haider et al. 2010 the frequency of pavement condition and distress data monitoring could be divided, in general, into three categories:

1. Short frequency (1 to 2 years); the data are typically used to develop or calibrate pavement performance models.
2. Long frequency (3 to 4 years); the data are typically used for assessing the general condition of the pavement network.
3. Localized monitoring; the data are typically used to address an unexpected pavement performance.

Most SHAs monitor the pavement condition data every 1-, 2-, or 3-years (McGee 2004). Some SHAs collect sensor-based data (longitudinal and transverse pavement profiles) more frequently than image-based data. However, it has been shown that, to make accurate pavement decisions; image based data (cracking) should be collected every year and sensor data could be collected less frequently (Haider et al. 2010, Haider et al. 2011).

2.4.9 Pavement Condition and Distress Data Evaluation

The pavement condition and distress data collected in the field must be evaluated in order to determine the condition or the health of the pavement network. Many SHAs assign distress points to each condition or distress type, severity level, and extent. The combination of those points is used to yield indices which can be used to describe the conditions of the pavement structures. Data evaluation is discussed with examples from LADOTD and MDOT in the following few subsections.

2.4.9.1 Pavement Condition and Distress Engineering Criteria

Pavement conditions and distresses are typically expressed using one or more of the following methods (see Figure 2.14) (Baladi et al. 1992):

1. A descriptive scale, such as very good, good, fair, poor, and very poor.
2. A distress index based on a continuous rating scale such as zero to ten or zero to hundred. The two ends of the scale define either “failed” pavement or pavement in excellent condition (such as in a new pavement) as shown in Figure 2.14. The distress index could be calculated using one distress type (individual distress index) or a combination of various types of distress (composite pavement index). Along the rating scale, a threshold value is typically established to trigger pavement action. A distress index value below the established threshold indicates substandard pavement condition whereas above the threshold indicates acceptable conditions.

Deduct distress points rating scale with a threshold value of 60 points
(100 = Excellent pavement)

0	10	20	30	40	50	**60**	70	80	90	100

Very poor	Poor	Fair	Good	Very good

Descriptive scale

Engineering criterion = maximum acceptable number of cracks = 50 transverse cracks per 0.1 mile long pavement segment, which are equivalent to 1 crack every 10.5 feet and correspond to 60 points on the rating scale

Deduct value method

Distress points per transverse crack = DP/TC

$$\text{DP/TC} = \frac{(\text{maximum rating scale value} - \text{threshold value})}{\text{Maximum acceptable number of transverse cracks}} = 0.8$$

$$\text{DP/TC} = \frac{(100 - 60)}{50} = 0.8$$

Distress points = DP = 100 - (0.8)(the number of TC)

Cumulative distress points rating scale with a threshold value of 40 points
(0.0 = Excellent pavement)

0	10	20	30	**40**	50	60	70	80	90	100

Very good	Good	Fair	Poor	Very poor

Descriptive scale

Cumulative distress points

Distress points per transverse crack = DP/TC

$$\text{DP/TC} = \frac{(\text{Threshold value})}{\text{Maximum acceptable number of transverse cracks}} = 0.8$$

$$\text{DP/TC} = \frac{(40)}{50} = 0.8$$

Distress points = DP = (0.8)(the number of TC)

Figure 2.14 Rating and descriptive scales, and distress points

After a SHA adopts a continuous rating scale (0.0 to 100.0, 0.0 to 10.0, or 0.0 to 5.0), a threshold value along the scale should be selected (such as 60 on the zero to 100 rating scale shown in Figure 2.14). It should be noted that the value of this threshold does not impact the relative condition of the pavement segment being rated. For each condition or distress type, an engineering criterion must be selected based on the experience of the agency. The engineering criterion addresses the extent of the condition or distress at which the pavement section in question is deemed in need of repair. Such criterion could be 50 transverse cracks along a 0.1 mile pavement segment. Based on the engineering criterion and the rating scale threshold value, distress points are assigned to each occurrence of the condition or distress (each transverse crack). To illustrate, consider the continuous rating scale 0.0 to 100.0 (the 100 is the rating of a perfect pavement section) and its threshold value of 60 points shown in Figure 2.14. Assume the engineering criterion for transverse cracks is 50 cracks per 0.1 mile long pavement segment as shown in the figure. In this illustration, the pavement structure would lose 40 points (from 100 to 60) when it accumulates 50 transverse cracks. Based on a linear scale, each transverse crack is worth 0.8 distress point (after Baladi et al. 1992).

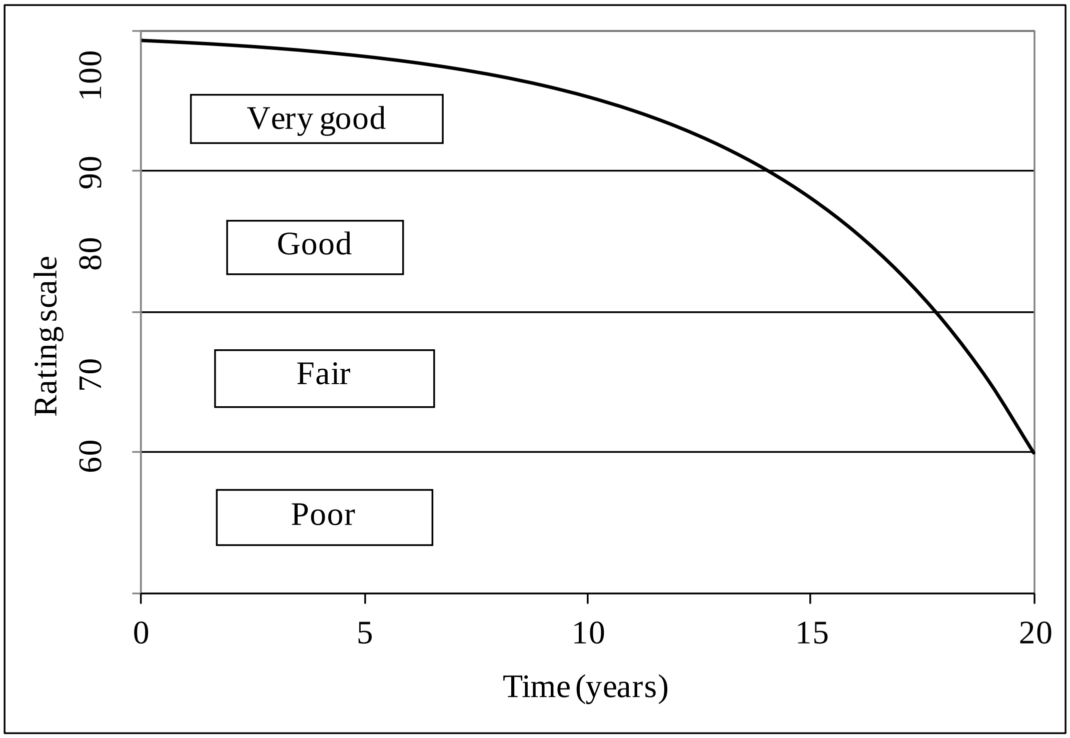
For the rating scales shown in Figure 2.14, it is assumed that the threshold values of 60 (deduct value scale) and 40 (cumulative distress point scale) indicate that the pavement section is in need of heavy rehabilitation actions. The continuous rating scale (based on deduct values or cumulative distress points) could be divided into various regions to trigger scheduled maintenance, preventive or preservation maintenance, heavy rehabilitation, and reconstruction as shown in Figure 2.15. It should be noted that any of the maintenance, preservation maintenance, rehabilitation, or reconstruction could be applied to any pavement segment at any time. However, the treatment will be most cost-effective if it is applied to pavement sections within particular region. For example, a pavement section could be reconstructed or subjected to heavy rehabilitation actions while its condition is in the do-nothing region. Such action or actions will not be cost-effective actions (Dawson et al. 2011).

Further, the engineering criterion may be different for each pavement type, pavement condition or distress type, unit of measure, distress severity level, and road class. Therefore, each scenario must be considered separately. The methods used to develop the engineering criteria should be well documented and the criteria should be studied and calibrated as more pavement condition and distress data become available.

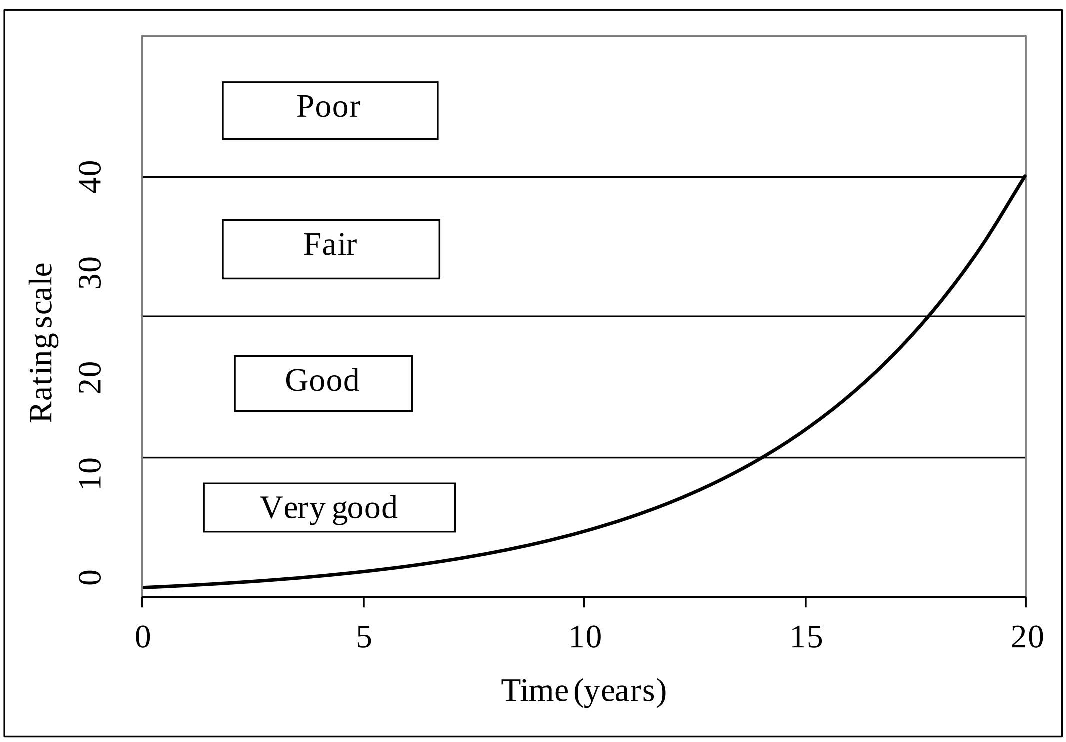
The LADOTD, for example, assign predetermined deduct points for each type and severity level of pavement condition or distress. The deduct points for alligator cracking are listed in Table 2.2. Further, for each type of condition or distress, a distress index is calculated by subtracting the sum of all deduct points for each 0.1 mile of pavement segment from 100. Hence, for all types of condition or distress, the distress index is based on a scale from 0 to 100 with 100 indicating “new” pavement surface conditions and 60 requiring treatment. Along the distress index scale, several threshold values were established by the LADOTD. Accumulation of deduct points to reach each threshold value triggers certain type or types of maintenance or rehabilitation. Hence, the pavement rehabilitation actions that are recommended by the LADOTD are based on the accuracy of the distress index, which is based on the assigned deduct points and on the predetermined threshold values (Khattak & Baladi 2007). It should be noted that the distress points listed in Table 2.2 are being examined and will be updated shortly.

2.4.9.2 Pavement Condition Indices

Pavement condition indices are often based on one or more condition or distress type. For example, alligator cracking index (which is an individual index) is based on the severity levels



a) Rating scale with deduct distress points



b) Rating scale with cumulative distress points

Figure 2.15 Various regions of rating scales or pavement condition (Dawson et al. 2011)

Table 2.2 LADOTD deduct points for alligator cracking (Khattak & Baladi 2007)

Alligator cracking deduct points						
Severity	Extent (ft^2)					
	0-51	51-701	701-1301	1301-2401	2401-3168	3168-9999.99
Low	0	1-16	16-21	21-25	25-28	28
Medium	0	1-21	21-29	29-36	36-49	48
High	0	1-29	29-43	43-50	43-61	61

(low, medium, and high) of the alligator cracks and their associated extents. On the other hand, a combined pavement index (such as pavement condition index, pavement rating index, and so forth) consists of two or more condition or distress types. The index is calculated based on each individual condition or distress type (see Tables 2.3 through 2.5), the condition or distress severity levels and their associated extents, as well as on a weight factor assigned to each severity level and extent of the distress type (Baladi et al. 1992). For all measured or estimated pavement condition and distress data, MDOT has defined sets of deduct points for each pavement condition or distress type, severity level and extent. The sum of all of the assigned deduct points are used to calculate one composite index called “distress Index, DI” (see Equation 2.3). Pavement rehabilitation is triggered by a DI value of 50. It should be noted that the MDOT DI scale starts at zero for perfect pavement (no distress) and is open ended (Baladi et al. 1999).

$$DI = \frac{\sum DP}{N} = \left(\frac{0.1}{L}\right)\left(\sum DP\right) \quad \text{Equation 2.3}$$

Where, DI is the distress index;

$\sum$ DP is the sum of the distress points along the project;

N is the number of 0.1-mile long segments along the project (N = L/0.1);

L is the project length in miles

Table 2.3 MDOT pavement condition categories (flexible pavement) (Baladi et al. 1999)

Code	Terminology	Code	Terminology
0101	Transverse tear	0320	Good partial width patch (white)
0103	TC (straight)	0321	Fair partial width patch (white)
0104	TC (irregular)	0322	Poor partial width patch (white)
0201	LC-centerline	0323	Good partial width patch (black)
0202	LC-center of lane	0324	Fair partial width patch (black)
0203	LC-edge	0325	Poor partial width patch (black)
0204	LC-right wheel path	0404	Surface treatment type
0205	LC-left wheel path	0405	Raveling
0220	Alligator crack - rwp	0406	Flushing
0221	Alligator crack - lwp	0407	Intensive miscellaneous cracks
0304	Full width patch (white)	0601	Rut depth - rwp
0310	Block cracked	0602	Rut depth - lwp
0313	Full width patch (black)		
rwp = right wheel path; lwp = left wheel path			

Table 2.4 MDOT pavement condition categories (composite pavement) (Baladi et al. 1999)

Code	Terminology	Code	Terminology
0101	Transverse tear	0321	Fair partial width patch (white)
0110	Transverse crack	0322	Poor partial width patch (white)
0201	Longitudinal crack (LC)-centerline	0323	Good partial width patch (black)
0202	LC - col	0324	Fair partial width patch (black)
0203	LC-edge	0325	Poor partial width patch (black)
0204	LC - rwp	0404	Surface treatment type
0205	LC - lwp	0405	Raveling
0304	Full width patch (white)	0406	Flushing
0311	Intensive reflective cracks	0601	Rut depth - rwp
0313	Full width patch (black)	0602	Rut depth - lwp
0320	Good partial width patch (white)		
rwp = right wheel path; col = center of lane; lwp = left wheel path			

Table 2.5 MDOT pavement condition categories (rigid pavement) (Baladi et al. 1999)

Code	Terminology	Code	Terminology
0106	Transverse joint	0306	Shattered area
0107	Intensive transverse cracks	0307	Mud jacked area
0112	Transverse crack open > 1/4 in.	0313	Full width patch (black)
0113	Transverse crack	0320	Good partial width patch (white)
0208	Longitudinal joint, left	0321	Fair partial width patch (white)
0209	Longitudinal joint, right	0322	Poor partial width patch (white)
0227	Longitudinal crack open > 1/4 in. – rwp	0323	Good partial width patch (black)
0228	Longitudinal crack open > 1/4 in. – col	0324	Fair partial width patch (black)
0229	Longitudinal crack open > 1/4 in. – lwp	0325	Poor partial width patch (black)
0230	Longitudinal crack – right wheel path	0401	Corner crack
0231	Longitudinal crack – center of lane	0402	Pop-out
0232	Longitudinal crack – left wheel path	0403	Scaling
0301	Delamination	0404	Surface treatment type
0302	Reactive aggregate	0601	Rut depth - rwp
0303	High steel	0602	Rut depth - lwp
0304	Full width patch (white)		
rwp = right wheel path; col = center of lane; lwp = left wheel path			

Nevertheless, neither the distress points nor the condition index express the true nature of the pavement conditions. For example, immediately after construction, the cumulative distress points of a pavement project where 2-inch asphalt overlay were placed are exactly the same as the cumulative distress points for another project where 6-inch asphalt overlay were placed. The pavement surface conditions of both projects are free of distresses. Hence, the value of the condition index of both projects is the same. Stated differently, neither the distress points nor the condition index express the design life of the overlay or to that extent, the service life of the pavement segment itself. In addition, some SHAs use the improvement in the value of the condition index as a measure of the benefits of the maintenance or rehabilitation treatments. Based on that measure, for a given project, the benefits of 2-inch or 6-inch overlay are the same, whereas the cost is much different. Hence, many pavement projects were covered with thin overlays. This practice resulted, over time, in the degradation of the entire pavement network to unacceptable conditions (Baladi et al. 2009).

The true trigger values for actions (preventive maintenance, rehabilitation, or reconstruction) must be based on sound engineering criteria. These include the pavement distress or condition type (such as the number of high severity transverse and/or longitudinal cracks, and/or the serviceability level), the pavement rate of deterioration, and the cause of distress. In this study, the analyses and conclusions are based on the original condition and distress data, not the SHA calculated condition indices.

2.4.9.3 Pavement Project Delineation

Project delineation is the process of identifying certain lengths of pavement with common (uniform) conditions and rates of deterioration for the selection of project boundaries. The AASHTO unit delineation method is based on the cumulative differences between adjacent data points along the road. The method compares the overall average project condition to each data point of each pavement survey length such as 0.1 mile. Project boundaries are defined where there is a change in the slope (positive to negative or vice versa) of the cumulative differences between the average project condition and the condition of each pavement survey segment (AASHTO 1993). Other methods include grouping pavement segments with similar conditions and/or rates of deterioration, and selecting boundaries based on a visual inspection without quantifying the conditions of the pavement section.

2.4.10 Pavement Condition and Distress Data Quality Control and Quality Assurance

The accuracy of the pavement condition and distress data is the most important aspect of an effective PMS. Optimum management of the pavement systems cannot take place without accurate data that reflect the true pavement conditions and distresses and hence, they can be trusted by the PMS users. Unfortunately, the sources of errors in the data are difficult to identify and address. Errors can result from the data collection system, seasonal changes, human error, the evaluation strategy, the location reference system, or from multiple other sources. Table 2.6 lists some common sources of error in the pavement condition data (Fugro-BRE 2001). Reliable predictions of pavement condition are not possible without accurate, consistent, and dependable condition and distress data. Discrepancies between performance model predictions and observed field performance are conventionally attributed solely to errors in the data (Hall et al. 2003). In fact, significant errors could be attributed to inherent uncertainty in the measured data due to spatial variability such as sampling and measurement errors (Batac & Ray 1988, Brown 1988, Labi & Sinah 2005). Likewise, the frequency of data collection contributes to the uncertainty in pavement performance prediction. Further, the uncertainty associated with the method (manual, semi-automated, and automated) of condition and distress data collection has been well documented in the literature (Chong & Phang 1988, Sharaf et al. 1988, SHRP 1990, Rajagopal & George 1991, Al-Mansoor & Sinah 1994, Hicks et al. 1997, Morian et al. 1998, Eltahan et al. 1999, Hall & Corvetti 2000, Hall et al. 2003, M-E PDG 2004).

Table 2.6 Common sources of error in the measured pavement condition data (Fugro-BRE 2001)

Human error	Seasonal effects	Data collection methodology	Evaluation strategy	Unknown
7%	0^{+}%	6%	36%	41%
Condition definition	Thermal effects on crack width	Manual vs. automated	Nonlinear decrease in condition	
Summarization	Visibility of distress due to surface moisture	Color, contrast, and depth perception	Insufficient quantities of distress	
	Resolution	Undocumented maintenance and rehabilitation		

Ultimately the quality of the data must be managed in a two step process, quality control and quality assurance. The industry or the vendor responsible for the data collection should perform quality control. Quality control consists of those actions that must be taken by the vendors to ensure that the equipment and processes involved are producing high-quality and accurate results. Quality assurance, on the other hand, consists of those actions that must be taken by the SHA (the owner and user of the data) to ensure that the final products are in compliance with the contract provisions or specifications (TRB 1999, McGee 2004). The SHA could perform the quality assurance in-house or by contracting independent vendors other than those collecting the data.

Most SHAs have acceptance criteria by which they assess the accuracy of automated data versus manually collected data. An example of such is the acceptance criteria utilized by the Mississippi Department of Transportation (MDOT), listed in Table 2.7 (McGee 2004). Some SHAs rely on automated analysis tools to verify the accuracy of the automated data collection equipment. An example of such tool is the Road Inventory Program Data Assurance Testing Application (RIP-DATA). The program checks for discrepancies in the data, data format, data completeness, data range, and the degree of consistency between inter-dependent and related data (McGee 2004). Another procedure to assure the quality of a given pavement condition and distress data set is to check it against a system of existing records. In this procedure however, it is assumed that the data in the system of records are reliable and accurate. The above scenarios indicate that it could be extremely difficult to evaluate the true accuracy of a given set of pavement condition and distress data (Wolters et al. 2006).

Additionally, the required accuracy of the pavement condition and distress data is dictated by the condition or distress type and the types of data applications. For example,

Table 2.7 Mississippi DOT Acceptance criteria (McGee 2004)

Condition indicator	Tolerance
IRI	± 19 inch/mile
Rut depth	± 0.09 inch
Faulting	± 0.07 inch

decisions made at the network level require less accurate data than decisions made at the project level (Format 2003). In general, it is ideal and more cost-effective to collect the data to support project level decisions and to summarize the data for network level decisions.

Finally, the accuracy of the pavement performance prediction models is a function of the accuracy of the pavement condition and distress data. The effect of data accuracy on prediction and consequently the decision-making process is discussed in detail later.

2.4.11 International PMS Practice

Some degrees of PMS practice are exercised by various HAs across the world. Some authorities have established unique sets of pavement inventory and condition data and data collection procedure. Table 2.8 lists the types of inventory data collected by six HAs outside the USA (Sood et al. 1994, Gaspar 1994, Gutierrex-Bolivar & Achutegui 1994, Phillips 1994). Table 2.9 summarizes the collection and the usage of several types of pavement data by 15 nations at the network and project levels (Format 2005). The data presented in Tables 2.8 and 2.9 indicate that the collected PMS data vary greatly from one country to another. Each authority collects data on the conditions that are prevalent on the surfaces of their pavement networks (such as cracking, rutting, roughness, etc.) and records the data using various units (number, area, or length of cracks). Further, the data are used to calculate various condition indices. The frequency of data collection also varies from once every year to once every five years. Finally, as is the case in the USA, the methods of pavement condition data collection vary around the differences in the PMS practices and pavement rehabilitation strategies between the various countries are similar to those between the various SHAs in the United States.

Table 2.8 PMS data collected by six HAs outside the USA (Sood et al. 1994, Gaspar 1994, Gutierrex- Bolivar & Achutegui 1994, Phillips 1994)

Data type	Nation					
	India	Hungary	Spain	United Kingdom	Austria	South Africa
Pavement surface type	X		X	X	X	X
Pavement width	X		X	X	X	X
Pavement layer thickness	X	X	X			
Shoulder width	X		X	X		
Number of lanes						
Drainage type		X		X		
Traffic data	X		X			

2.5 Pavement Management System – State-of-the-Art

The progression and development of methodologies and technologies that constitute the PMS state-of-the-art are discussed below.

2.5.1 Automated Pavement Condition Data Collection and Processing

Technology is leading the way toward a faster, safer, more accurate, and more cost-effective system of collecting and processing pavement condition and distress data. The current state-of-the- art technology in computer aided imaging, sensing, and processing can quickly, efficiently, and cheaply collect and process data (McGhee 2004). Automated data collection and processing refers to the method of pavement imaging, sensing, and interpreting where humans are not directly involved.

Three major technologies are available for automated imaging; area scan, line scan, and 3-D imaging. Area scan technology produces a digital image of the pavement 6 to 12 feet wide and 10 to 15 feet long (see Figure 2.16). The images are collected consecutively along the

Table 2.9 PMS data items collected and used by 15 HAs outside the USA (Format 2005)

Percent of data collected and used by the 15 HAs	Network level	Project level
Wide usage 75 to 100%	Longitudinal unevenness	Longitudinal unevenness
	Transverse unevenness	Transverse unevenness
	Crack information	Crack information
	Friction	Pavement structure
		Bearing capacity
		Pothole
		Cross fall
Moderate usage 25 to 75%	Macro texture	Macro texture
	Curvature	Curvature
	Road marking condition	Road marking condition
	Mega texture	Mega texture
	Other surface defects	Other surface defects
	Gradient or hilliness	Gradient or hilliness
	Pothole	Friction
	Cross fall	Noise (outside vehicles)
	Bearing capacity	Stepping, Faulting
	Pavement structure	
Low usage 0 to 25%	Vibration (unevenness)	Vibration (unevenness)
	Pavement reflectance	Pavement reflectance
	Noise (inside vehicles)	Noise (inside vehicles)
	Rolling resistance	Rolling resistance
	Stepping, Faulting	
	Noise (outside vehicles)	

pavement. Line scan imaging technology works similar to a fax machine, by scanning one dimensional “lines” and connecting them to produce an image of the pavement (see Figure 2.17). Both technologies require proper lighting and favorable environmental conditions; and can detect cracks as small as 1 to 2 mm wide. On the other hand, laser-based imaging technology eliminates the need for lighting and is capable of developing 3-D profiles and images of the pavement surface. The technology has not yet been proven to be useful for cracking, but it can detect rutting and roughness very well (McGhee 2004).

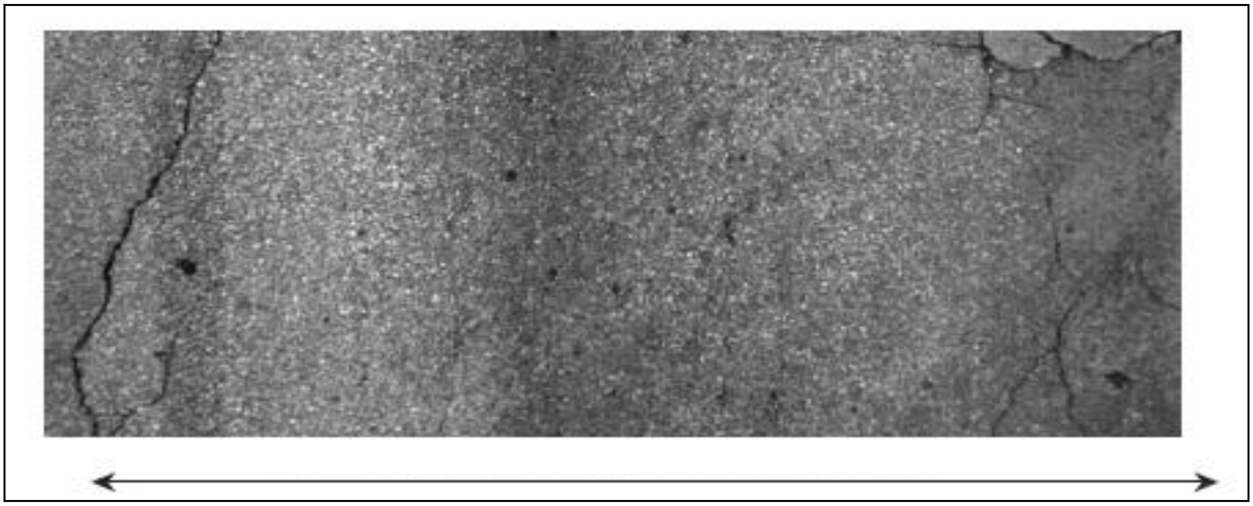


Figure 2.16 Area scan image (McGhee 2004)

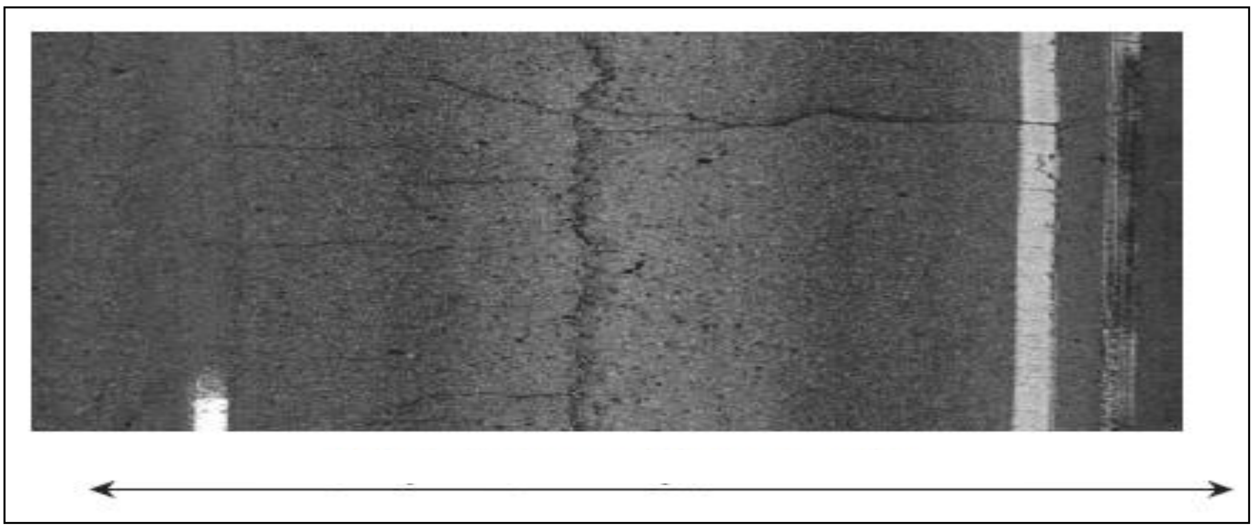


Figure 2.17 Line scan image (McGhee 2004)

Likewise, three major technologies exist for sensing pavement condition (rutting and roughness); laser, acoustic, and infrared sensing. Each technology is based on estimating the distance to the pavement surface in reference to a stationary panel of sensors (3 to 37 sensors) mounted to a travelling vehicle. Each sensing method works on the same principal; measurement of the time required for a wave traveling at a known speed to bounce off the pavement and return to the sensor. The major difference is the type of wave produced; laser (light), acoustic (sound), or infrared (electro-magnetic). The state-of-the art is moving toward laser based technology for their speed and more robust working conditions (McGhee 2004).

The automated collection of images and profiles is followed by the automated processing of the data. Roughness and rut depth data are typically processed in real time as the sensors are collecting the data, while cracking data must be retrieved from the image data. Few technologies are available which can detect, quantify, and classify cracking in real-time or very short time in comparison to manual image digitization. Automated condition data are more consistent (no human interaction), safer, likely more accurate, and can be obtained much more quickly (McGhee 2004). The advantages of time and accuracy would likely outweigh the relatively high equipment cost disadvantage.

2.5.2 Rolling Wheel Deflectometer

Most SHAs only survey their pavement networks for condition and distress data, while few are beginning to collect network level structural data. The most common method of collecting pavement structural data is non-destructive deflection tests (NDT) using a Falling Weight Deflectometer (FWD). The measured deflection data are used to study the variability of the structural capacity along the pavement, estimate the pavement layer moduli, for rigid pavements, the load transfer efficiency and void detection under the slabs (Diefenderfer 2010).

The FWD is practically limited to project level analyses due to its slow operating speed. The FWD testing at the network level is limited to 20 to 25 lane-miles per day (Galal et al. 2007, Diefenderfer 2008). To overcome this constraint, the Rolling Wheel Deflectometer (RWD) was developed. The RWD is similar to the FWD in that pavement surface deflection is measured due to the application of 9,000 pound load. The RWD is mounted on a 53 foot semi-trailer and can be used at traffic speed. Hence, the RWD is capable of testing 200 to 300 lane-miles per day, qualifying it for network level analyses (Diefenderfer 2010).

The accuracy of the RWD measured deflection data was assessed by the Virginia Department of Transportation (VDOT). The VDOT conducted RWD and FWD tests along sections of I-81 and I-64 in Virginia and compared the measured deflection data. The RWD tests consisted of three passes along each pavement section. The measured deflection data were then normalized to 68°F ambient temperature and 9,000 pound applied load, and recorded for 0.1 mile pavement segments. The RWD results were statistically analyzed, the results indicated that (Diefenderfer 2010):

- The deflection data measured in three consecutive passes are significantly different. Hence, the RWD test results are not repeatable.
- No significant statistical correlations were found between the FWD and the RWD data.

2.5.3 Pavement Management

Pavement management and PMSs are evolving and have become more technically and operationally advanced. The improvements in the speed of data collection, the accuracy of the data, and the availability of data have allowed PMSs to make more informed and cost-effective decisions. The state-of-the-art in pavement management is discussed in the next few sections.

2.5.3.1 The Remaining Service Life and the Remaining Life

To overcome the disadvantages of the distress index, (Baladi et al. 1985) presented an algorithm for the calculation of the pavement RSL. For a given pavement project, the RSL is defined as the time, in years, between the condition survey year and the time when the condition or distress reaches a critical value (for example, two high severity transverse cracks in a 20 foot long concrete slab, or 0.5-inch rut on Interstate flexible pavement). After construction, the RSL of a given pavement project is equal to or less than the design life of the applied treatment action or the design service life of the newly constructed or reconstructed pavement. For example, for a given pavement section, the RSL after the construction of a 2-inch overlay that was designed to last 3 years is 3 years whereas after construction, the RSL of a pavement project subjected to 6-inch overlay that was designed to last 9 years is 9 years. This implies that the RSL algorithm:

a. Must include the assumed pavement or treatment design life. This would assist in the integration of pavement design into the PMS.
b. Can be used to calculate the resulting improvement in the RSL (the service life extension (SLE)) after the application of preventive maintenance or rehabilitation actions. The SLE represents the benefit of the treatment to the agency in terms of years of service.

The RSL is a self calibrating indicator, the RSL of any pavement segment where no preservation or rehabilitation actions were applied, should decrease by one year for each calendar year. If not, the RSL algorithm or the condition and distress data used for calculating the RSL must be checked for accuracy and corrected (Baladi et al.1985, Baladi et al. 1992, Baladi 2007).

Further, the RSL values could be used to calculate the percent of the pavement network having the same RSL value or range of values. The information could then be used to display the distribution of the RSL along the pavement network as shown in Figure 2.18. The ideal network distribution of RSL is uniform, as shown in Figure 2.19 (Baladi et al. 2008).

For a given pavement network, the RSL concept could be used to calculate the weighted average RSL of all pavement segments within the network. Hence, the RSL of a pavement network expresses the longevity of the network. The RSL of a network can then be used to assess the impact of a given budget on the conditions of the network. Such assessment may yield three possible scenarios:

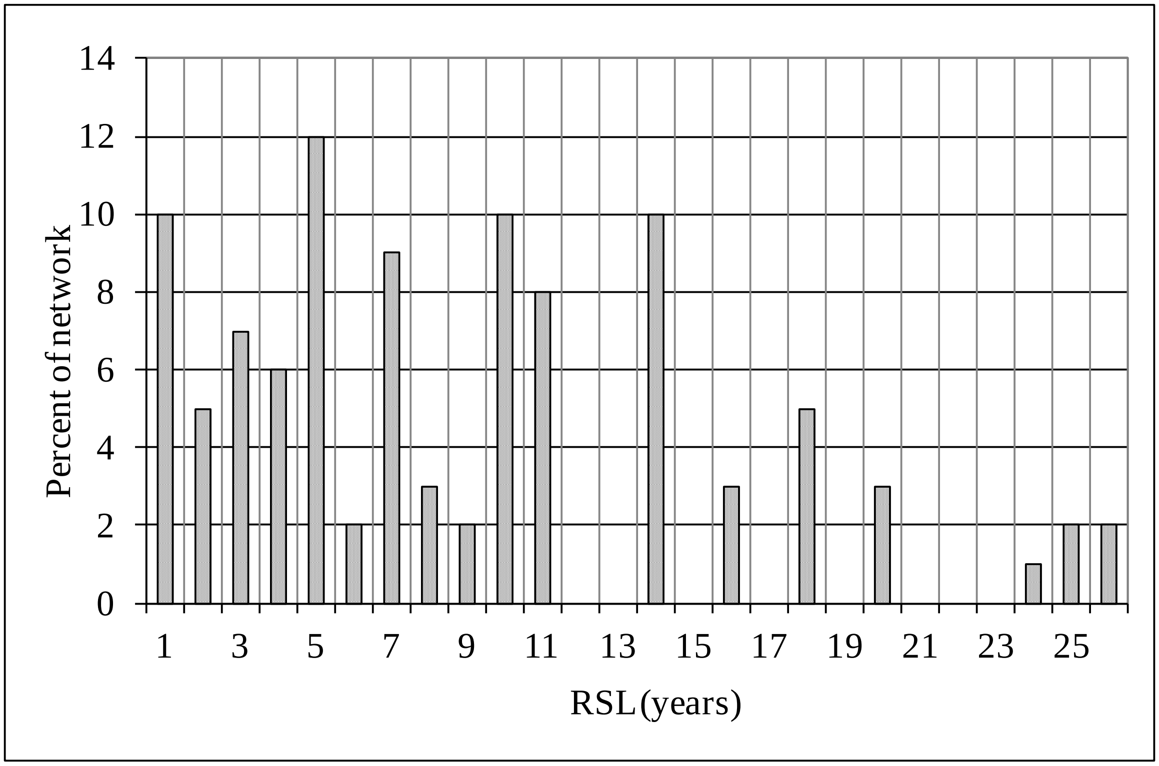


Figure 2.18 RSL distributions for a pavement network (Baladi et al. 2008)

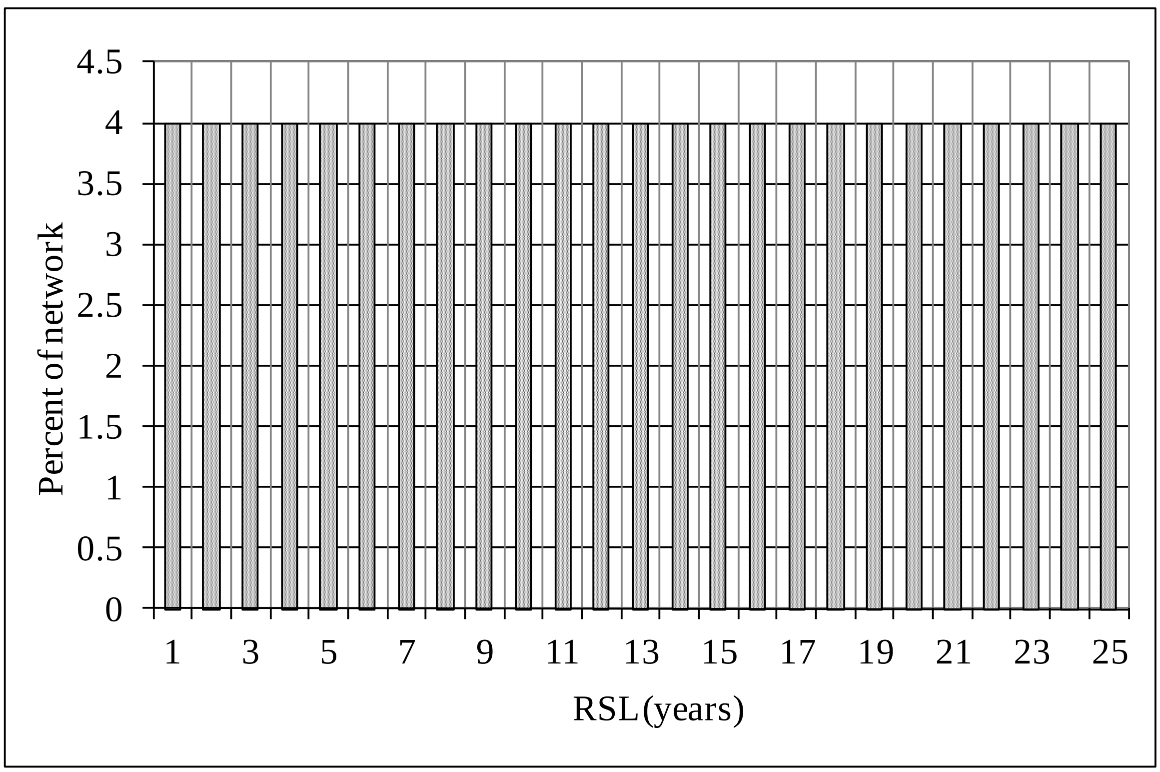


Figure 2.19 Uniform RSL distribution of a pavement network (Baladi et al. 2008)

1. The budget is adequate and the RSL is constant (status quo).
2. The budget is inferior, RSL is decreasing, and the network is deteriorating.
3. The budget is appropriate to achieve the agency goal of increasing the longevity of the pavement network by a specified number of years.

The RSL is a good communication tool regardless of the audience. Since the publication of the RSL concept and algorithm in 1992, more than 15 SHAs calculate the RSL of pavement segments and the RSL of the network. Finally, the RSL can be used as an indicator of the transportation asset value and its rate of depreciation (Baladi 2007).

The remaining life (RL) of a pavement segment is the predicted time, in years, between now and the time at which the pavement needs to be reconstructed. Hence the RL of a pavement segment constitutes one or more RSL values depending on the number of treatments that are planned to be taken in the future before the pavement segment will be reconstructed. Since the value of the RL is dependent on several future actions, its accuracy is less than that of the RSL (Baladi et al. 1985).

2.5.3.2 Theoretical and Actual Trends in RSL

Theoretically, the RSL decreases by one year for every elapsed year. In other words, a pavement segment having 10 years RSL this year will have 9 years RSL next year, 8 years RSL the year after that, and so on. The actual RSL of a pavement section will follow this trend if:

1. The measured condition and distress data are accurate, consistent from one year to the next, and have minimal variability.
2. The mathematical function used to model the data is accurate.

In reality, pavement condition and distress data vary from one year to the next due to various reasons including:

1. Variability in the measurement time, sensor locations along the pavement, and pavement temperature.
2. Measurement errors due to equipment calibration.
3. The accuracy of the location reference system used to reference the data. The linear location reference system used by most SHAs is accurate within 500 ft.
4. For image based data, the variability of the estimation of crack severity levels (crack lengths and widths) between the various surveyors and even of the same surveyor from one year to the next.
5. The sampling procedure used by the SHA.
6. The robustness of the quality control and quality assurance procedures used.

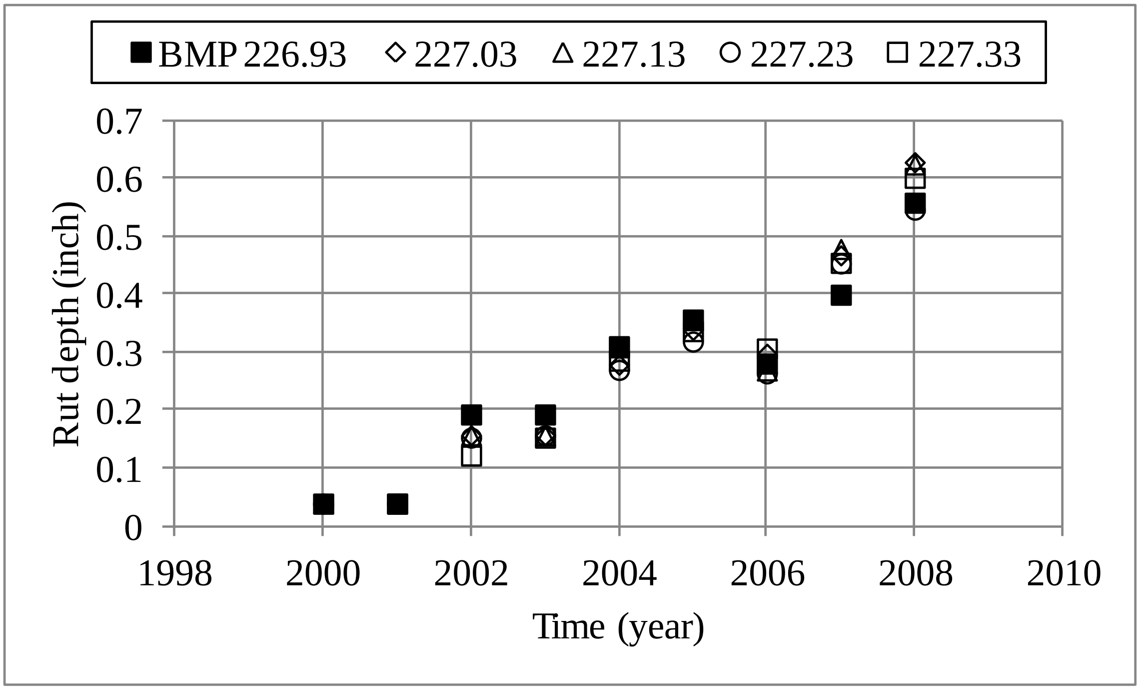
The variability of the pavement condition and distress data manifests itself in the variability of the RSL over time. The more accurate and less variable are the pavement condition and distress data, the closer the RSL will be to the theoretical trend. Figure 2.20 depicts rut depth data along five 0.1 mile long flexible pavement segements (226.93, 227.03, 227.13, 227.23, and 227.33) along State Rout Identification (SRID) 090 in Washington and the deviation of the calculated RSL from the line of equality based on the measured rut depth data along beginning mile point (BMP) 226.93. The RSL was calculated over time based on rut depth threshold values of 0.4, 0.5, and 0.6 inch. The data in the figure indicate that, in general, the RSL follows the three theoretical solid lines indicated in the figure. In another example, Figure 2.21 shows data along five 0.1 mile long flexible pavement segments (263.48, 263.58, 263.68, 263.78, and 263.88) along SRID 005 in Washington and the deviation of the calculated RSL from the line of equality based on the measured rut depth data along BMP 263.48. The RSL was calculated over time based on rut depth thresholds of 0.3, 0.4, and 0.5 inch. With the exception of the RSL calculated for year 5 (2003), the

RSL follows the three theoretical solid lines inicated in the figure. The RSL calculated for year 5 is obviously different from those calculated for the other years and indicates an issue in the time-series data. This anomaly in year 5 is easily identified in Figure 2.21 a where the rut depth drops from about 0.1 to 0.05 inch and then increases to about 1.8 inch from years 4 to 6. This anomaly could have been caused from many things, such as an un-reported treatment, equipment error, data manipulation or storage error, etc. The important point is that the data anomally was identified by studying the linearity of the RSL over time, and that this tool could be used to perform quality control on the time-series pavement condition and distress data. Real-time or early identification of anomalies in the collected data would allow for investigation and correction of the errors before the erroneous data are used in the analyses (Baladi et al. 2011).

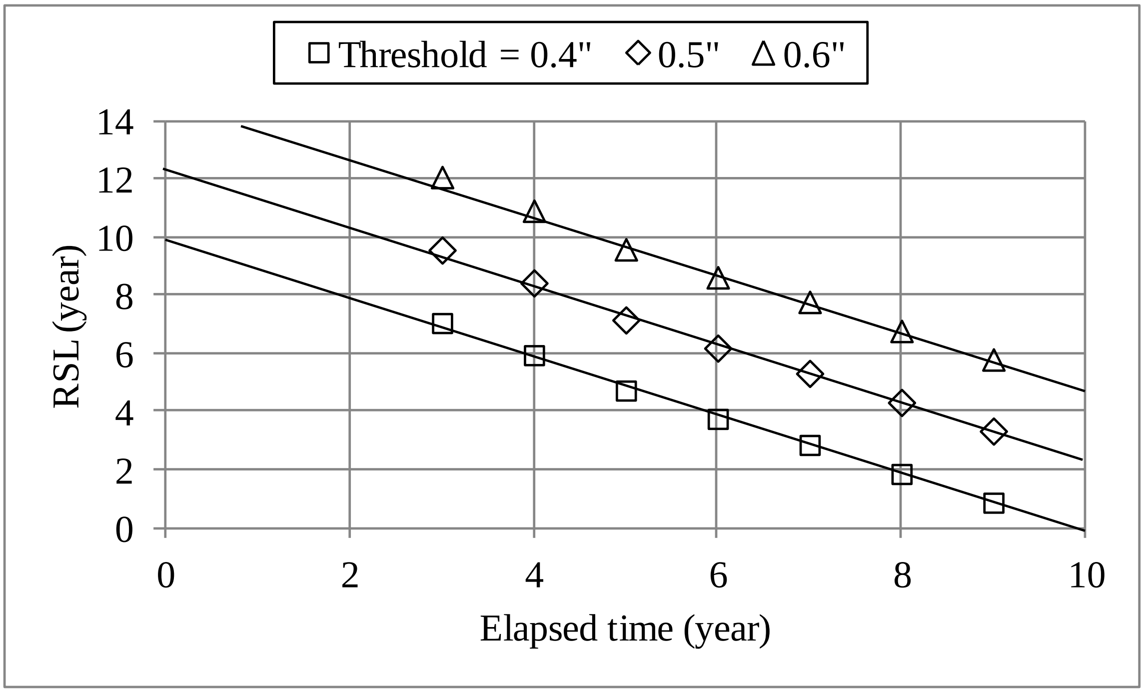
2.5.3.3 Pavement Project Ranking and Prioritization

After dividing the pavement network into uniform sections based on the delineation of the condition and distress data, the various uniform sections (pavement projects) are ranked and prioritized. The ranking and prioritization methodologies involve the assessment of several variables affecting the costs and benefits of each project. Table 2.10 lists the various processes, methods, and analytical tools available to assist SHA personnel in ranking and prioritizing pavement projects. A given project may be preferred over another for various reasons, including (Cambridge Systematics et al. 2005):

- Low agency and users costs
- Increased safety
- Decreased congestion
- Political reasons
- Better pavement network improvement

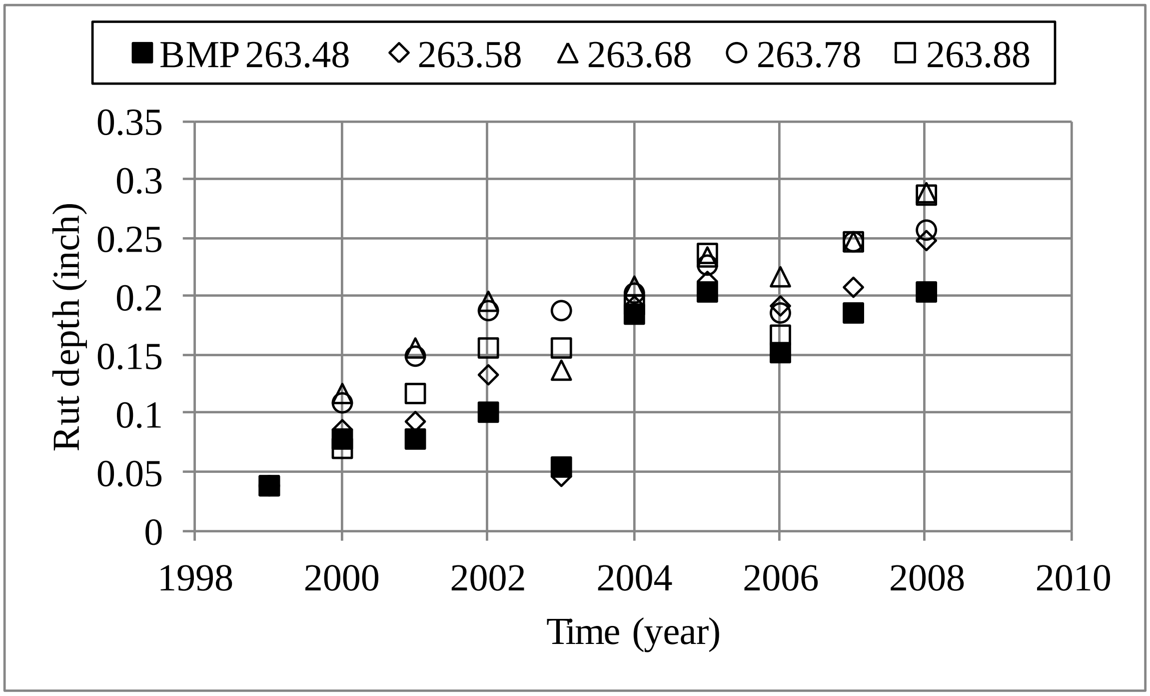


a) Measured rut depth data versus elapsed time BMP 226.93 to BMP 227.33

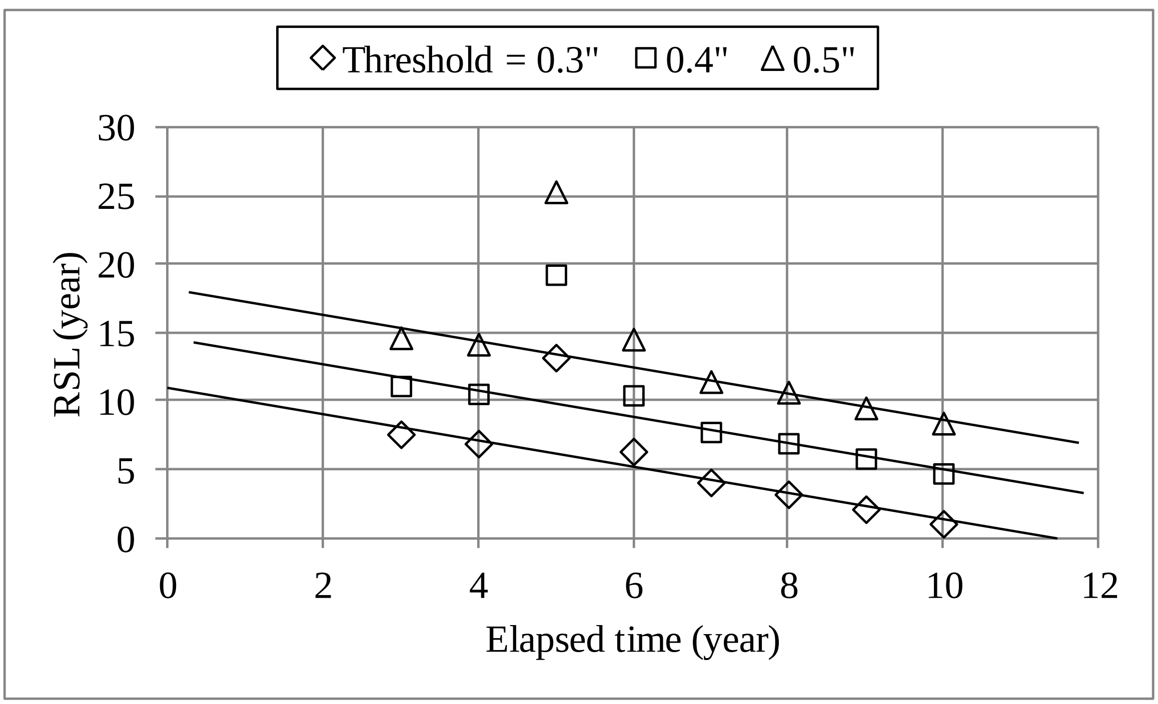


b) Calculated RSL values versus elapsed time BMP 226.93

Figure 2.20 Measured rut depth data and calculated RSL values versus elapsed time, SRID 090, flexible pavement, Washington



a) Measured rut depth data versus elapsed time BMP 263.48 to BMP 263.88



b) Calculated RSL values versus elapsed time BMP 263.48

Figure 2.21 Measured rut depth data and calculated RSL values versus elapsed time, SRID 005, flexible pavement, Washington

- Road class
- Pavement type

Table 2.10 Methods and analytical support tools for ranking and prioritization (Cambridge Systematics et al. 2005)

Process	**Method**	**Analytical tool**
Evaluate investment levels and tradeoffs	"Back-of-the-envelope analysis of budget level versus output"	"Queries to database with average costs per unit of output (e.g., miles of resurfacing, square feet of deck area for bridge replacement)"
	"Bottom-up method: identify projects within a set budget limit and estimate aggregate output and performance impacts"	"Network and sketch planning tools to assess impacts of multiple projects"
	"Optimization/ simulation – project level"	"Tools that select an optimum set of projects to meet a defined budget or performance target and that report both specific projects and aggregate costs and performance impacts of the selected projects"
	"Optimization/ simulation – network level"	"Tools to analyze performance versus cost tradeoffs at an aggregated level (not location- specific)"
Identify needs and solutions	"Informed engineering judgment"	"Database and GIS queries of condition and performance"
	"Application of standards, warrants, or rules of thumb for deficiencies and preferred solutions"	"Automated identification of deficiencies and solutions based on inventory and inspection data"
		"Database and GIS queries of deficiencies based on standards"
	"Simulation/Optimization"	"Automated identification of deficiencies and solutions, and recommendation of preferred solution based on economic criteria or decision rules"

Table 2.10 Cont'd

Process	Method	Analytical tool
Evaluate and compare options	"Informed engineering judgment"	"Queries of " knowledge base" on strategy costs and impacts"
		"Template to display "guesstimates" of strategy costs and impacts"
	"Life-cycle cost analysis"	"Queries of specialized database(s) with average costs and service lives for different strategies"
		"Simulation of alternative activity profiles over time"
		"Automated calculation of equivalent uniform annual cost, net present value"
	"Benefit/cost analysis"	"Queries of specialized database(s) with average costs and impacts for different strategies"
		"Automated calculation of strategy impacts, benefits, and costs"
	"Multi objective ranking"	"Automated calculation of strategy rating/ranking given set of objectives, performance measures, weights, and impacts"
	"Multi objective impact table"	"Queries of specialized database(s) with average costs and impacts for different strategies"
		"Tools to predict likely impacts of different strategies (e.g., network models, sketch-planning tools)"
		"Template to display strategy impacts for consistent set of performance measures"

Since, for all practical purposes, the annual budget of a given SHA is relatively consistent from one year to the next, it would be beneficial to have relatively consistent work load from one year to the next. This could be achieved if the surface conditions and distresses of the pavement network are evenly distributed across the various condition states. Hence, the goal of the SHA is to arrive at a pavement treatment strategy that would utilize the available budget and achieve uniform condition (Baladi et al. 2008). Thus, the selected ranking and prioritization procedures must be compatible with the goal of the SHA.

2.5.4 Pavement Preservation

In the past, some SHAs allowed their pavement assets to deteriorate to levels requiring major rehabilitation or reconstruction for many years. Their treatment policies were based on "worst-first" while the rest of the pavement network is deteriorating. Recently, many SHAs have initiated and implemented pavement preservation programs at the network level. The programs are based on cost-effective treatment of sections of the pavement network in relatively good condition to restore their conditions, decrease their rates of deterioration, and enhance the safety of the motorists. Over time, the preservation program becomes a part of the annual pavement treatment strategy of the SHA (Geiger 2005). The pavement preservation program typically consists of light pavement treatments such as crack sealing, non-structural overlay, light rehabilitation actions, mill and fill, and so forth. The alternative to pavement preservation is the old practice of letting the pavement network deteriorate until rehabilitation or reconstruction actions are necessary. Several studies have been conducted on the effectiveness of pavement preservation and are summarized in the next few subsections.

2.5.4.1 Pavement Preservation Cost-Effectiveness at the Project Level

The cost-effectiveness of pavement preservation at the project level can be quantified using life cycle cost analysis (LCCA). The analysis could be conducted on the various alternative pavement preservation treatments which could be applied to a pavement section over time and on the do-nothing scenario followed by reconstruction. Comparing the results of the analyses of the various strategies, indicate the cost savings or extra expenditures of performing preservation over the life of the pavement segment.

The cost-effectiveness of pavement preservation, at the project level, has been well documented. Most literature agree that pavement preservation can be conducted at minimal cost and create large savings over the life of the pavement. One study found that the cost savings of pavement preservation was as high as five dollars saved for every dollar spend on preservation (Construction and Maintenance Fact Sheets 2000). Another reports savings of 4 to 10 dollars for every dollar spend on preservation (Baladi et al. 2002). Other benefits include improving ride quality and creating a pavement network with consistently even needs from year to year (Adams & Kang 2006).

2.5.4.2 Pavement Preservation Effectiveness at the Network Level

The effectiveness of pavement preservation at the network level is more difficult to quantify than at the project level. Funds designated to preservation reduce the amount of funds available for rehabilitation and reconstruction. In other words, pavement preservation is thought to decrease the life cycle cost of a given pavement project, but the effect on the network is often unknown. Additionally, the public and the legislators may not understand why pavements in seemingly good condition are being treated while others in poor condition are not. The SHAs should document and communicate the effects of preservation maintenance on the health of the pavement network and on the life cycle cost in a clear and consistent mean. Educating the public and the legislature is necessary to establish and maintain a successful pavement preservation program (Adams & Kang 2006).

The short and long term benefits and effectiveness of pavement preservation at the project and network levels should be quantified. Short term benefits include improving ride quality and addressing safety issues, while long term benefits (cost savings) are not realized until years or decades into the future. Therefore, pavement preservation strategies must be optimized through projection of needs and funds into the future. In this way the effects of performing or deferring various scenarios of pavement projects can be evaluated (DelDOT 2001, Adams & Kang 2006).

2.5.5 Pavement Performance Prediction

The performance of a pavement segment is often illustrated by the progression of pavement condition or distress over time, such as that shown in Figure 2.22. The level of performance at any given time is equivalent to the level of pavement condition or distress at that time relative to the threshold value. Therefore, the performance of a pavement segment over its service life is defined by the level of service over time or by the accumulation of damage over time (Chatti et al. 2005).

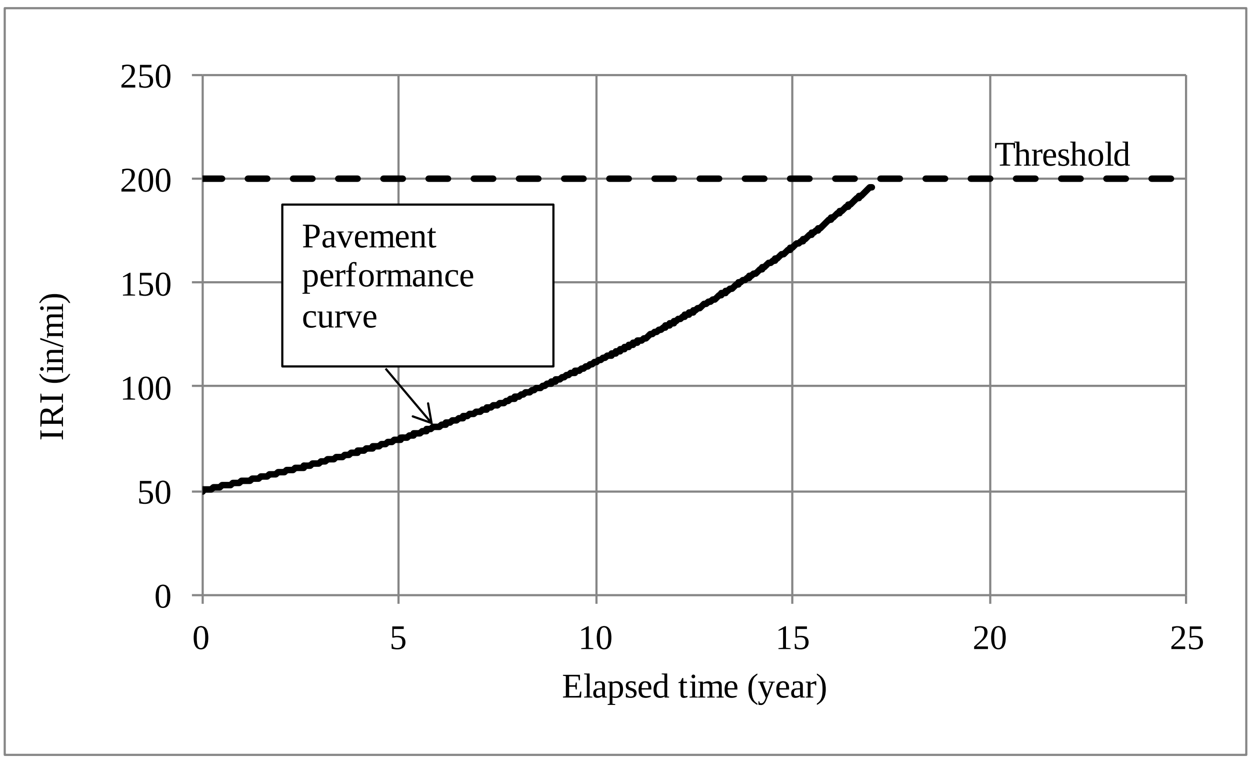


Figure 2.22 Typical pavement performance curve

Most SHAs collect pavement condition and distress data. Some use the data to observe the condition of the pavement, while others use the time-series pavement condition and distress data to predict future pavement conditions. The combination of both practices allows for the development of current and future strategies for management of the pavement network.

Many SHAs have studied the effectiveness of various pavement treatments using historical pavement performance data. Based on the various studies, the minimum and maximum treatment lives listed in Table 2.11 were published in the various sources listed for each treatment type. In this study, the average treatment lives for thin (< 2.5 inch) and thick (≥ 2.5 inch) HMA overlay, single and double chip seal, and thin (< 2.5 inch) and thick (≥ 2.5 inch) mill and fill treatments were used. These estimated averages are adequate to be used in the analysis at the network level. For project level analysis, more accurate estimates are required, as determined in the analyses of Chapter 4. The service lives for cold-in-place asphalt concrete recycling, crack sealing, and micro-surfacing listed in Table 2.11 are provided for reference but were not used in this study, as indicated in the table.

Several predictive pavement performance models have been developed to estimate the pavement performance curve based on parameters such as traffic, weather, and pavement type. The various methods include straight-line extrapolation, regression, polynomial constrained least squares, application of S-shaped curves, use of probability distributions, and Markov chain models (Ahmed et al. 2006). One such example, for thin HMA overlays is presented in Equation 2.4. The β parameters were determined for different performance indicators (IRI, PCR, and Rut) as well as different road types (Interstate, non-Interstate highway, and non-highway) (Irfan et al. 2009).

$$PI = e^{\beta_1 + \beta_2 * CAATT + \beta_3 * CAFDX} \qquad \text{Equation 2.4}$$

Where, PI is the value of the performance indicator (IRI, PCR, RUT) for a pavement segment in a given year;

Table 2.11 Estimated and reported pavement treatment life

Treatment type	References	Estimated treatment service life expectancy (year)		
		Minimum	Average	Maximum
Thin (< 2.5 inch) HMA overlay	Geoffroy 1996, Hicks et al. 2000, Johnson 2000, ODOT 2001, Wade et al. 2001, MDOT 2001, Peshkin et al. 2004	2	8	12
Thick (≥ 2.5 inch) HMA overlay	FHWA 2010	6	10	17
Single chip seal	Geoffroy 1996, Hicks et al. 2000, Johnson 2000, Bolander 2005, Gransberg & James 2005	1	6	12
Double chip seal	Hicks et al. 2000, Johnson 2000, MDOT 2001, Bolander 2005, Maher et al. 2005	4	9	15
Thin (< 2.5 inch) mill & fill	MDOT 2001, FHWA 2010	4	8	20
Thick (≥ 2.5 inch) mill & fill	FHWA 2010	6	10	17
Cold-in-place HMA recycling*	Hicks et al. 2000, Morian et al. 2004, Maher et al. 2005	5	10	20
Crack sealing*	Geoffroy 1996, Johnson 2000	2	3	10
Micro-surfacing*	Geoffroy 1996, Smith & Beatty 1999, Johnson 2000, Wade et al. 2001, Peshkin et al. 2004, Labi et al. 2006	4	6	10
* = Treatment type not used in this study				

CAATT is the cumulative average annual daily truck traffic (in millions) experienced by the pavement segment from the time of treatment to the given year;

CAFDX is the cumulative annual freeze index (in thousands) experienced by the pavement segment from the time of treatment to the given year (degree-day);

β_1, β_2, and β_3 are statistical parameters

The most common method for fitting models (equations) to pavement condition and distress data is by ordinary least squares regression. It should be noted that a minimum of three data points are required to model the data. The method is used to determine the parameters of the selected mathematical function, such as those listed in Table 2.12 (M-E PDG 2004), by minimizing the sum of squared errors. This method works when the particular pavement segment is deteriorating following the selected model. However, when the model does not capture the true progression of the condition or distress over time, other models may be required (Luo 2005).

Table 2.12 Typical pavement condition models (M-E PDG 2004)

Pavement condition type	**Model form**	**Generic equation**
IRI	Exponential	$IRI = \alpha * \exp(\beta t)$
Rut depth (inch)	Power	$\text{Rut depth} = \gamma\, t^{\omega}$
Longitudinal crack (ft)	Logistic (S-shaped)	$\text{Crack} = \frac{k}{1 + \exp[-\theta(t - \mu)]}$
Where, α, β, γ, ω, k, θ, and μ are regression parameters and t = elapsed time (year)		

Another method of modeling pavement condition and distress data is the cluster wise regression procedure, which was introduced by (Spath 1979, 1981, 1982), and was modified by (Meier 1987, DeSarbo et al. 1989, Lau et al. 1999, and Luo 2005). Cluster wise regression involves splitting the data into sub-groups based on their characteristics and fitting separate models to each sub-group. The resulting pavement performance models can be much more accurate as they model small subsets of data with similar trends.

2.5.6 Optimum Treatment Timing

Performing pavement treatments at the optimum time will provide the greatest benefit-to-cost ratio. The idea of optimum timing is not new, in fact, the concept was built into the AASHTO 1993 design guide. Few methodologies for the determination of optimum treatment timing for preventive maintenance and rehabilitation actions were developed. Two of these methods are presented below.

2.5.6.1 Analysis of the Optimum Treatment Time Embedded in the AASHTO 1993 Design Guide

One of the most widely used pavement design methodologies is that based on the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO 1993). The AASHTO Guide is used for the design of flexible and rigid pavements in the USA and in various other parts of the world. The guide could also be utilized to select pavement treatment timing by providing for pavements to be treated when some service life remains and before the threshold conditions or distresses have been reached. The preliminary analyses outlined in this section were conducted to explore the optimum treatment timing developed and implemented as part of AASHTO guide. Although not commonly referenced, the AASHTO likely developed the first optimum treatment timing methodologies which have served as a stepping stone to the analyses of this research, presented in Chapter 4.

The inputs to the 1993 AASHTO Design Guide include a pavement analysis and a pavement performance period. Two options are available to the users as follows:

1. The analysis and performance periods are the same. In this case, the design guide yields a pavement cross-section that will serve the anticipated traffic level during the specified performance period, as shown in Figure 2.23. At the end of this period, the pavement should be subjected to major rehabilitation or reconstruction.
2. The analysis period is longer than the performance period. This option is typically used for stage construction for estimating the required thickness of the overlay that needs to be applied at the end of the first performance period, as shown in Figure 2.23.

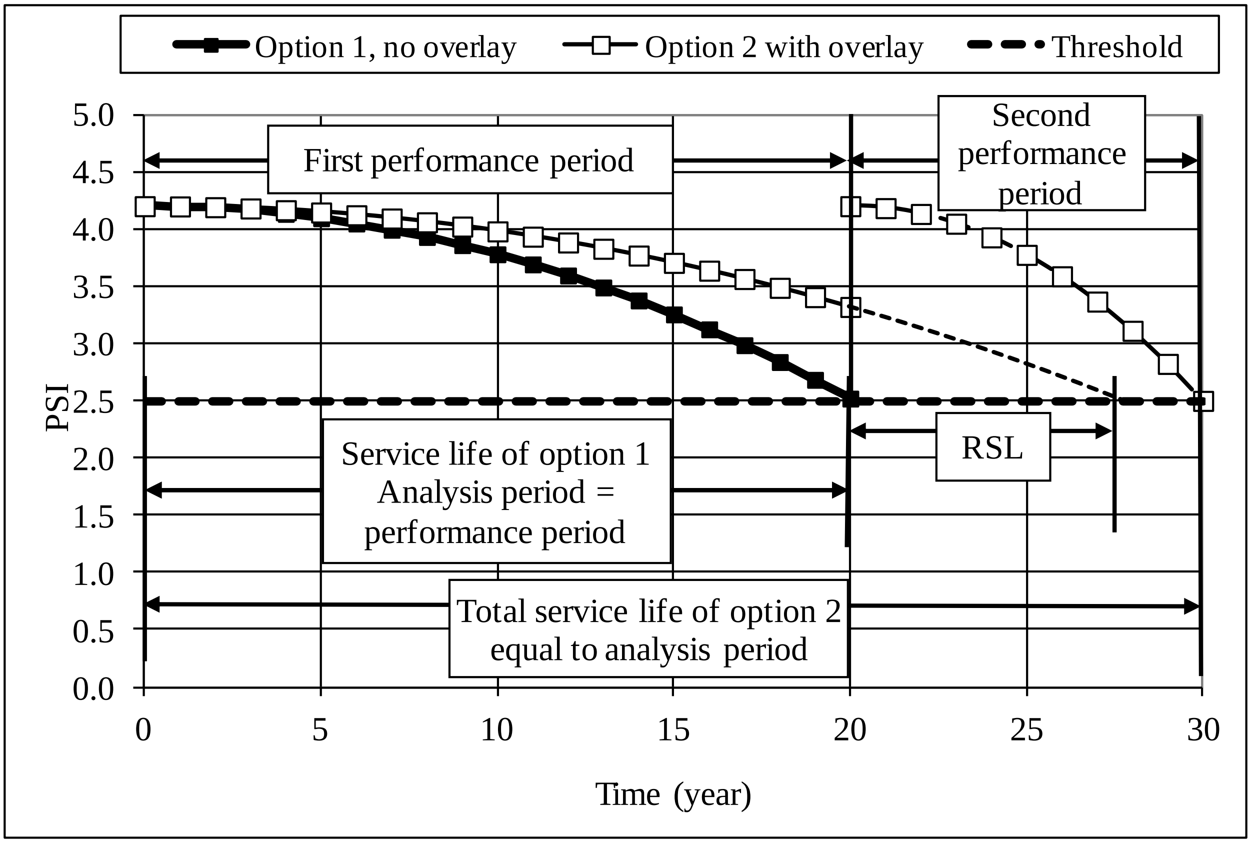


Figure 2.23 Analysis and performance periods

Analysis Methodology

The above stated options of the AASHTO Design Guide were used to investigate their impact on the timing of pavement rehabilitation. To accomplish this, flexible and rigid pavement cross-sections were designed using the input variables listed in Table 2.13. The resulting pavement cross-sections (layer thicknesses) are listed in Table 2.14. The two pavement cross-sections were then redesigned using the same input parameters except that the analysis period was changed from 20 to 30 years (option 2) creating two performance periods (first and second). In the analyses, the thicknesses of the base and subbase layers were fixed as those obtained in option 1. The 1993 AASHTO design procedure yielded the asphalt and concrete thicknesses listed in Table 2.14.

Table 2.13 Input parameters for the design of flexible and rigid pavement sections, option 1

Design inputs	Flexible pavement		Rigid pavement	
	Option 1	Option 2	Option 1	Option 2
Analysis period (yrs.)	20	30	20	30
Performance period (yrs.)	20	20	20	20
Yearly one directional ESAL	250,000	250,000	250,000	250,000
Design reliability (%)	95	95	95	95
Standard deviation	0.45	0.45	0.35	0.35
Initial PSI	4.2	4.2	4.2	4.2
Terminal PSI	2.5	2.5	2.5	2.5
Roadbed MR (psi)	5,000	5,000	5,000	5,000
HMA Elastic modulus (psi)	450,000	450,000	-	-
HMA layer coefficient	0.42	0.42	-	-
PCC Elastic modulus (psi)	-	-	4,200,000	4,200,000
PCC Modulus of rupture (psi)	-	-	690	690
Base Elastic modulus (psi)	30,000	30,000	-	-
Base layer coefficient	0.12	0.12	-	-
Subbase Elastic modulus (psi)	15,000	15,000	15,000	15,000
Subbase layer coefficient	0.1	0.1	-	-
Subbase thickness (inch)	-	-	12	12
Drainage coefficient (all layers)	1	1	1	1
Load transfer coefficient	-	-	3.2	3.2
Loss of support factor	-	-	0	0

Table 2.14 The AASHTO design outputs (layer thicknesses) for flexible and rigid pavement sections using option 1 (20 years design and performance periods)

Performance period	Layer	Layer thickness (inch)			
		Flexible pavement		Rigid pavement	
		Option 1	Option 2	Option 1	Option 2
Original layer thicknesses	HMA	6.75	7.33	-	-
	PCC	-		8.99	9.38
	Base	7.16	7.16	-	-
	Subbase	8.50	8.50	12	12
HMA overlay thickness (inch)		-	3.36	-	0.85

Examination of the HMA and the PCC thicknesses for options 1 and 2 indicate that as the analysis period increases from 20 to 30 years:

1. The original HMA thickness increases from 6.75-inch to 7.33-inch and at the end of the first performance period of 20 years, 3.36-inch of HMA overlay must be applied in order to extend the service life of the pavement to 30 years over the 10 year second performance period.
2. The PCC thickness increases from 8.99 to 9.38-inch at the end of the first performance period of 20 years, 0.85-inch HMA overlay must be applied in order to extend the service life of the pavement to 30 years over the 10 year second performance period.

The pavement sections of option 1 with HMA thickness of 6.75-inch and PCC thickness of 8.99-inch would serve the traffic for a 20 year period as the Pavement Serviceability Index (PSI) deteriorates from its initial value (see Table 2.13) to its terminal value of 2.5. Whereas the PSI of the pavement sections of option 2 with an HMA thickness of 7.33-inch and PCC thickness of 8.99-inch would deteriorate from their initial values to a value higher than the terminal value of 2.5. Stated differently, the conditions of the pavement sections of option 1 at the end of the first performance period are worse than those of option 2 when the overlays will be placed. In fact the RSL is about 38% and 28%, respectively, of the first performance period when the overlays are applied for the flexible and rigid pavements of option 1. These results imply that the concept of treating pavements at an early stage is embedded in the 1993 AASHTO design procedure.

Results of the 1993 AASHTO design of the flexible and rigid pavement sections of option 2 were used to determine the RSL of the two sections at the time when the overlays will be applied. This was accomplished using trial and error approach. This approach consisted of the following steps:

1. Two pavement sections (one flexible and one rigid) were designed using the same input parameters as those listed in Table 2.13 and the same performance and analysis periods.
2. Two pavement sections (one flexible and one rigid) were redesigned using the same input parameters except that the analysis period was changed from 20 to 30 years (option 2).
3. The performance and analysis periods were gradually and equally increased, from 20, until the AASHTO design procedure yielded the same pavement sections as those of the pavement sections of step 2.
4. The RSL of each pavement section was then calculated as the difference between the last performance period (step 3) and 20 years. The obtained RSL values of 7.6 years for the flexible pavement section and 5.7 for the rigid section are listed in Table 2.15.

Table 2.15 The AASHTO design outputs for flexible and rigid pavements, options 1 and 2

Pavement type	Analysis period (year)	First performance period (year)	HMA thickness (inch)		RSL at the end of the first performance period (year)
			Original section	Overlay	
Flexible	20	20	6.75	0.0	0.0
	30	20	7.33	3.36	7.6

Pavement type	Analysis period (year)	First performance period (year)	PCC thickness (inch)		RSL at the end of the first performance period (year)
			Original section	Overlay	
Rigid	20	20	8.99	0.0	0.0
	30	20	9.38	0.85	5.7

The implication of the above analysis is that the original pavement structure needs to be in better conditions in order to support the overlay which will carry the additional ten years traffic. Further, the original AASHTO cross-section design of the two pavement sections of option 2 (the pavement sections that will be overlaid at the end of the 20-year performance period) yielded flexible and rigid pavement sections that would last 27.6 and 25.7 years, respectively if no overlay was applied. Hence, at the end of the 20 year performance period, the RSL of the flexible and rigid sections are 7.6 and 5.7 years, respectively.

Additional analyses were conducted to explore the effects of the other design inputs (Equivalent Single Axle Load (ESAL), roadbed modulus, design reliability, and so forth) on the RSL of flexible and rigid pavement sections. Results of these analyses are presented in the next few subsections.

Analyses of the Effects of Design Variables on Remaining Service Life

In these analyses, the values of several design inputs were changed, one at a time, while keeping all other variables at constant values to study the effects, if any, of each variable on the pavement cross-section. The objective of the analyses was to determine the time (corresponding to condition level) at which the methodology of the AASHTO 1993 design guide calls for conducting rehabilitation activities. The difference in time between the pavement age when applying an overlay and the performance period for the original pavement section is the RSL of the pavement at the time of the overlay.

ESAL

These analyses considered the effect of traffic level (ESAL) on the RSL. The analyses were conducted using three initial year ESAL values of 50,000, 250,000, and 1,000,000 ESAL. The results, listed in Table 2.16, indicate that regardless of the ESAL level, the RSL, at the time of rehabilitation, remains 7.6 and 5.7 years for flexible and rigid pavements, respectively.

Roadbed soil modulus

These analyses considered the effect of the roadbed soil Resilient Modulus (MR) on the RSL. The analyses were conducted using three MR values of 5,000, 10,000, and 20,000 psi. The

Table 2.16 Effect of ESAL on the RSL of flexible and rigid pavement sections

Pavement type	Yearly ESAL	Analysis period (year)	First performance period (year)	Difference between analysis and performance periods (year)	RSL at the end of the first performance period (year)
Flexible	50,000	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
	250,000	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
	1,000,000	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
Pavement type	Yearly ESAL	Analysis period (year)	First performance period (year)	Difference between analysis and performance periods (year)	RSL at the end of the first performance period (year)
Rigid	50,000	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7
	250,000	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7
	1,000,000	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7

results, listed in Table 2.17, indicate that regardless of the roadbed soil MR value, the RSL, at the time of rehabilitation, remains 7.6 and 5.7 years for flexible and rigid pavements, respectively.

Overlay design life

These analyses considered the effect of the second performance period on the RSL. The analyses were conducted using three second performance period lengths of 5, 10, and 20 years. The results, listed in Table 2.18, indicate that regardless of the overlay design life, the RSL, at the time of rehabilitation, remains 7.6 and 5.7 years for flexible and rigid pavements, respectively. As expected, the overlay thickness does increase with increasing analysis period.

Table 2.17 Effect of roadbed soil modulus on the RSL of flexible and rigid pavement sections

Pavement type	Roadbed modulus (ksi)	Analysis period (year)	First performance period (year)	Difference between analysis and performance periods (year)	RSL at the end of the first performance period (year)
Flexible	5	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
	10	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
	20	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
Pavement type	**Roadbed modulus (ksi)**	**Analysis period (year)**	**First performance period (year)**	**Difference between analysis and performance periods (year)**	**RSL at the end of the first performance period (year)**
Rigid	5	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7
	10	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7
	20	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7

Table 2.18 Effect of overlay design life on the RSL of flexible and rigid pavement sections

Pavement type	Second performance period (year)	RSL at the end of the first performance period (year)
Flexible	5	7.6
	10	7.6
	20	7.6
Pavement type	**Second performance period (year)**	**RSL at the end of the first performance period (year)**
Rigid	5	5.7
	10	5.7
	20	5.7

Performance period

These analyses considered the effect of the performance period on the RSL. The analyses were conducted using three performance periods of 15, 20, and 25 years. The results, listed in Table 2.19, indicate that the RSL, at the time of rehabilitation, is dependent on the performance period (see Figure 2.24). However, after more careful examination, the data indicate that regardless of the performance period the ratio of RSL to performance period, at the time of rehabilitation, remains 38 and 28% for flexible and rigid pavements, respectively, as listed in Table 2.20.

Table 2.19 Effect of performance period on the RSL of flexible and rigid pavement sections

Pavement type	First performance period (year)	Analysis period (year)	RSL at the end of the first performance period (year)
Flexible	15	15	0.0
	15	25	5.7
	20	20	0.0
	20	30	7.6
	25	25	0.0
	25	35	9.4
Pavement type	First performance period (year)	Analysis period (year)	RSL at the end of the first performance period (year)
Rigid	15	15	0.0
	15	25	4.2
	20	20	0.0
	20	30	5.7
	25	25	0.0
	25	35	7.1

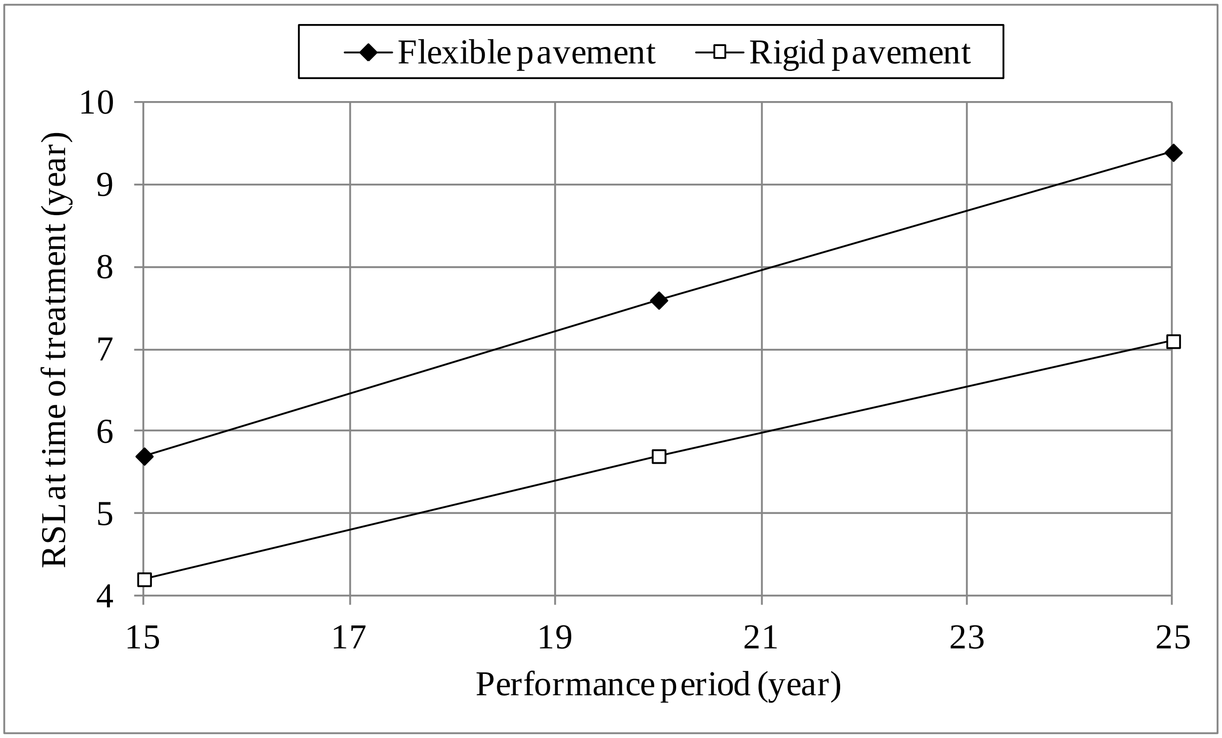


Figure 2.24 RSL at time of treatment vs. performance period

Table 2.20 Effect of performance period on the (RSL/performance period) for flexible and rigid pavement sections

Pavement type	First performance period (year)	Second performance period (year)	RSL/first performance period (%)
Flexible	15	10	38
	20	10	38
	25	10	38

Pavement type	First performance period (year)	Second performance period (year)	RSL/first performance period (%)
Rigid	15	10	28
	20	10	28
	25	10	28

Design reliability

These analyses considered the effect of the design reliability on the RSL. The analyses were conducted using four design reliability levels of 99, 95, 85, and 60%. The results, listed in Table 2.21, indicate that the RSL, at the time of rehabilitation, is dependent on the design reliability. The ratios of RSL to performance period, at the time of rehabilitation, for the various design reliabilities, of flexible and rigid pavements, are listed in Table 2.22 and shown in Figure 2.25.

Terminal serviceability at time of rehabilitation

The objective of this analysis was to determine the condition level (PSI) that corresponds to the timing of the rehabilitation action. In order to establish this, results of the 1993 AASHTO design for the flexible and rigid pavement sections of option 2 (Table 2.14) were used to determine the PSI of the two sections at the time when the overlays will be applied. This was accomplished using a trial and error approach. This approach consisted of the following steps:

1. Two pavement sections (one flexible and one rigid) were designed using the same input parameters and performance and analysis periods listed in Table 2.13.
2. Two pavement sections (one flexible and one rigid) were redesigned using the same input parameters except that the analysis period was changed from 20 to 30 years (option 2).
3. The terminal PSI value was gradually and equally increased, for 20 year performance and analysis periods, until the AASHTO design procedure yielded the same pavement sections as those of the pavement sections of step 2.

Repetition of step 3, with various performance and analysis period pairs, yields the PSI at several points during the life of the pavement sections. The results, listed in Table 2.23 and shown in Figure 2.26, indicate that the methodology applies rehabilitation actions when the PSI

Table 2.21 Effect of design reliability on the RSL of flexible and rigid pavement sections

Pavement type	Design reliability (%)	Analysis period (year)	First performance period (year)	Difference between analysis and first performance periods (year)	RSL at the end of the first performance period (year)
Flexible	99	20	20	0	0.0
		25	20	5	5.8
		30	20	10	5.8
	95	20	20	0	0.0
		25	20	5	7.6
		30	20	10	7.6
	85	20	20	0	0.0
		25	20	5	9.7
		30	20	10	9.7
	60	20	20	0	0.0
		25	20	5	13.7
		30	20	10	13.7

Pavement type	Design reliability (%)	Analysis period (year)	First performance period (year)	Difference between analysis and first performance periods (year)	RSL at the end of the first performance period (year)
Rigid	99	20	20	0	0.0
		25	20	5	4.4
		30	20	10	4.4
	95	20	20	0	0.0
		25	20	5	5.7
		30	20	10	5.7
	85	20	20	0	0.0
		25	20	5	7.3
		30	20	10	7.3
	60	20	20	0	0.0
		25	20	5	10.0
		30	20	10	10.0

is about 2.73 and 2.87 for flexible and rigid pavements, respectively. The data also indicate that the AASHTO 1993 methodology predicts a non-linear PSI deterioration curve for flexible pavements and a linear deterioration curve for rigid pavements.

Table 2.22 Effect of design reliability on the (RSL/performance period) ratio for flexible and rigid pavement sections

Pavement type	Design reliability (%)	RSL/first performance period (%)
Flexible	99	29
	95	38
	85	48
	60	68
Pavement type	Design reliability (%)	RSL/first performance period (%)
Rigid	99	22
	95	28
	85	36
	60	50

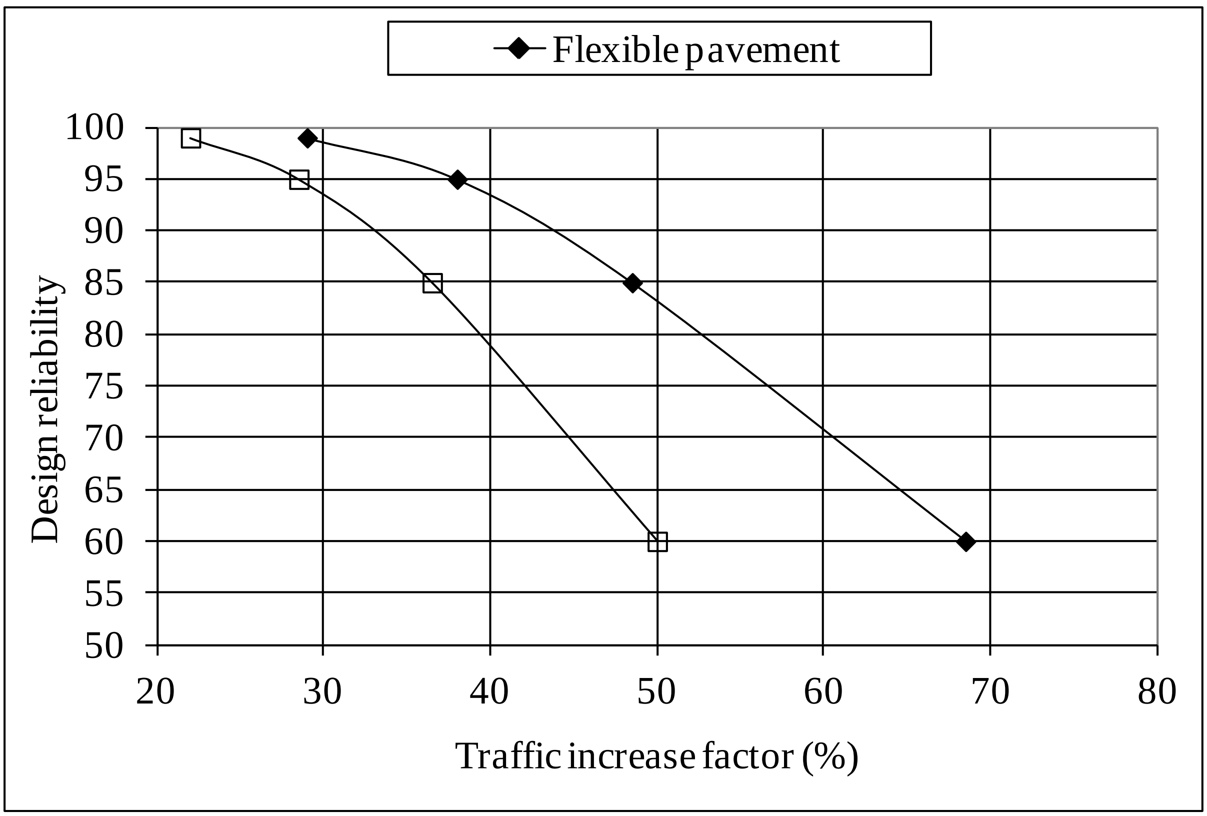


Figure 2.25 Effect of design reliability on the (RSL/performance period) ratio for flexible and rigid pavement sections

Table 2.23 Deterioration of flexible and rigid pavement sections

Pavement type	Analysis period (year)	Performance period (year)	PSI terminal
Flexible	0	0	4.2
	5	5	3.42
	10	10	3.13
	15	15	2.91
	20	20	2.73
	27.6	27.6	2.5
Pavement type	Analysis period (year)	Performance period (year)	RSL/Performance period (%)
Rigid	0	0	4.2
	5	5	3.87
	10	10	3.53
	15	15	3.20
	20	20	2.87
	25.7	25.7	2.5

The 1993 AASHTO design methodology was further used to determine if the results of this analysis are dependent on the pavement sections. The deterioration was calculated for five flexible pavement sections with structural numbers (SN) of 2.5, 3, 3.25, 4, and 6. The results of the analysis shown in Figure 2.27 indicate that the 1993 AASHTO procedure predict flexible pavement deterioration as a function of the as-constructed SN. Pavement sections with an SN value of 3.25 deteriorate linearly. Whereas, the deteriorations of thicker pavement sections (higher SN value) follow concave curves and of thinner sections convex curves. The deterioration of rigid pavements was also calculated for four pavement sections with slab thicknesses of 7, 9, 11, and 13 inches. The results, shown in Figure 2.28, indicate that the deterioration of rigid pavement sections in the 1993 AASHTO procedure is linear and independent of slab thickness.

Similar results have been reported in the literature. Flexible pavements with an SN of 3.25 follow Miner's Law and deteriorate at a constant rate. However pavements with SN

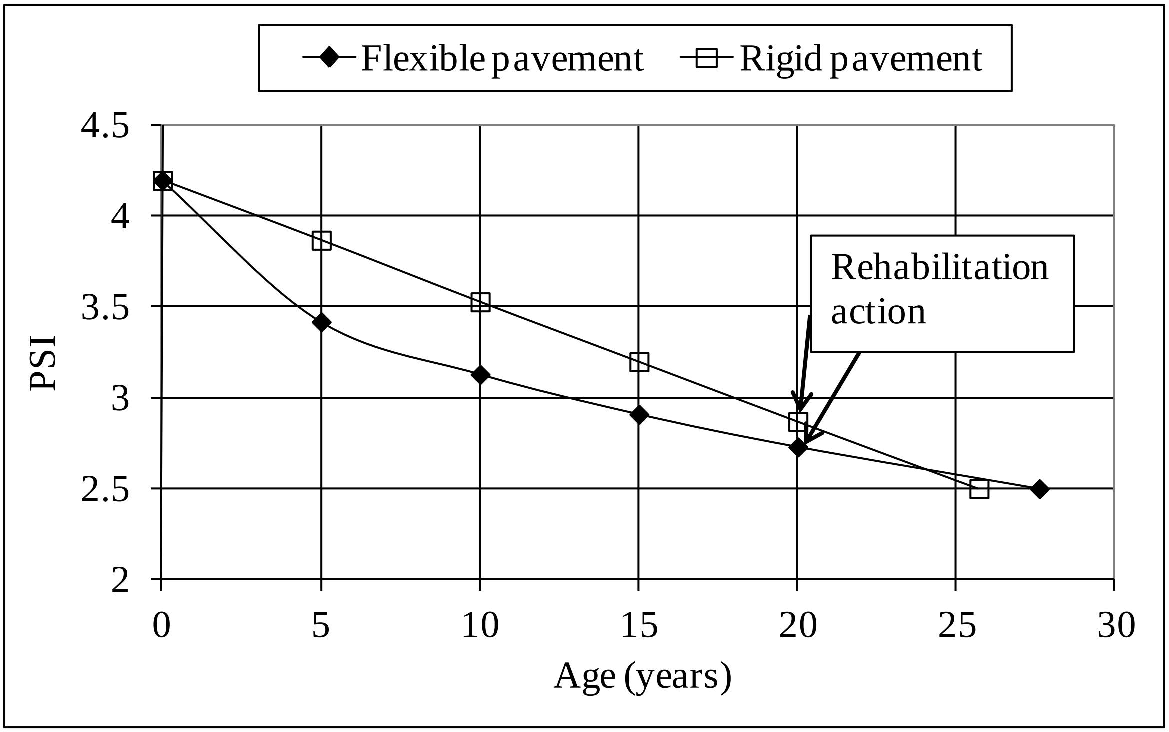


Figure 2.26 Deterioration curves for flexible and rigid pavement sections

different from 3.25 deteriorate based on the condition of the pavement. Thinner pavement sections deteriorate as typically thought of with increasing rate. However thicker pavements deteriorate with decreasing rate. These trends are based on the observations and analyses of the AASHO Road Test (Ullidtz 1987, Ullidtz 1998).

Summary

The methodology of the 1993 AASHTO Design Guide was analyzed and found to produce the same RSL to performance period ratio regardless of the design factors, with the exception of the design reliability. For design reliability of 95%; the RSL to performance period ratio are 38 and 28 percent and the PSI value at the time of rehabilitation is about 2.73 and 2.87 for flexible and rigid pavements, respectively. These values are based on the results of the AASHO Road Test and are not meant to be considered as optimum, but are presented to explain the concept of pavement preservation and RSL within the AASHTO methodology.

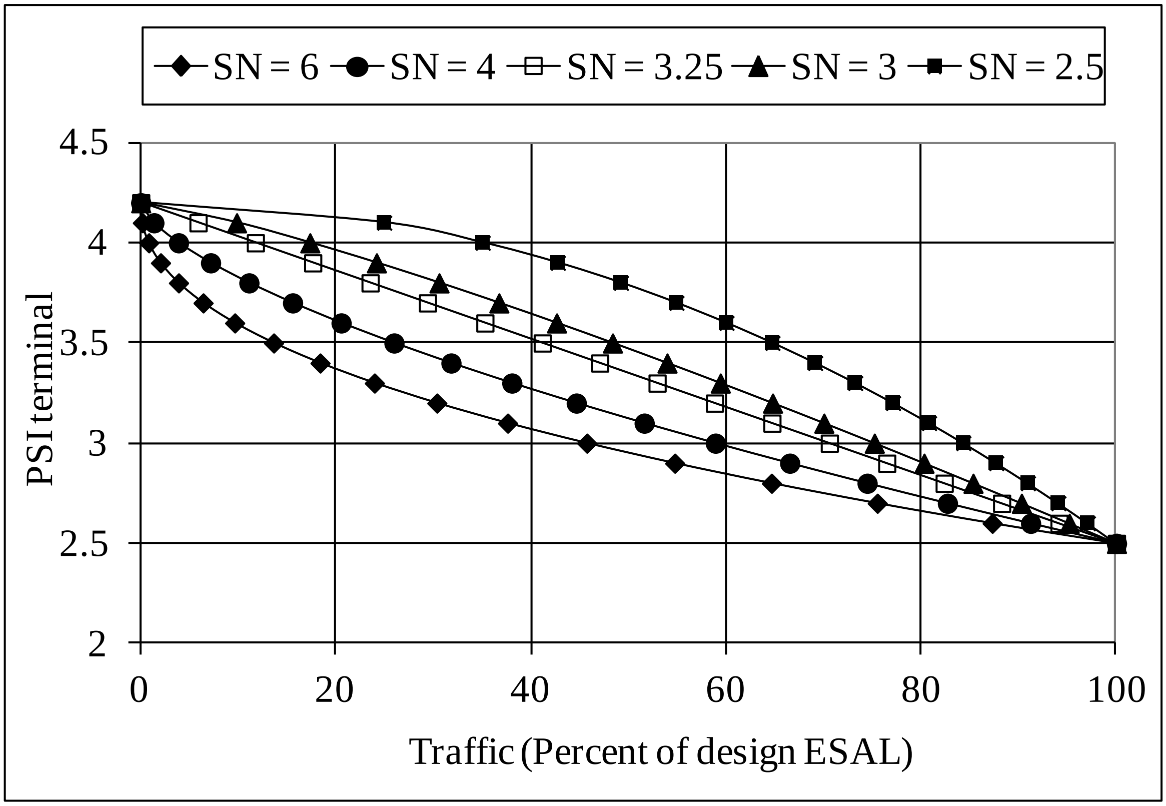


Figure 2.27 Flexible pavement deterioration curves for different SN values

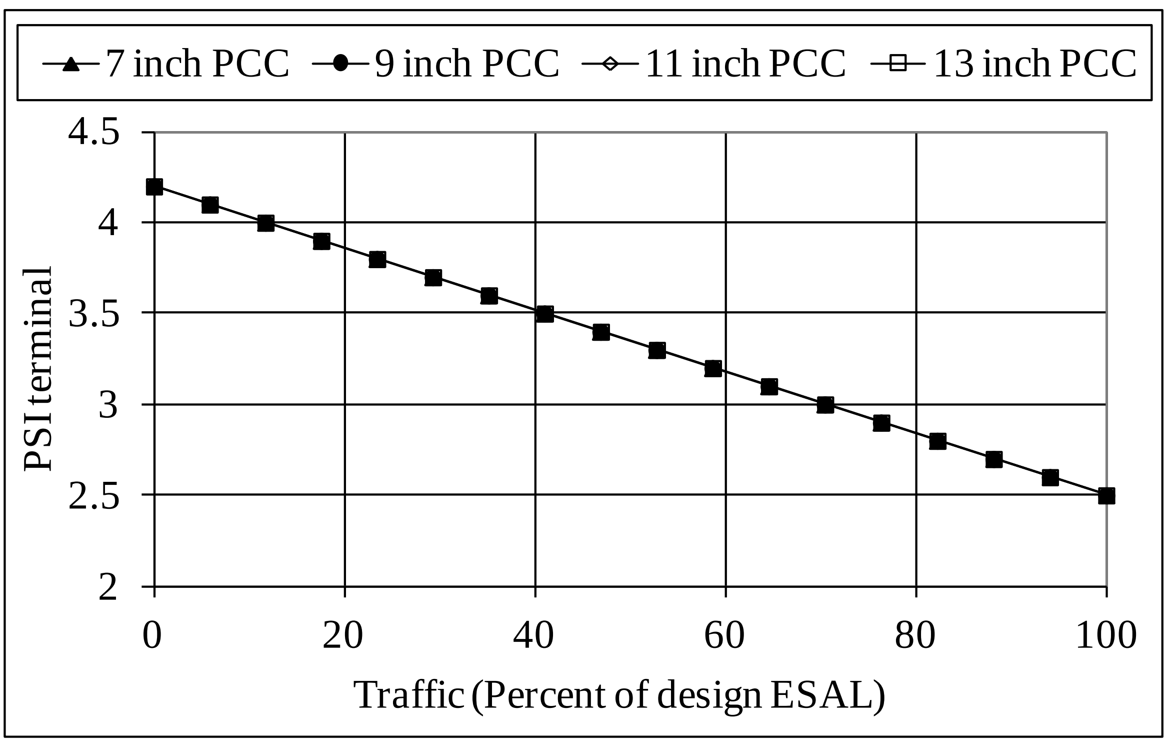


Figure 2.28 Rigid pavement deterioration curves for different slab thicknesses

2.5.6.2 NCHRP Report 523

A methodology for the selection of the optimum timing for preventive maintenance treatments on rigid and flexible pavements was developed and presented in NCHRP Report 523 (Peshkin et al. 2004). The methodology was designed to consider a wide variety and degree of pavement condition and distress types, treatments, and agency and user costs to calculate the optimum timing for a given preventive maintenance activity. A Microsoft Excel based tool OPTime was developed for the implementation of the methodology and it can be found at: http://144.171.11.107/Main/Public/Blurbs/155142.aspx (Peshkin et al. 2004). Regardless of the pavement condition or distress scale used (increasing or decreasing), the methodology calls for the selection of upper and lower cutoff values on the pavement condition or distress scale. The two values represent the range in pavement conditions or the range in time when preventive maintenance activities will be beneficial. The area between the upper and lower values and the performance curve represents the benefit, as shown in Figure 2.29 (Peshkin et al. 2004). Preventive maintenance activities conducted outside the two values are considered to be done too early in pavement service life to provide benefit.

In the methodology, the optimum timing of preventive maintenance activities is based on maximizing the "benefit" of the preventive maintenance activity over the area of the "do-nothing" case. To quantify this benefit (Peshkin et al. 2004):

1. The performance of the pavement section prior to preventive maintenance treatment must be known from the time of construction/rehabilitation to the end of the pavements serviceable life. The area between the performance curve and the lower cutoff value (the do-nothing area) is then calculated.

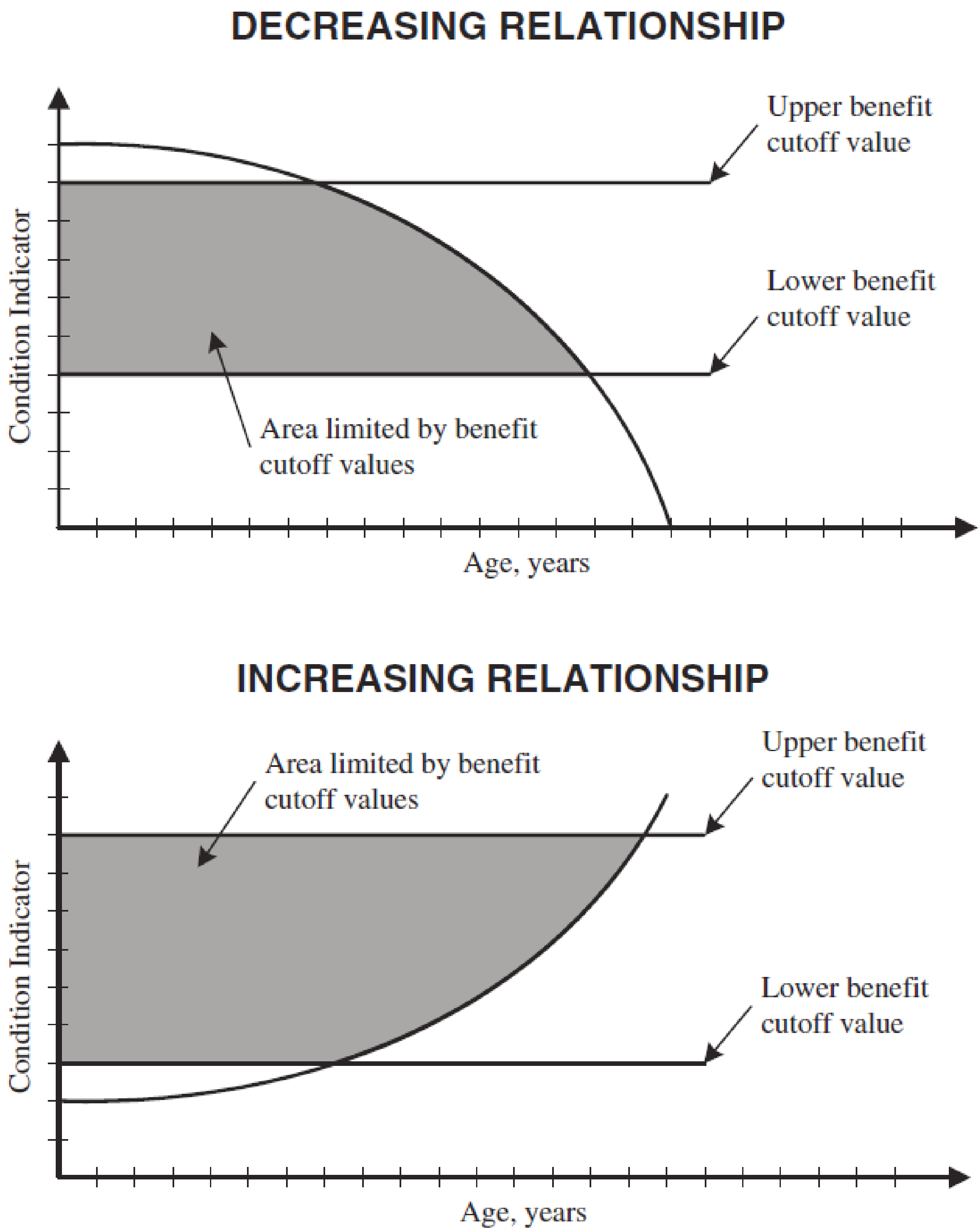


Figure 2.29 Benefit cutoff values (Peshkin et al. 2004)

2. The pavement performance after treatment must be estimated or predicted, from the time when the preventive maintenance is applied to the time when the pavement reaches the lower cutoff value as shown in Figure 2.30. The benefit area (the area between the two performance curves and the lower cutoff value shown in the figure) is then calculated.

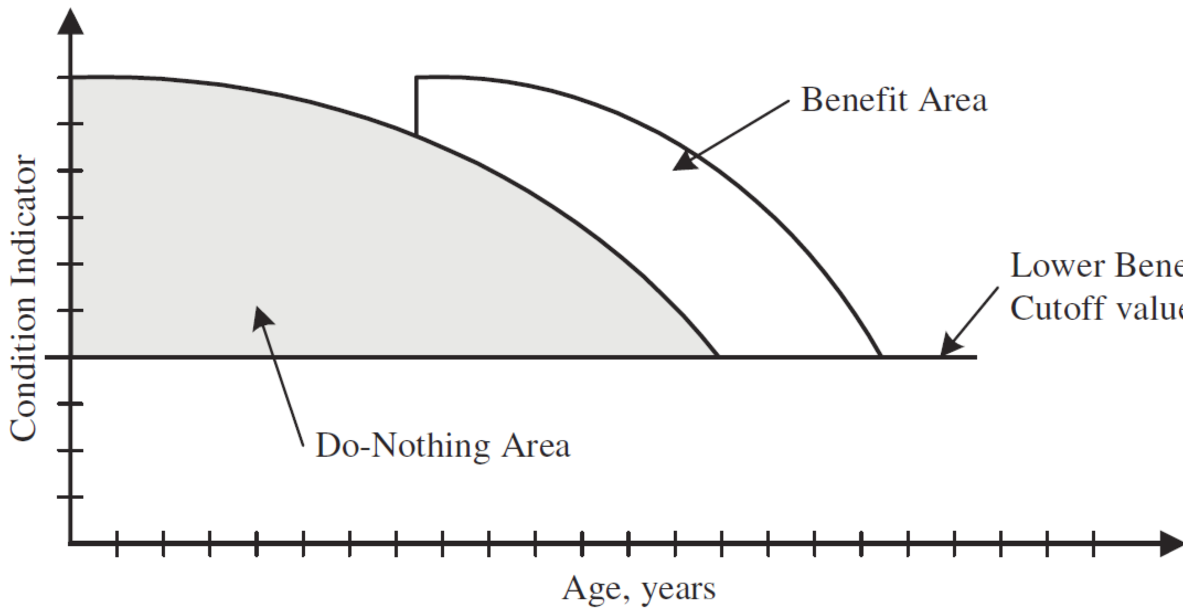


Figure 2.30 Preventive maintenance benefit area (Peshkin et al. 2004)

3. The total benefit of the treatment is then calculated as the ratio of the benefit area to the do-nothing area.

The costs associated with a given preventive maintenance activity are also included in the methodology. The treatment costs, future rehabilitation and routine maintenance costs, and the user costs are combined to calculate the total present worth and Equivalent Uniform Annual Cost (EUAC), for each year selected in the analysis. The user of the Excel sheet has the option of including any or all of the sources of expenditure as well as selecting the discount rate.

The selection of optimum timing for a given preventive maintenance activity is based on maximizing the benefit to cost ratio. Once again, the total benefit is defined as the benefit area divided by the do-nothing area, as shown in Equation 2.5 and Figure 2.31 (Peshkin et al. 2004). The benefit to cost ratio is simply the total benefit divided by the EUAC for a given year of analysis, as stated in Equation 2.6. The year of analysis which produces the highest benefit to cost ratio is considered the optimum timing for the preventive maintenance activity. The ratios are normalized and expressed as the treatment effectiveness index (EI), which is defined by the ratio of a given benefit to cost ratio divided by the maximum benefit to cost ratio as stated in Equation 2.7.

$$TOTALBENEFIT = \frac{AREA(Benefit)}{AREA(Do-Nothing)} \qquad \text{Equation 2.5}$$

$$\left(\frac{Benefit}{Cost}\right)_i = \frac{TOTALBENEFIT_i}{EUAC_i} \qquad \text{Equation 2.6}$$

$$EI_i = \left[\frac{\left(\frac{Benefit}{Cost}\right)_i}{\left(\frac{Benefit}{Cost}\right)_{max}}\right] \times 100 \qquad \text{Equation 2.7}$$

Where, i is the i^{th} treatment timing scenario;

EUAC is the equivalent uniform annual cost;

EI is the effectiveness index;

max is the maximum outcome among the “i" treatment timing scenarios

The OPTime tool is a Microsoft Excel based program. The tool allows the user flexibility in the type and amount of data used in analysis. The user is able to define the following data:

- Pavement type (HMA or PCC)
- Condition indicator(s) (IRI, cracks, etc.)
    - Units (number, length, area, etc.)

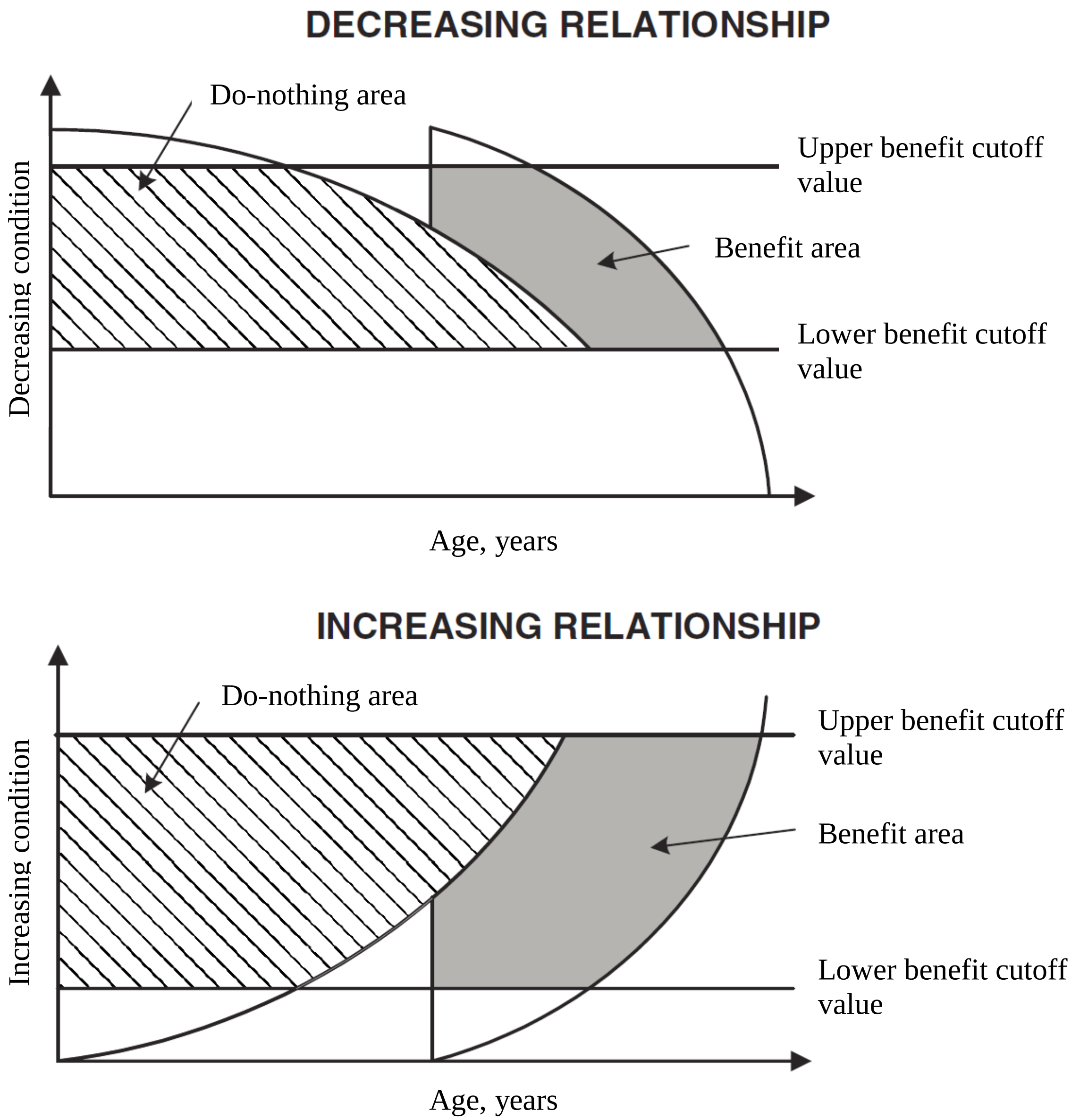


Figure 2.31 Total benefit areas (Peshkin et al. 2004)

- Trend (increasing or decreasing)
- Weight each condition and distress carries (if more than one condition or distress is considered in analysis)
- Lower and upper benefit cutoffs

- Treatment type (crack seal, diamond grinding, etc.)

- Pavement age(s) to apply treatment
- Routine/reactive maintenance schedules

- Performance data or performance equation forms (linear, power, exponential, logarithmic, or 2nd order polynomial) for both pre and post-preventive maintenance activity
- Cost including the costs of the treatment, future maintenance and rehabilitation, and users.

Case Studies

Four case studies were presented to evaluate the methodology. The case studies utilized data from four states (Arizona, Kansas, Michigan, and North Carolina) (Peshkin et al. 2004). Analyses of each of the case studies follow.

Arizona

The case study from Arizona dealt with the effectiveness of seal coat on roughness, friction, and cracking. The three conditions were weighted to 15, 60, and 25% respectively. The analysis was conducted using linear regression equations, listed in Table 2.24. Only treatment cost was analyzed, and the results are listed in Table 2.25 (Peshkin et al. 2004).

Table 2.24 Arizona performance equations (Peshkin et al. 2004)

Do-nothing	
Pavement condition indicator	Regression equation
IRI	0.0207 * AGE + 0.89
Friction	-0.22 * AGE + 60.76
Cracking	0.33 * AGE + 0.6
Post-treatment	
Pavement condition indicator	Regression equation
IRI	$0.0273 * T_{AGE} + 1.52$
Friction	$-0.54 * T_{AGE} + 69.2$
Cracking	$0.7 * T_{AGE} + 3.2$

Table 2.25 Peshkin et al. 2004 Arizona result summary

Application age (year)	Effectiveness index	Total benefit	EUAC ($)	Expected life (year)	Expected extension of life (year)
1	2.38	0.04	2,847	10.7	-15.5
4	17.32	0.23	2,088	13.7	-12.5
7	38.66	0.39	1,606	16.7	-9.5
10	66.30	0.53	1,275	19.7	-6.5
13	100.00	0.65	1,035	22.7	-3.5

The methodology suggests 13 years as optimum because it produced the largest EI. This example is useful in understanding the methodology and the software. However, because of the regression equations used in the analysis, the selected treatment resulted in shorter expected life than the do-nothing case. This magnifies the importance of accurate performance data and accurate modeling algorithm.

Kansas

The case study from Kansas deals with rout and seal treatment of transverse cracks. Kansas uses a special methodology to predict transverse cracking based on the pavement structure and historical data. The data is used to create do-nothing (see Figure 2.32) and post-treatment curves. Only treatment cost was analyzed, and the results are listed in Table 2.26 (Peshkin et al. 2004).

The methodology suggests treatment application at the age of 11 years, it produced the largest EI. Although the maximum total benefit of 1.22 could be achieved if the treatment is applied at the age of 7 years, and the extension in life of 13.0 years could be achieved if the treatment is applied at the age of 13 years; neither time was selected. This implies that the methodology may not lead to the optimum timing. Further, the example suggests that the older is the pavement, the higher is the life extension. This contradicts common engineering judgment; treated older pavements (worse condition) should be expected to have a shorter life extension.

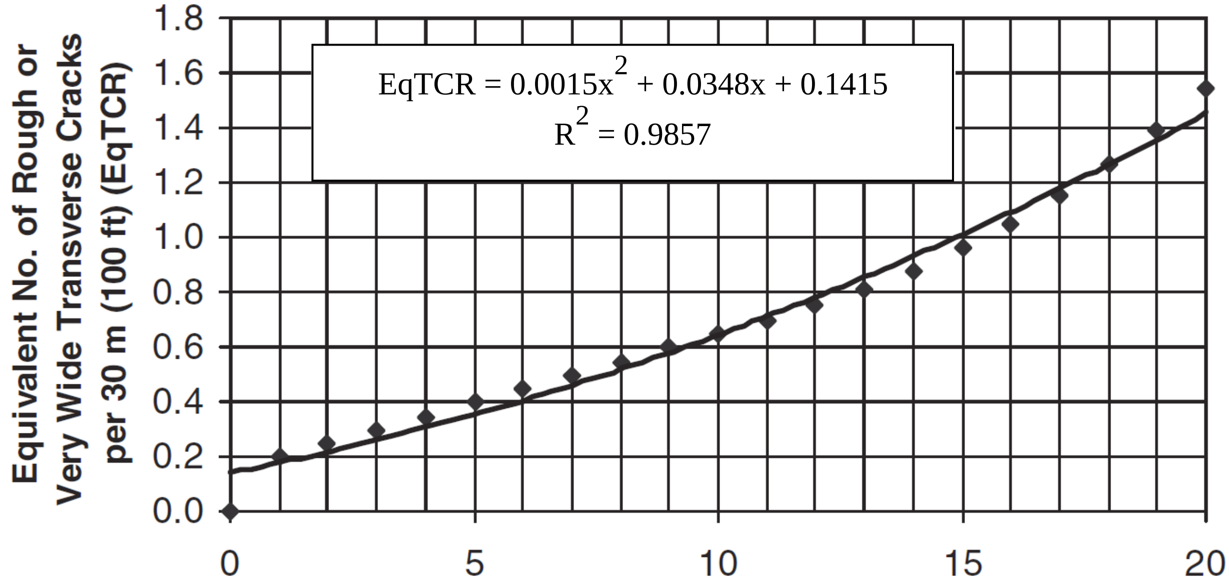


Figure 2.32 The Kansas do-nothing curve (Peshkin et al. 2004)

Table 2.26 Kansas result summary (Peshkin et al. 2004)

Application age (year)	Effectiveness index	Total benefit	EUAC ($)	Expected life (year)	Expected extension of life (year)
1	41.38	0.81	243	14.0	4.3
3	58.89	1.01	214	15.5	5.7
5	75.45	1.15	191	16.9	7.2
7	89.09	1.22	171	18.4	8.7
9	97.77	1.21	154	19.9	10.2
11	100.00	1.12	140	21.3	11.6
13	99.24	1.02	128	22.7	13.0

This anomaly may be due to the data analyzed or the model used, not the methodology.

Michigan

The case study from Michigan deals separately with chip seals and crack sealing of bituminous pavement. Michigan provided actual data for use in predicting the post-treatment distress index (DI) curves. However, the curves model the data linearly using “engineering judgment”. The linear do-nothing curve was created by assuming a terminal age of 13 years.

Only treatment costs were analyzed, and the results are listed in Tables 2.27 and 2.28 (Peshkin et al. 2004).

Table 2.27 Michigan result summary (chip seal) (Peshkin et al. 2004)

Application age (year)	Effectiveness index	Total benefit	EUAC ($)	Expected life (year)	Expected extension of life (year)
10	56.99	0.33	744.62	15.0	2.0
11	100.00	0.49	634.99	17.7	4.7
12	98.16	0.46	602.80	18.0	5.0

Table 2.28 Michigan result summary (crack seal) (Peshkin et al. 2004)

Application age (yrs.)	Effectiveness index	Total benefit	EUAC ($)	Expected life (yrs.)	Expected extension of life (yrs.)
3	17.38	0.21	601.51	13.4	0.4
4	74.01	0.66	452.51	18.8	5.8
5	100.00	0.81	411.46	20.4	7.4
7	38.44	0.37	483.19	14.5	1.5
9	78.55	0.62	399.26	17.9	4.9

The methodology suggested the application of chip seals at the age of 11 years and crack seal at the age of 5 years; the largest EIs. The chip seal example was similar to the case for Kansas, where the largest EI did not correspond to the longest life extension or the lowest EUAC. However, the total benefit was the largest. In the case of the crack seal example, the suggested application age (5 years) did correspond with the longest life extension and highest total benefit, but not the lowest EUAC.

The Michigan case study may be misleading due to the selection of linear pavement performance trends. The performance trends were based on real data, however, it is well known that pavements do not deteriorate in a linear manner. Perhaps, the examples would be more useful if more accurate performance trends had been used in the analysis.

North Carolina

The case study from North Carolina deals with HMA seal coat. North Carolina provided actual pavement condition rating (PCR) data to model PCR as a function of time (see Figure 2.33). In the analysis, a linear do-nothing curve was used from a set of pavements that had received a seal coat treatment. Only treatment cost was analyzed. The results are listed in Table 2.29 (Peshkin et al. 2004).

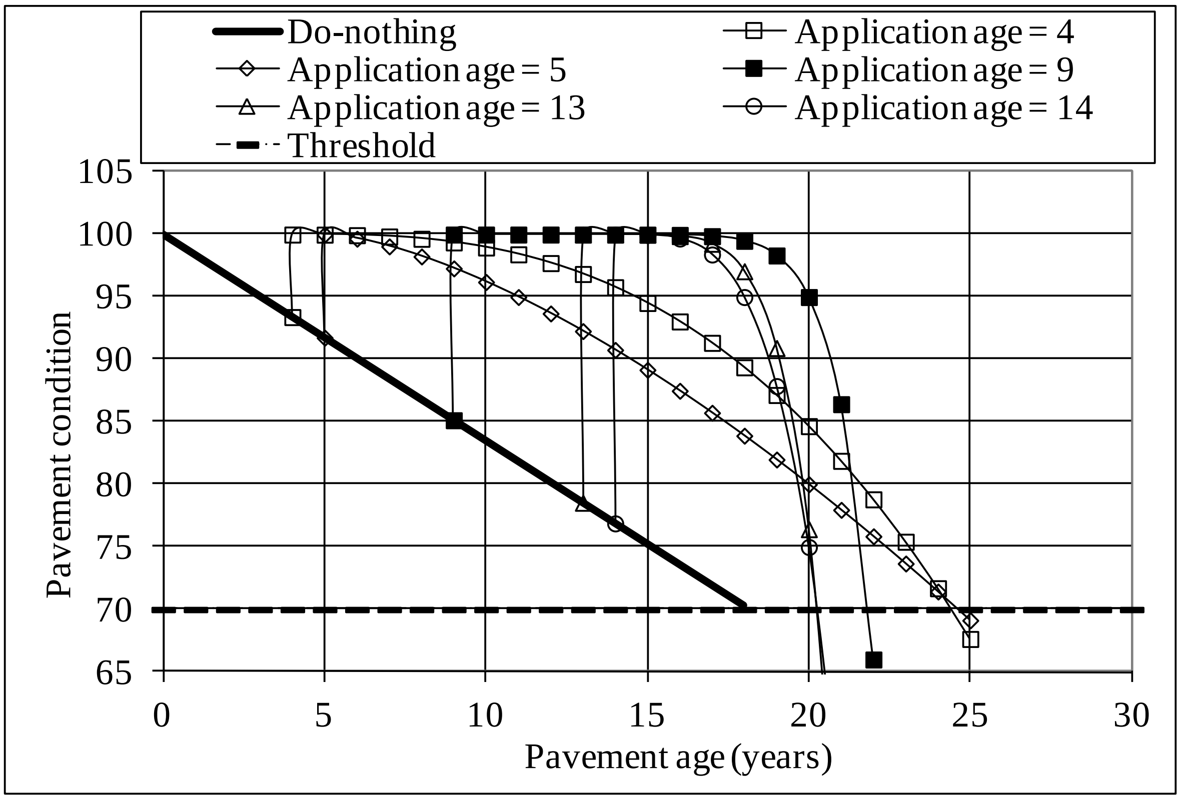


Figure 2.33 North Carolina post-treatment performance curves (Peshkin et al. 2004)

Table 2.29 North Carolina result summary (Peshkin et al. 2004)

Application age (year)	Effectiveness index	Total benefit	EUAC ($)	Expected life (year)	Expected extension of life (year)
4	87.43	1.04	943.00	24.4	6.3
5	68.04	0.77	902.39	24.6	6.5
9	100.00	1.05	829.97	21.9	3.7
13	64.84	0.61	744.37	20.3	2.1
14	55.32	0.50	715.75	20.3	2.1

The methodology suggested the treatment application at the pavement age of 9 years because it produced the largest EI. However, the application age which provided the longest life extension (6.5 years) was not selected. This implies that the methodology may not lead to the optimum timing. For example, the life extension to treatment cost (as the present worth) ratio if the treatment is applied at the age of 4 years would be 4.34×10^{-4} year per dollar compared to only 3.10×10^{-4} year per dollar for the selected application age. The explanation for this anomaly is related to the shapes of the performance curves (see Figure 2.33). Consider the curves from the 5 and 9 year application ages. Due to the vastly different shapes, the 9 year application curve produced a larger area but a significantly smaller life extension than the 5 year curve. The results simply question the validity of the benefit area concept.

Summary and Discussion

The presented methodology was based on the assumptions made. However, some of the assumptions can be questioned. The following section discusses some of those assumptions.

Performance curves

In order to use the methodology, accurate data must be available for modeling both the do-nothing performance curve and the post-treatment performance curves. The methodology is based on calculating the area created by these curves, and therefore the accuracy of this data is reflected in the entire methodology. Consider the case study from Arizona; the performance curves were assumed to be linear, which resulted in a negative extension of life. This is counter-intuitive, and reflects the sensitivity of the methodology to the data. The confidence of the methodology is directly correlated to the confidence in the performance data.

Additionally, the methodology is intended to provide means of determining the optimum timing for conducting a preventive maintenance treatment. However, the data required to do this, following the methodology, cannot be obtained until after the pavement has reached the end of its serviceable life. All of the case studies either used assumed performance curves or data made available only at the end of the pavement life. After-treatment performance is a function of timing, type, traffic, and existing conditions. Therefore, the only way to follow the methodology is to use network-level data to create the post-treatment performance curve and to complete the do-nothing performance curve. The network-level model(s) should be detailed enough to adjust and take into account the various project-level inputs.

However, it may be possible to create the future performance curves by use of the Mechanistic-Empirical Pavement Design Guide (M-E PDG). It is not known if the performance curves from the M-E PDG will be compatible with the known data. Also, very few SHAs are trained in the use of the M-E PDG.

In addition, the M-E PDG is only designed for rehabilitation activities, and the methodology is designed for preventive maintenance activities. However, there appears to be no reason why the methodology can't be extended to rehabilitation activities. The upper and lower bounds would simply enumerate the condition cutoff values for the zone where rehabilitation activities are beneficial rather than preventive maintenance activities.

Benefit Area

The methodology is based on calculating the area created by the performance curves, assuming the curves can be created. However this method may not address the objectives of the agencies that may use this methodology. Most SHAs are likely interested in the service life extension of the pavement rather than the "benefit area". Consider the post-treatment performance curves in Figure 2.33. The total benefit associated with an application age of 9 years is negligibly greater than that of the 4 year application age (1.05 to 1.04). However, the life extension of the 9 year application age is much less than that of the 4 year (3.7 to 6.3 years). The service life extension is likely to be much more accepted by SHAs than the total benefit because it maximizes cost-effectiveness.

Likewise, the optimum timing is selected based on the application age which produces the highest benefit to cost ratio. However, the highest service life extension to cost ratio is likely more applicable to most SHAs. The methodology may be more meaningful if it suggested optimum timing based on an alternate calculation of EI, such as that shown in Equation 2.8.

$$\mathrm{EI_i} = \left[\frac{\left(\frac{\text{Life extension}}{\text{Cost}}\right)_i}{\left(\frac{\text{Life extension}}{\text{Cost}}\right)_{max}} \right] \times 100 \qquad \text{Equation 2.8}$$

Where, EI is the effectiveness index;

i is the i^{th} treatment timing scenario;

max is the maximum outcome among the “i" treatment timing scenarios

2.5.6.3 Summary of Treatment Benefits

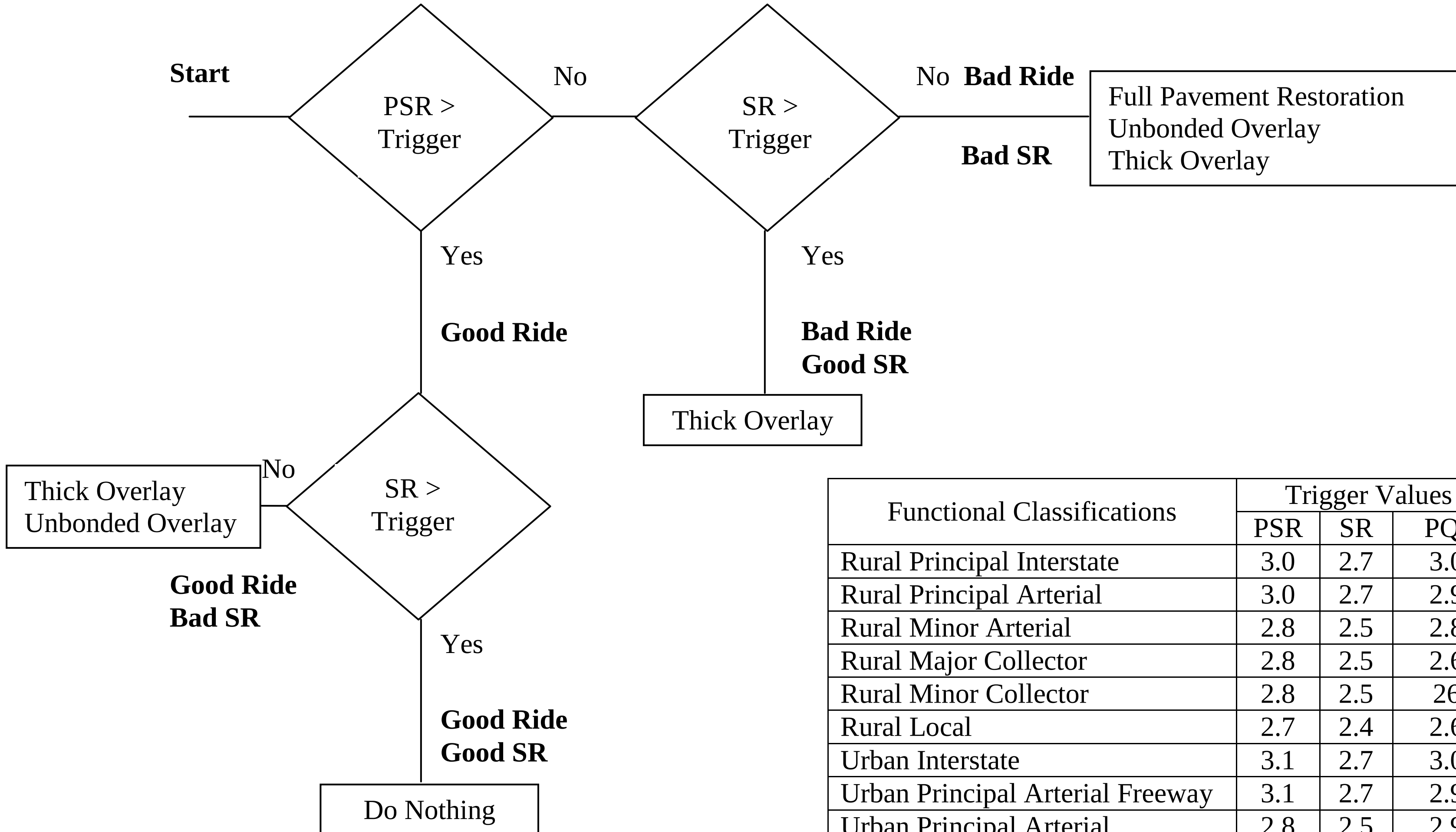
The most common procedure in the literature to optimize treatment type and timing is by benefit-cost analysis. The costs are determined as discussed in section 2.7. The benefits are determined by one of the four methods listed below (Hall et al. 2002, Chen et al. 2003, Peshkin et al. 2004, Smith et al. 2005, Shuler & Schmidt 2009, Jahh et al. 2010, Li et al. 2010, Zhang et al. 2010).

1. The area under or above the performance curve, and bound by a single or a set of threshold values. This method measures benefit as the pavement condition (distress or distress index) over a period of time.

2. The extension in time for the pavement to reach a given condition or distress value (distress or distress index). This method measures benefit as the extension in time before the next treatment is required.
3. The actual time between the applications of pavement treatments based on the state-of-the-practice. This method measures benefit by the real elapsed time between two consecutive pavement treatments regardless of the physical conditions, distress levels, or the values of the distress index.
4. The immediate improvement in pavement condition due to treatment. The method measures benefits as the decrease in pavement distress following the treatment without regard to future conditions. The method does not account for the pavement rate of deterioration. To illustrate, consider a pavement section with an "X" level of distress. The pavement was subjected to 1-inch overlay and the distress after the overlay is zero. The treatment benefit is the "X" level of distress. If the same pavement section is subjected to 2.5-inch overlay, the benefits will be exactly the same. The pavement rates of deterioration of both treatments are drastically different. The 1-inch overlay may be expected to last 3 years whereas the 2.5-inch is expected to last about 8 years.

2.6 Pavement Treatment Type Selection

Many SHAs have developed plans and methodologies for selecting pavement treatments. The most common are decision trees and matrices. These are often developed from past experience and tend to focus on one or two options. The trees/matrices are rarely updated and often neglect new technology. Example of a decision tree and a decision matrix are shown in Figures 2.34 and 2.35. None-the-less, they are typically based on the following (Hicks et al. 2000):



Functional Classifications	Trigger Values		
	PSR	SR	PQI
Rural Principal Interstate	3.0	2.7	3.0
Rural Principal Arterial	3.0	2.7	2.9
Rural Minor Arterial	2.8	2.5	2.8
Rural Major Collector	2.8	2.5	2.6
Rural Minor Collector	2.8	2.5	26
Rural Local	2.7	2.4	2.6
Urban Interstate	3.1	2.7	3.0
Urban Principal Arterial Freeway	3.1	2.7	2.9
Urban Principal Arterial	2.8	2.5	2.9
Urban Minor Arterial	2.7	2.4	2.8
Urban Collector	2.6	2.4	2.6
Urban Local	2.5	2.4	2.6

Figure 2.34 Example of decision tree for continuously reinforced concrete pavement (CRCP) (Hicks et al. 2000)

Problem	Possible Cause				Maintenance				Rehabilitation					
	Structural Failure	Mix Composition	Temperature or Moisture Changes	Construction	Patching & Routine Maintenance	Fog Seal	Surface Treatment	Slurry Seal	Surface Recycling	Thin Overlay	Open-Graded Surface	Structural Overlay	Structural Recycling	Reconstruction
Alligator Cracking	X				X		X	X				X	X	X
Edge Joint Cracks	X		X	X	X									
Reflection Cracks					X		X	X			X	X	X	
Shrinkage Cracking		X	X				X	X	X		X	X	X	
Slippage Cracks				X	X									
Rutting	X	X		X					X	X		X	X	X
Corrugation	X	X		X					X	X		X	X	X
Depressions	X			X	X								X	X
Upheaval			X		X								X	X
Potholes	X		X	X	X							X		
Raveling		X		X		X	X	X	X	X				
Flushing Asphalt		X		X			X		X		X			
Polished Aggregate		X	X				X		X	X	X			
Loss of Cover Aggregate		X		X			X							

Figure 2.35 Example of decision matrix (Asphalt Institute 1983)

- Pavement surface type and/or construction history.
- An indication of the functional classification and/or traffic level.
- At least one type of condition index, including distress and/or roughness.
- More specific information about the type of deterioration present, either in terms of an amount of load-related deterioration or the presence of a particular condition or distress type.
- Geometric data indicating whether or not pavement widening or shoulder repair are required.
- Environmental conditions in which the treatment is to be used.

The pavement treatment type should be selected based on the pavement conditions or distresses present and their cause(s). The selected pavement treatment type should address (cover, retard progression, and/or eliminate) the pavement conditions or distresses (Baladi et al. 1999). Note that a pavement treatment that addresses the conditions but not the cause of the conditions is an option and may be "optimum" at the network level, considering certain budgetary constraints. The treatment which addresses the most pavement conditions or distresses and their causes will be the preferred choice with respect to cost. The selection of pavement treatment type based on the pavement conditions or distresses and their causes are discussed in the next few subsections.

2.6.1 Pavement Treatment Selection Based on the Pavement Condition

One objective of performing a treatment on a pavement section is to improve the conditions of the pavement. When the appropriate treatment is selected, it should improve the pavement surface distress and ride quality and decrease the rate of deterioration. Pavement treatments that do not address the pavement conditions and ride quality should not be considered. Each SHA has its own process for treatment type selection; most of which are based on some combination of the pavement conditions and distresses, the severity of the distresses, and the life expectancy of the treatment and its cost (Hicks et al. 2000).

2.6.2 Pavement Treatment Selection Based on the Causes of Pavement Condition

One objective of applying pavement treatments is to retard the rate of pavement deterioration. This could be achieved if the selected treatment reduces or eliminates the causes of the conditions or distresses. Pavement treatments that do not address the causes of conditions or distresses will not prevent the previous conditions from returning (Baladi et al. 1999).

2.7 Pavement Treatment Costs

The costs of any pavement treatment can be divided into two categories; agency costs and user costs. The agency cost is the physical cost of the pavement project; including design and construction less the residual value of the pavement section at the end of its life. This is often referred to as direct costs (Morgado & Neves 2008). User costs are much more difficult to estimate than agency costs as they are not based on specific monetary value, but on vehicle operating costs (VOC), delay costs, and accident costs. The three types of user costs and how they relate to normal and work zone conditions are listed in Table 2.30 and discussed in the next few subsections (Morgado & Neves 2008).

Table 2.30 Review of user cost components (Reigle & Zaniewski, 2002)

Component	Normal operation	Work zone conditions
VOC	Based on total delay hours caused by accidents	Based on total delay hours caused by work zone and accidents in the work zone
Delay	Total delay hours (due to accidents)	Total delay hours (due to work zone and accidents in work zone)
Accidents	Number and severity of accidents	Number and severity of work zone accidents

One problem that arises when estimating user costs is the transformation of delay, accidents, etc. to a monetary value (Khurshid et al. 2009). Some believe that user costs should be defined as “user benefit” and expressed qualitatively as improvements in performance or safety (Mouaket & Sinha 1991, Lamptey et al. 2004). The user benefit of one treatment relative to another or to the do-nothing alternative could be used to select between treatments with similar agency costs. This would greatly simplify the process which is often considered to be complicated and deficient, especially when applicable data are not available for the various detailed user cost models (Fazil & Paredes 2001). These costs and life cycle cost analyses are discussed in the next few subsections.

2.7.1 Net Present Worth and Equivalent Uniform Annual Cost

The net present worth (NPW) or net present value is a common economic indicator. NPW is the monetary value of an action accounting for the transformation of the value of money over time using the discount rate (see Equation 2.9). The Equivalent Uniform Annual Cost (EUAC) is also commonly used and, is often derived from NPW (see Equation 2.10). The use of either value allows for fair comparison of actions taken at different times by converting to a common unit of measure (Walls & Smith 1998).

$$\text{NPW} = \text{Initial Cost} + \sum_{k=1}^{N} \text{Preservation Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right] \qquad \text{Equation 2.9}$$

$$\text{EUAC} = \text{NPW} \left[\frac{(1+i)^n}{(1+i)^n - 1} \right] \qquad \text{Equation 2.10}$$

Where, NPW is the net present worth;

N is the total number of preservation treatments;

i is the discount rate;

n is the number of years into the future;

k is the k^{th} action;

EUAC is the equivalent uniform annual cost

The discount rate reflects the rate of inflation adjusted to the opportunity cost to the public. The opportunity cost is often indicated by the conservative US Treasury Bill. Figure 2.36 shows the historical trend in the first half of the 1990s, where the adjusted discount rate was between 3 and 5% with an average of about 4%. Table 2.31 lists common discount rates used by SHAs in the 1990s. The discount rate should reflect historical trends in the nation or region where the analysis is conducted (Walls and Smith 1998). Alternatively, the discount rate could be determined from the consumer price index (CPI). The average CPI discount rate from 2001 to 2010 was about 2.54% (Baladi et al. 2011, BLS 2011).

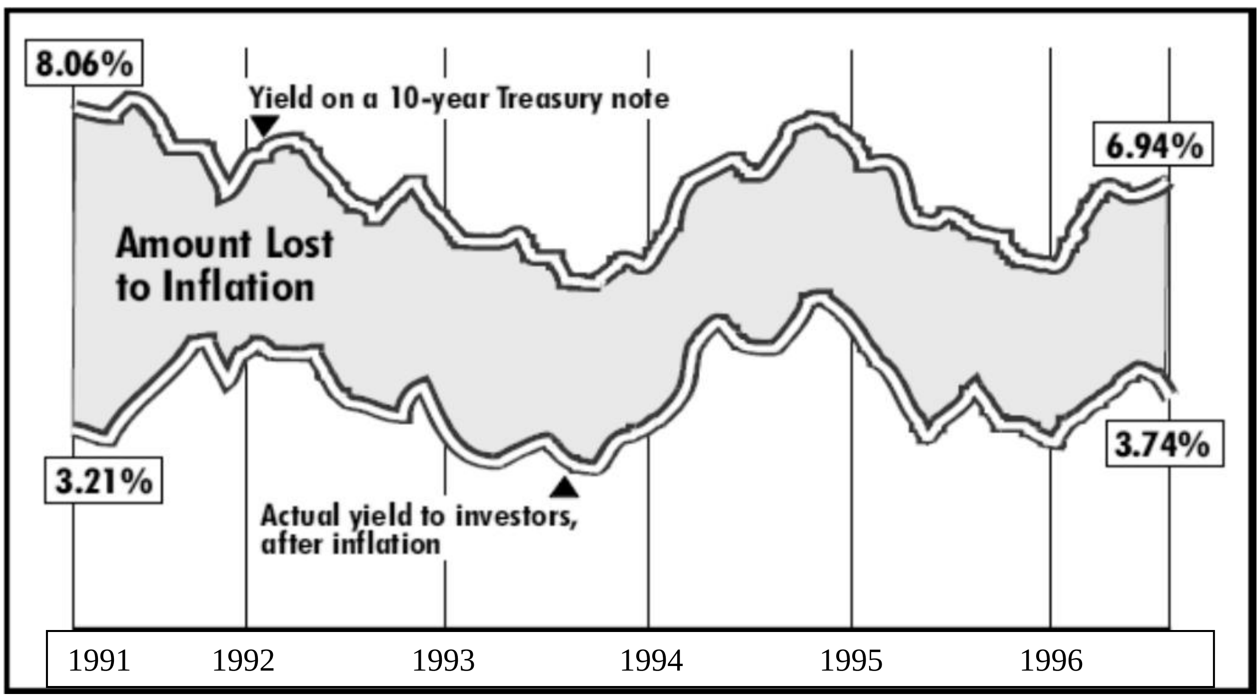


Figure 2.36 Historical trends of US Treasury note

2.7.2 Life Cycle Cost Analysis (LCCA)

The information presented in this subsection are mainly based on those reported by Baladi et al. 2011. Today SHAs are faced with many constraints such as public demand for quality

Table 2.31 Historical discount rates

Year	Analysis period (year)				
	3	5	7	10	30
1992	2.7	3.1	3.3	3.6	3.8
1993	3.1	3.6	4.0	4.3	4.5
1994	2.1	2.3	2.5	2.7	2.8
1995	4.2	4.5	4.6	4.8	4.9
1996	2.7	2.7	2.8	2.8	3.0
1997	3.2	3.3	3.4	3.5	3.6
1998	3.4	3.5	3.5	3.6	3.8
Average	3.1	3.3	3.4	3.6	3.8

pavement and budget short fall, the questions for a SHA become:

- What pavement preservation alternatives should be used and hence, how often should a given pavement section be preserved?
- How many miles of each pavement class should be preserved?
- What is the optimum time or the optimum pavement conditions and distresses at which pavement preservation actions should be taken?
- What are the associated agency and user costs and benefits of each pavement treatment?
- What are the optimum and most cost-effective short and long term pavement preservation strategies that can be applied to keep the pavement network healthy in a cost-effective manner?

These questions cannot be properly and accurately answered unless LCCA is conducted. Such analysis should address the agency and the user costs and must be based on accurate and up to date data so that the costs and benefits of various pavement preservation alternatives can be compared.

2.7.2.1 The Need for LCCA

In general, highway pavements are designed and constructed to provide services for a limited time called the service life. Over time, the combined effects of traffic loads and environmental factors accelerate the pavement deterioration and reduce its level of serviceability.

Maintenance, preservation, and rehabilitation treatments are designed and applied to pavement sections to slow their rates of deterioration and to extend their service lives. The application of any pavement treatment requires traffic control (lane closures and/or detours), which significantly impacts traffic flow, increases travel time, and increases VOCs. The costs and benefits of pavement treatments are comprised of many elements including:

1. Agency costs of the pavement treatment, which consist of many attributes including:
    - Material and contractual costs.
    - The cost of traffic control in the work zone, which is defined as an area along the highway system where maintenance and construction operations adversely affect the number of lanes opened to traffic or affect the operational characteristics of traffic flow through the work zone (Chien et al. 2002).
    - Quality assurance and quality control (QA/QC) costs.
    - The costs of future treatments.
2. Agency benefits, which could be measured by the life of the treatment or the SLE of the treated pavement sections.
3. User costs, which are also comprised of many attributes including (Lewis 1999, Daniels et al. 2000):
    - Time delay user costs or work zone user costs, which are defined as the associated costs of time delays due to lane closures because of roadway construction, rehabilitation, and maintenance activities (Berthelot 1996).
    - Costs incurred by those highway users who cannot use the facility because of either agency or self imposed detour requirements (Walls & Smith 1998).
    - VOCs in terms of fuel, wear and tear, and depreciation over the delay periods.

- Accident costs.
- Environmental costs due to air pollution caused by excessive uses of gasoline or diesel fuel due to lower speed and time delay, including noise pollution.

4. User benefits which are comprised of improved serviceability and ride quality that would lower the VOCs and improve traffic flow.

User costs due to planned changes in highway capacity or improvement in pavement condition by means of road maintenance, rehabilitation, and/or reconstruction could also be divided into two categories; work zone costs and non-work zone costs (Lewis 1999, Daniels et al. 2000). These two general categories and their various components are shown in Figure 2.37 (Martenelli & Xu 1996, Chien & Schonfeld 2001, Chien et al. 2002,).

Estimation of the work zone user costs requires significant and detailed information on the work zone length, speed restrictions, lane closures, VOCs, and traffic capacity in the work zone (Hall et al. 2003). For instance, work zone length has a significant impact on the user costs when the demand is greater than the capacity allowed by the traffic control practices (Dudek et al. 1986). Hence, it is important to estimate an optimum work-zone length which minimizes both user and construction costs. Various models have been developed to estimate optimum work zone lengths for both two and four lane freeways by minimizing both construction and additional user costs.

Traffic control procedures also have a significant impact on the agency and user costs in work-zones. Some traffic control procedures may accelerate construction, therefore decreasing the time the work-zone is in place. In this way, the agency cost increases while the user cost decreases. On the other hand, other traffic control procedures may cause the duration of the work- zone to increase, decreasing agency cost and increasing the user cost. Stated differently,

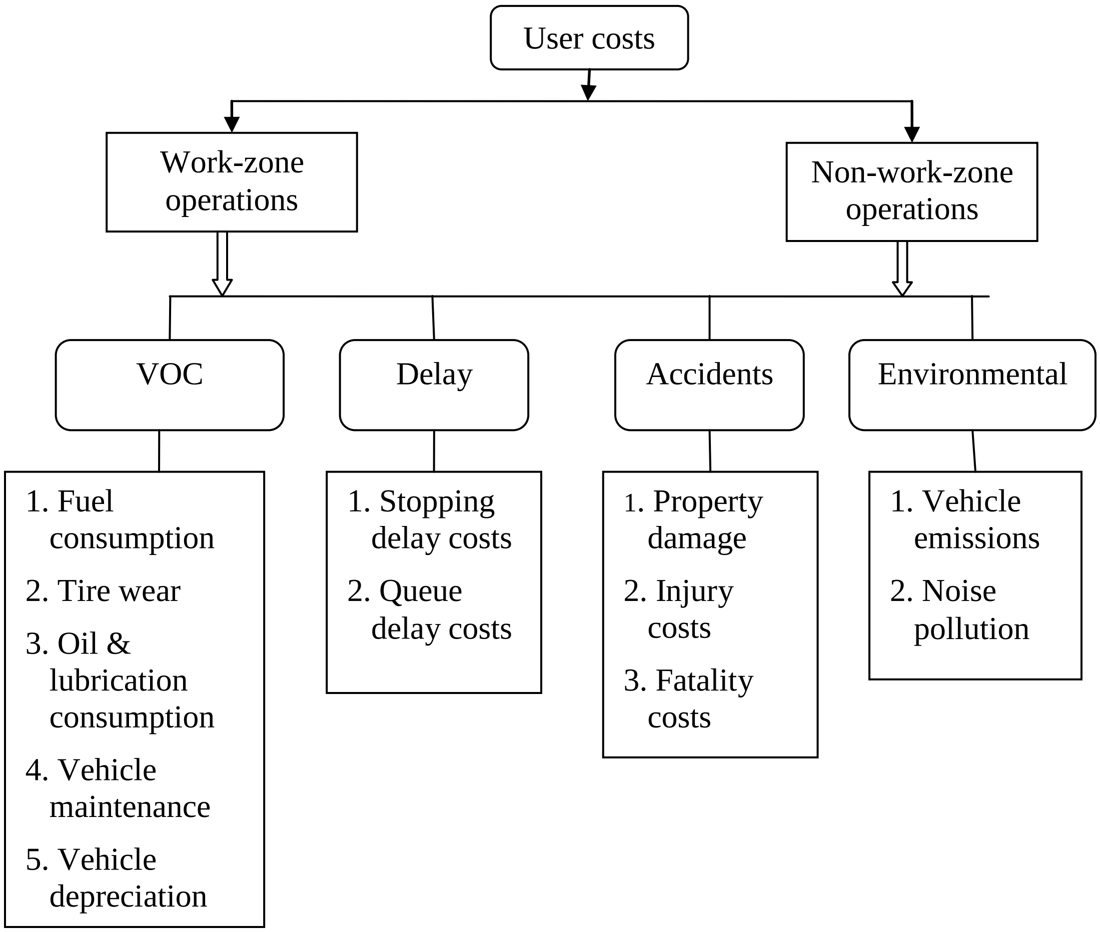


Figure 2.37 Components of user costs (Morgado & Neves 2008)

there is a direct relationship between traffic control procedures, agency cost, and user cost. Various analyses indicate that the cost effectiveness of traffic control procedures depends on several site factors and hence, it should be evaluated on a site-by-site basis (Dudek et al. 1986).

As shown in Figure 2.37, non-work zone user costs are comprised of the costs associated with standard travel such as VOCs, accident, and environmental costs. The delay time is assumed to be zero. The VOCs are also a function of several other variables including vehicle type, vehicle speed, speed changes, gradient, curvature, and pavement surface (Morgado & Neves 2008). Detailed methods to estimate the road user cost (delay cost) in both non-work zones and work zones are presented in the next subsections.

2.7.2.2 Methods to Estimate Road User Cost

One method of estimating road user cost consists of multiplying the delay time by the dollar value of time (VOT) (Daniels et al. 2000). The VOT, adjusted for inflation and the discount rate, for passenger cars and trucks are estimated to be $27.20 and $23.30 per hour, respectively (USDOT 1997).

The road user cost, or delay cost in work-zones is the sum of the costs associated with vehicle delay due to maintenance activities on the road and the value of the time lost for the vehicle users. To estimate the costs due to travel delays as a result of work zone operations, four delay costs are considered. These costs include speed change delay, reduced speed change delay, stopping delay, and queue delay costs (Lamptey et al. 2004, Morgado & Neves 2008). The following models are used to quantify these delay costs:

1. Speed Change Delay Costs - Speed Change Delay Costs are defined as the costs due to additional time required for decelerating from the upstream approach speed to work-zone speed and accelerating back to the initial approach speed after traversing the work zone (Lamptey et al. 2004). It is given by Equation 2.11:

$$\text{Speed Change Delay Cost} = \sum_{i}^{n} \left(Nveh_i * SCTime_i * DCost_i \right) \qquad \text{Equation 2.11}$$

Where, $Nveh_i$ is the number of vehicles delayed by the speed change for vehicle class i in 1000 vehicles;

$SCTime_i$ is the added time for the speed changes for vehicle class i in hours/1000 vehicles;

$DCost_i$ is the delay cost rate for vehicle class i in $/vehicle-hr;

i is the vehicle class;

n is the number of vehicle classes

2. Reduced Speed Delay Costs - Reduced Speed Delay Costs are the costs associated with the additional time required to traverse the work zone at the reduced speed limit (Lamptey et al. 2004). It is given by Equation 2.12:

$$\text{Reduced Speed Delay Cost} = \sum_{i}^{n}\left(\text{Nveh}_i * \text{RSTime}_i * \text{DCost}_i\right) \qquad \text{Equation 2.12}$$

Where, RSTime_i is the added time for the reduced speed for vehicle class i (hours/1000 vehicles)

3. Stopping Delay Costs - Stopping Delay Costs are the costs associated with the additional time required for the vehicle to come to a complete stop from the upstream speed and accelerate back to the work zone speed limit (Lamptey et al. 2004). It is given by Equation 2.13:

$$\text{Stopping Delay Cost} = \sum_{i}^{n}\left(\text{Nveh}_i * \text{STime}_i * \text{DCost}_i\right) \qquad \text{Equation 2.13}$$

Where, STime_i is the added time for stopping for vehicle class i (hours/1000 vehicles)

4. Queue Delay Costs - Queue Delay Costs are the costs associated with the additional time required to go through the queue that is formed as a result of the work zone *(*Lamptey et al. 2004*)*. It is given by Equation 2.14:

$$\text{Queue Delay Cost} = \sum_{i}^{n}\left(\text{Nveh}_i * \text{QTime}_i * \text{DCost}_i\right) \qquad \text{Equation 2.14}$$

Where, QTime_i is the queue delay for vehicle class i (hours/1000 vehicles)

Besides the costs outlined above, average additional user costs are estimated to be $0.11 per vehicle when there is no significant queue and $0.96 to $1.43 per vehicle in the presence of a significant queue (Dudek et al. 1986). It is estimated that, on average, the fuel consumption increases by 13% and the vehicle emissions increase by 25% on a congested road (Kerali et al. 2000). A steep drop in the average moving speed was observed as the Volume – Capacity Ratio (VCR) approaches 1.0 and the Level of Service (LOS) of the freeway reduces from "A" to "F". The percent trucks also has a significant impact on the LOS as they tend to move slowly causing a reduction in the operating speed and thus an increase in the user cost (Martenelli & Xu 1996).

2.7.2.3 Value of Time Costs

The time spent traveling in a vehicle also has a cost associated with it, often referred to as the opportunity cost. It is defined as the value associated with the activities that could be conducted instead of traveling (Lewis 1999). Time related cost savings could further be divided into the savings in working time and the savings in non-working time. Time related cost savings during the working time are related to the average wage rates, fringe benefits, and overhead allowances calculated while the workers are in transit. The following approaches, summarized from a previous literature review, are generally used to measure the value of working time (Lewis 1999):

- Macro Choice Models – These models are based on the reaction of individuals traveling for work purposes, who face a time trade-off. The value of the working time based on this approach is estimated to be close to the average wage rate of the travelers.
- Case Studies of States Costs - Costs of the resources associated with the travel during work time are considered in estimating the working time related savings. The working time value estimated based on this approach was found to be equal to the wage rate of the traveler.

- Survey Techniques - This approach is based on surveys of work time savings. The surveys were conducted among long distance trucking operations and commercial vehicles to evaluate the uses of the saved travel time. About 29 to 35 percent of the responses from the surveys concluded that the travelers use the saved time for some leisure activities. In this scenario, the working time value is estimated to be 40 to 57 percent of the wage rate. If the time saved is used for other productive work, the working time value is estimated to be slightly higher than the wage rate.

The time related costs during the non-working time are estimated by the concept of compensation. These costs are measured in terms of the amount of compensation required by the traveler to forego a reduction in the travel time while maintaining their initial level of utility. The following approaches, summarized from a previous literature review, are used to estimate the time related costs during the non-working hours (Lewis 1999):

- Discrete Choice Models - The model assumes that the traveler estimates the value of the time savings through their selection of various modes of transportation and/or routes. In this approach, random utility theory is used to estimate the probability of an individual choosing a given transportation mode or route.
- Travel Demand Models - The value of the non-working time is estimated using the modal demand analysis with time and out-of-pocket expenses as the variables.
- Stated Willingness-to-Pay Studies - This approach consists of surveying the travelers regarding the amount of money they are willing to pay for reduction in travel time. The results of this survey were found inconsistent due to various biases in the responses.
- Speed Choice Models - The value of the time is estimated by comparing the costs and times of all available alternate transit facilities such as busses, subway, train, etc.

2.7.2.4 Vehicle Operating Costs (VOCs)

VOCs account for a significant portion of the total road user cost. VOCs are incurred to the user for owning, operating, and maintaining a vehicle. The VOCs consist of several independent cost components including the costs of fuel consumption, oil, tire wear, regular maintenance, and vehicle depreciation. Each component is a unique function of vehicle class, vehicle speed, grade level, and the pavement surface condition. VOCs are also a function of the pavement roughness (Berthelot et al. 1996). Table 2.32 presents a list of the vehicle and roadway factors that affect the VOCs components.

Table 2.32 Roadway factors affecting vehicle operating costs (Lewis 1999)

Roadway factors	Vehicle Operating Cost Component				
	Fuel	Oil	Tire wear	Maintenance	Depreciation
Vehicle class	X	X	X	X	X
Vehicle speed	X	X	X	X	X
Road grade	X	X	X	X	
Pavement surface type	X	X	X	X	X
Pavement surface condition	X	X	X	X	X
Road curvature	X		X	X	

In addition, Figure 2.38 presents the average VOCs for commercial trucks estimated for various pavement conditions (Barnes and Langworthy 2004). As can be seen from the figure, for commercial trucks, fuel cost represents the majority of the VOCs. A similar trend has been observed for automobiles and pickup vans. It should be noted that a fuel price of $1.50 per gallon was used in the calculations. The American Automobile Association (AAA) estimates the average fuel cost for all categories of automobile to be 11.36 cents/mile, while the total road user costs are estimated at about 47.6 cents/mile (Haas and Hudson 1982). The insurance company AAA used a fuel price of $2.603 per gallon in its estimation of fuel cost.

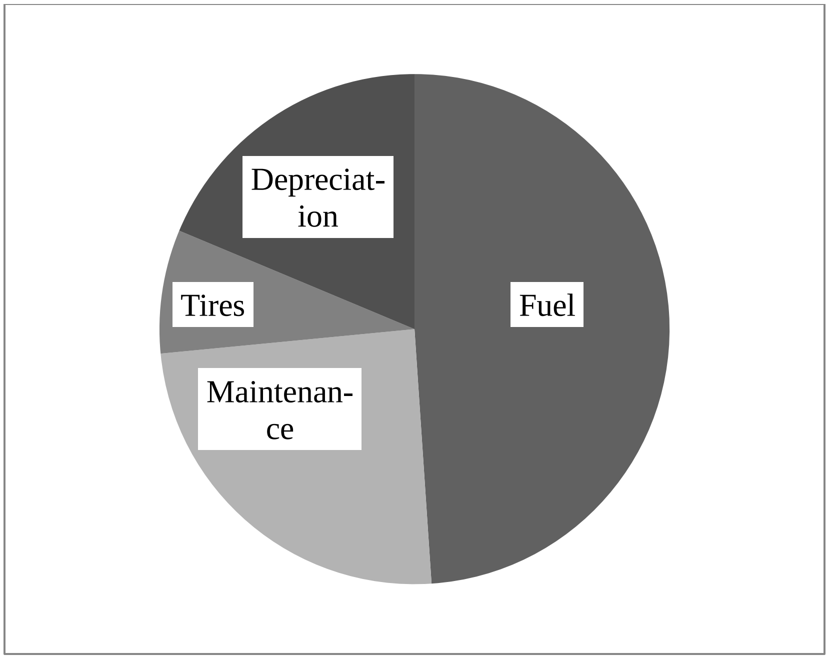


Figure 2.38 Vehicle operating costs for commercial trucks (Barnes & Langworthy 2004)

2.7.2.5 Models to Estimate VOCs

Several models have been developed to estimate VOCs. Table 2.33 provides a list of various models that were developed and used by various researchers to estimate the VOCs and its relationship to various roadway factors. Some of the principal models include:

1. The AASHTO Red Book.
2. The Texas Research and Development Foundation’s (TRDF) VOC relationships.
3. The World Bank’s Highway Design and Maintenance Standards Model (HDM – III or IV).
4. The Australian Road Research Board’s Road Fuel Consumption Model (ARFCOM).
5. The National Association of Australian State Road Authorities Improved Model for Vroject Assessment and Costing (NIMPAC).
6. The Swedish Road and Traffic Research Institute’s Vejstandard och Transportomkostninger (VETO) model.
7. The British Cost Benefit Analysis Program (COBA).

Table 2.33 A summary of VOC models (Zaabar 2010)

Feature	VOC Models				
					TRDF
	HDM-III	VETO	ARFCOM	HDM-IV	MicroBENCOST
Empirical	✓	-	-	-	✓
Mechanistic	✓	✓	✓	✓	-
Level of aggregation					
Simulation	-	✓	✓	✓	-
Project level	✓	✓	✓	✓	✓
Network level	✓	-	✓	✓	-
Vehicle operation					
Uniform speed	✓	✓	✓	✓	✓
Curves	✓	✓	✓	✓	✓
Speed change	-	✓	✓	✓	✓
Idling	-	✓	✓	✓	✓
Typical vehicles					
Default	✓	✓	✓	✓	✓
User specified	✓	✓	✓	✓	✓
Modern truck	✓	✓	✓	✓	✓
Road-related variables					
Gradient	✓	✓	✓	✓	✓
Curvature	✓	✓	✓	✓	-
Super-elevation	✓	✓	✓		-
Roughness	✓	✓	✓		-
Pavement type	✓	✓	✓		-
Texture	-	✓	✓	✓	-
Snow, water	-	✓	✓	✓	✓
Wind, temperature	-	✓	✓	✓	-
Absolute elevation	✓	✓	✓		-
VOC components					
Fuel, oil, tires, repair/maintenance, depreciation	✓	✓	✓	✓	✓
Interest	✓	✓	-	✓	-
Cargo damage	-	✓	-		-
Overhead	✓	✓	-		-
Fleet stock	-	✓	-		-
Exhaust emissions	-	✓	-	✓	-

A summary of each of the aforementioned models is presented hereafter. Detailed literature review of all models listed in Table 2.33, can be found in (Lewis 1999).

1. AASHTO Red Book VOCs Model – This model provides relationships to calculate the VOCs on uniform sections of highway, in transition between sections with different characteristics, and at intersections. Estimation of VOCs using AASHTO Red Book's VOC model is based on empirical equations and is time consuming (the calculations are done manually). Hence the model has limited applications.
2. TRDF VOCs Model –This model is based on the survey done by the World Bank in Brazil, the survey data used in developing the VOCs model in AASHTO's Red Book, and fuel consumption data in the United States for all vehicle classes. The TRDF VOCs relationships estimate the tire wear and vehicle depreciation along with fuel costs. While the TRDF relationships are partly based on the operating characteristics of newer vehicles, they do not account for changing vehicle technologies. The computer program MicroBENCOST utilizes this model.
3. Highway Design and Maintenance Standards Model - The HDM–III was developed by the World Bank from data collected during surveys of road users in Brazil, India, Kenya and the Caribbean. The basic task of HDM-III model is to estimate the total life cycle cost, which is considered to be the sum of construction, maintenance, and road user costs, as a function of road design, maintenance standards, and policy options (Jiang & Adeli 1982). The road user costs are estimated as a function of geometric design, road surface condition, vehicle speed, vehicle type, and unit costs. In HDM-III, only VOCs are estimated as user costs, while the delay costs and accident costs are ignored. By the year 1995, the vehicle technology improved significantly and the relationships developed for estimating the VOCs required

update. HDM-IV was created to include these updates, as well as additional capabilities such as the model for estimating the effects of traffic congestion, pavement types and structures, and environment (Watanatada et al. 1987).

4. ARFCOM VOCs Model – The ARFCOM model is capable of estimating the fuel consumption due to speed changes that are induced by change in grade and curvature or due to traffic control. ARFCOM calculates fuel consumption by estimating the power that must be produced by the engine. The users have the liberty to select from three different models, which differ with regards to the usage of vehicle speed data and the level of aggregation. This flexibility makes the VOC model suitable for both rural and urban traffic and transport management applications.
5. NIMPAC – This is a computer program used by Australian Road Research Board to estimate the VOCs for both rural and urban roads. The NIMPAC VOC model estimates fuel, oil, tire wear, repairs, maintenance, and depreciation costs. A study conducted by the National Association of Australian State Road Authorities (NAASRA) found that NIMPAC overestimates fuel costs of trucks because it does not account for the improved performance over the decades.
6. VETO – This model, developed by Swedish Road and Traffic Research Institute (VTI), is purely based on mechanistic analysis. VETO estimates the costs of fuel consumption, tire wear, repair cost, distance and time related depreciation and interest charges for vehicle and cargo.
7. COBA - This mainframe computer program is used to analyze the complicated roadway networks consisting of numerous links and nodes. COBA assumes the traffic to be the same before and after improvement and the user cost savings are calculated directly as the savings

per trip for alternative routes consisting of travel through different nodes and routes. The COBA model does not take into account the impact of pavement condition on the vehicle speed and VOCs.

Some of the issues associated with the aforementioned VOC models include (Lewis 1999):

- The data used to develop the VOCs relationships are based on the number of surveys around the world and the transferability of the relationships without calibration could be questioned.
- Most of the VOC relationships depend on the assumed vehicle class, age, road class etc, which reduces the accuracy of the estimated VOCs components.
- Newer energy efficient vehicles are not included in developing the VOCs relationships. The newer vehicles could have significant impacts on the accuracy of the estimates.

Various models have also been developed to specifically calculate the VOCs due to work zone operations. These models include speed change vehicle operating costs, stopping vehicle operating costs, and idling vehicle operating costs (Morgado & Neves 2008, Lamptey et al. 2004). Each of these models is described in detail below.

1. Speed Change VOCs - VOCs associated with deceleration from the upstream approach speed to the work zone speed, and then acceleration back to the approach speed after leaving the work zone (Lamptey et al. 2004). It is given by Equation 2.15:

$$\text{Speed Change VOC} = \sum_{i}^{n} \left(Nveh_i * USCCost_i \right) \qquad \text{Equation 2.15}$$

Where, $Nveh_i$ is the number of vehicles affected by the speed change for a vehicle class i in 1000 vehicles;

$USCCost_i$ is the added cost of speed changes for vehicle class i in $/1000 vehicles;

i is the vehicle class;

n is the number of vehicle classes

2. Stopping VOCs – These are the additional VOCs associated with stopping from the upstream approach speed and accelerating back up to the approach speed after traversing the work zone (Lamptey et al. 2004). It is given by Equation 2.16:

$$\text{Stopping VOC} = \sum_{i}^{n} \left(Nveh_i * USCost_i \right) \qquad \text{Equation 2.16}$$

Where, $USCost_i$ is the added cost of stopping for vehicle for vehicle class i (hours/1000 vehicles)

3. Idling VOCs – These are the additional VOCs associated with stop-and-go driving in the queue (Lamptey et al. 2004). It is given by Equation 2.17:

$$\text{Idiling VOC} = \sum_{i}^{n} \left(Nveh_i * UICost_i \right) \qquad \text{Equation 2.17}$$

Where, $UICost_i$ is the added cost of idling for the vehicle class i ($/1000 vehicle-hour)

2.8 Pavement Treatment Effectiveness

The effectiveness of pavement treatments can be measured in the short-term and/or the long-term. Short-term benefits are defined by the immediate improvement to the pavement conditions and rates of deterioration, while long-term benefits are defined over the service life of the pavement section by the performance and extension in service life. The costs can also be short-term (individual treatment) or long-term (LCCA). The benefits and their relation to effectiveness are discussed in the next few subsections.

2.8.1 Short-term Benefits of Pavement Treatments

The short-term benefits of pavement treatments can be measured by the immediate improvement in the pavement condition and by the immediate reduction in the pavement rate of deterioration. The immediate improvement in condition is referred to as the performance jump (PJ) (see Figure 2.39). The PJ could be expressed as the difference between the conditions, the ratios of conditions, or the percent change in condition before and after a treatment. The PJ could then be modeled with other explanatory variables (before treatment conditions, treatment type, etc.) to help estimate the benefits of different treatments applied to different pavements (Culluci-Rios & Sinha 1985, Lytton 1987, Markow 1991, Rajagopal & George 1991). For example, a diamond grinding may cause roughness PJ relative to the before treatment roughness.

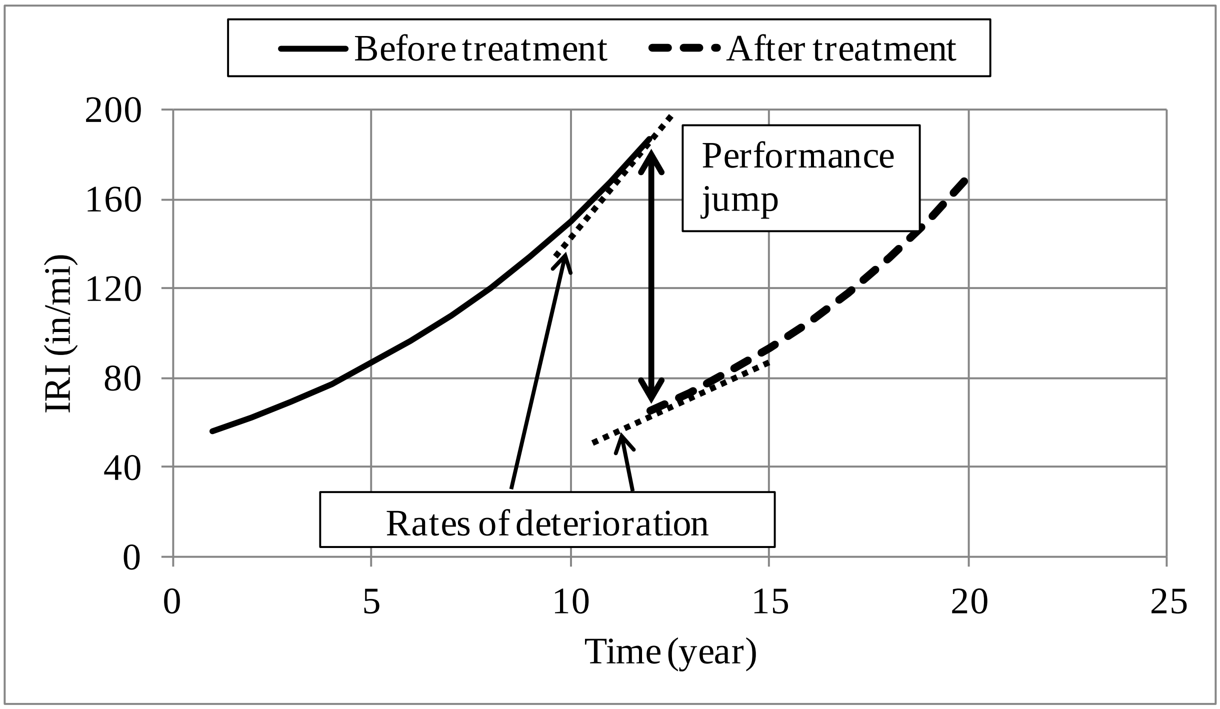


Figure 2.39 Short-term measures of pavement treatment effectiveness

Others have expressed the PJ measured from different points in time other than immediately before and after a treatment. For example, the PJ could be measured between the most recent survey before the treatment and the next survey after treatment. This is often necessary because SHAs don't typically conduct condition survey immediately before and after treatments (Labi & Sinha 2003). Depending on the data available, the SHA could measure the PJ from the most applicable point before the treatment to the most applicable point after the treatment. The proximity of the data points in time to the treatment time will affect the value of the PJ, hence, SHAs should standardize the procedure.

The other measure of short-term treatment benefit, deterioration rate reduction (DRR), is often considered a short-term benefit, despite the fact that it can only be properly determined through long-term analyses (Labi & Sinha 2003). The physical deterioration rate is calculated immediately before and after application of a pavement treatment (see Figure 2.39), but the data required to make the calculation must be acquired over the long-term. Minimum of three data points before and after the treatment are required to define the performance curves, which are used to calculate the rate of deterioration or slope. Therefore the change in rate of deterioration can be considered as a short- or long-term benefit. The change can be expressed as the difference between the rates of deterioration, the ratios of rates of deterioration, or the percent change in rates of deterioration before and after a treatment.

2.8.2 Long-term Benefits of Pavement Treatments

The long-term benefits of pavement treatments are measured over the service life of the treatment or of the pavement segment as a whole. The benefit can be expressed as the SLE or in terms of the pavement condition or performance area, as discussed in section 2.5.6.2. The long-term benefits could also be modeled with other explanatory variables (before treatment conditions, treatment type, etc.) to help estimate the benefits of different treatments applied to different pavements (Al-Mansour et al. 1994).

2.8.3 Measures of Pavement Treatment Effectiveness

Due to various constraints (such as data availability) various SHAs use various methods to estimate the effectiveness of pavement treatments. The methods include benefit or cost only, and cost-and-benefits for a given treatment or preservation strategy. Performance measures, data requirements, and the suitability of each method are listed in Table 2.34 (Khurshid et al. 2009).

The effectiveness of pavement treatments are often determined simply based on the benefits gained, as mentioned above. The benefit, however, does not indicate effectiveness relative to cost, which is the main constraint in all SHAs. Most literature agree that treatments applied to pavements in better conditions will precipitate more benefits (Al-Mansour & Sinha 1994, Labi & Sinha 2003 and 2004); however, the cost of the treatment is also a function of the condition as more or less repairs are required to prepare the pavement prior to the treatment. Additionally, the time-value relationship with money will affect the cost of the treatment and therefore the cost-effectiveness. Therefore, benefits must be compared relative to the costs to determine the cost-effectiveness of the treatment (Khurshid et al. 2009).

2.9 PMS Data Integration

Data integration is "the process of combining or linking two or more data sets from various sources to facilitate data sharing, promote effective data gathering and communication, and support overall information management activities in an organization" (USDOT 2001). SHAs typically have several offices or divisions that handle different tasks, such as traffic, PMS, safety, design, maintenance, construction, etc. Each division collects and utilizes information and data, and the management of this data is critical to the effectiveness of the SHA. The integration of all data within a SHA is necessary to monitor and report on the conditions and performance of the network, develop performance objectives and measures, identify cost-effective strategies, and assess different types of assets. For example, the maintenance division needs to know the conditions of the assets to determine which assets require maintenance. Likewise, the PMS division or office needs to know where maintenance activities were conducted to develop and

Table 2.34 Typical Pavement Treatment Effectiveness Measures (Khurshid et al. 2009)

Benefits only				
Evaluation period	**Performance measure**	**Requirements**	**Output**	**Suitability**
Short-term	Performance jump	Pre and post treatment condition values	Instantaneous improvement in pavement condition	Highly recommended
	Deterioration rate reduction	Pre and post treatment performance curves	Reduction in the rate of pavement condition deterioration due to a treatment	Recommended supplementary performance measure
Long-term	Treatment service life	Post treatment condition data and performance model	Extended pavement life due to a treatment	•Highly recommended •When post treatment condition data is available to develop reliable performance model
	Increased average pavement condition over treatment life	Post treatment condition data and/or performance model and service life	Estimate of the average improved pavement condition over treatment service life	Recommended as supplementary performance measure
	Area bound by treatment performance curve	Post treatment performance model and service life	Surrogates the monetized user benefits	•Highly recommended •When user benefits are difficult to be monetized appropriately

Table 2.34 Cont'd

Cost Only				
Evaluation period	**Performance measure**	**Requirements**	**Output**	**Suitability**
Short-term	Agency costs	Construction costs	Costs incurred by the agency during intervention	•Highly recommended •Essential part of cost analysis
	User costs	Work-zone user costs	Costs incurred by the facility user during intervention	Recommended because it presents a measure of motorist inconvenience during the intervention
Long-term	Agency costs and monetized benefits	Maintenance expenditure over the service life of a treatment or over the time span between two treatments	•Life cycle costs incurred by the agency •Reduced maintenance costs over life cycle	•Highly recommended •Essential part of cost analysis •Estimation of agency benefits is based on the condition of appropriate monetization
	User costs and monetized benefits	VOC, crash costs over the service life of a treatment or over the time span between two treatments	•Life cycle costs incurred by the user •User costs savings due to improved infrastructure condition over life cycle	•Recommended because it presents a measure of long term benefits to public due to an intervention •Recommended when user benefits can be appropriately monetized
	Life-cycle costing	Interest rate, initial construction and maintenance costs, salvage values, analysis period and/or estimated treatment service life	•Represents the life cycle costs either as EUAC or present worth of costs •Identify the alternative with lowest life cycle costs	For long term evaluation of treatments/strategies with different service lives, initial construction and maintenance costs, and salvage values but same levels of benefits

Table 2.34 Cont'd

Cost-and-benefits (cost effectiveness)				
Evaluation period	**Performance measure**	**Requirements**	**Output**	**Suitability**
Short-term	Performance jump/total initial cost	Performance jump and total initial unit cost of treatment	Performance jump per total initial unit cost	Highly recommended as supplementary performance measure
	Deterioration rate reduction/total initial cost	Deterioration rate reduction and total initial unit cost of treatment	Deterioration reduction rate per total initial unit cost	Recommended as supplementary performance measure
Long-term	EUAC	Unit cost of treatment, expected life of treatment	Unit cost per expected life of treatment	•For initial estimates •Recommended as supplementary performance measure
	Longevity cost index	Present value of unit cost over life of treatment, traffic loading and life of the treatment	Relates present value of cost of treatment to life and traffic	Recommended as supplementary performance measure
	Benefit-cost analysis	Non-monetized benefits and life cycle costing	•Treatment benefits per unit life cycle cost •Treatment benefits and cost product	•Highly recommended •When user benefits are difficult to be monetized

Table 2.34 Cont'd

Cost-and-benefits (economic efficiency)				
Evaluation period	**Performance measure**	**Requirements**	**Output**	**Suitability**
Economic efficiency	Net present value	Costs and monetized benefits, interest rate, analysis period and/or estimated treatment service life	•Present worth of all benefits after deduction all costs •Highest net present value is the best	Highly recommended as it provides the value of project at the base year
	Present worth of costs	Initial and maintenance costs, interest rate, analysis period and/or estimated treatment service life	•Present worth of all costs Least present worth cost is the best	Recommended when alternatives have different service lives but same levels of benefits
	Benefit/cost ratio (present worth, present value, or EUAC)	EUAC or net present value or present worth of costs and monetized benefits	Monetized benefit per unit cost of investment	When benefits can be monetized explicitly

calibrate treatment strategies (USDOT 2001, FHWA 2008).

Data integration promotes the following benefits (USDOT 2001):

1. Enhance intra communication.
2. Increase availability/accessibility of data.
3. Improve timeliness of data inclusion into the database.
4. Improve accuracy, correctness, and integrity of data.
5. Improve consistency and clarity of data between various sources.
6. Increase completeness of the database.
7. Reduce duplication of data collection.
8. Expedite processing of data and turnaround time.
9. Decrease data acquisition and storage costs.
10. Facilitate informed and defensible decisions based on complete data sets.
11. Enhance integrated decision-making between departments.

2.9.1 PMS Data Items

The data items required by one division of a SHA are likely different from those of another. However, there is no reason to keep data separate since the objectives of the SHA are shared by all divisions. For example, the optimum set of pavement data items constitutes all of the information and data from every division and office of the agency. Without data such as pavement cross-section, materials, rehabilitation, preservation, maintenance, and their associated costs the pavement condition and distress data cannot be properly analyzed. Example of the optimum set of data, with respect to pavement conditions, is listed in Table 2.35 (Baladi et al. 2009).

Table 2.35 Comprehensive data elements

Data items	Example uses of data
Inventory data	
• Reference location	Locate pavement segments and link various data files
• Pavement type	Deterioration models
• As constructed asphalt concrete layer	
○ Layer thickness	Backcalculation of layer moduli and deterioration models
○ Material properties	Deterioration models and quality control
○ Asphalt mix data	Deterioration models and quality control
○ Number and thicknesses of AC courses	Deterioration models and quality control
○ QC and QA data	Deterioration models
○ Cost per ton	Life cycle cost
○ Deviation from design	Link pavement design to management
• As constructed PCC layer	
○ Layer thickness	Backcalculation of layer moduli and deterioration models
○ Concrete properties	Deterioration models and quality control
○ Concrete mix data	Deterioration models and quality control
○ Joint spacing	Deterioration models
○ Dowel and tie bars	Deterioration models
○ QC and QA data	Deterioration models
○ Cost per ton	Life cycle cost
○ Deviation from design	Link pavement design to management
• Base layer	
○ Layer thickness	Backcalculation of layer moduli and deterioration models
○ Material properties	Deterioration models and quality control
○ QC and QA data	Deterioration models and quality control
○ Cost per ton	Life cycle cost
○ Deviation from design	Link pavement design to management
• Subbase layer	
○ Layer thickness	Backcalculation of layer moduli and deterioration models
○ Material properties	Deterioration models and quality control
○ QC and QA data	Deterioration models and quality control
○ Cost per ton	Life cycle cost
○ Deviation from design	Link pavement design to management

Table 2.35 Cont'd

Data items	Example uses of data
Inventory data continued	
• Roadbed soil	
○ Soil classification	Deterioration modeling
○ Roadbed modulus	Backcalculation of layer moduli and deterioration models
○ Hydraulic conductivity	Deterioration models and drainage design
○ Cost of rolling per lane-mile	Life cycle cost
• Traffic	
○ ADT	Capacity, safety and planning
○ Mixed traffic volume and weight	Traffic spectrum for design purposes using the M-EPDG
○ ESAL	1993 AASHTO Design Guide and deterioration models
• Environment	
○ Daily low and high temperatures	Causes of pavement conditions or distresses
○ Rain fall	Causes of pavement conditions or distresses
○ Snow fall	Causes of pavement conditions or distresses
○ Depth of frost penetration	Causes of pavement conditions or distresses and deterioration models
Cracking Data • Alligator s (bottom up) • Alligator cracks (top-down) • Block • Blow-up • Corner break • Durability "D" • Longitudinal • Mat • Reactive aggregate • Reflective • Slippage • Transverse	Performance models, RSL, and selection of rehabilitation action …
Joint Data • Construction joint in AC pavements • Transverse joints in PCC pavements • Longitudinal joints in PCC pavements • Joint spalling	Performance models, RSL, and selection of rehabilitation action …

Table 2.35 Cont'd

Data items	Example uses of data
Sensor data	
• Longitudinal profile (IRI) • Transverse profile (Rut depth) • Faulting at joints • Faulting at cracks • Deflection data	Performance models, RSL, and selection of rehabilitation action …
Other condition data	
• Asphalt hardening/oxidation	Performance models and selection of rehabilitation action
• Bleeding	Safety problem
• Corrugation	Performance models and selection of rehabilitation action
• Depression	Performance models and selection of rehabilitation action
• Lane/shoulder drop off or heave	Safety problem
• Lane/shoulder separation	Safety problem
• Patching	Performance models and selection of rehabilitation action
• Polished aggregate	Safety problem
• Potholes	Safety and ride problem, effects of freeze-thaw cycles
• Pumping	Voids under slab, select the proper action
• Raveling	Safety and ride quality issues
• Scaling	Concrete mix and construction problems
• Spalling	Deterioration rates
• Segregation	Causes of cracks and potholes
• Swell	Lower layer problems, selection of proper action
• Weathering/stripping	Presence of water under the asphalt mat
Rehabilitation actions	
• Type	Determine its service life and performance
• Materials	Deterioration models
• Year of inception	Determine the pavement conditions at inception
• Year of construction	Determine the pavement prior and after construction
• Expected/design life	Link performance to design
• Location reference	Identify the proper pavement project/segment
• Cost	Life cycle cost

Table 2.35 Cont'd

Data items	Example uses of data
Pavement preservation actions	
• Type	Determine its service life and performance
• Materials	Deterioration models
• Year of inception	Determine the pavement conditions at inception
• Year of construction	Determine the pavement prior and after construction
• Expected/design life	Link performance to design
• Location reference	Identify the proper pavement project/segment
• Cost	Life cycle cost
Pavement maintenance actions	
• Type	Determine its service life and performance
• Materials	Deterioration models
• Year of application	Determine the effects on the pavement conditions
• Expected/design life, when applicable	Link performance to design
• Location reference	Identify the proper pavement project/segment
• Cost	Life cycle cost
Other data	
• Date of data collection	Safety versus pavement conditions
• Air and/or pavement temperature	Impact of specifications on performance
• Reference location*	Track the propagation of distress
• Accident type, date and location	Effects of the provisions on pavement performance
• Material specifications	Seasonal impacts on the data
• Special pavement project provisions	Effects of temperature on crack opening
* It is desirable to include reference location for the beginning and ending of each condition along the pavement structure. Such data are essential to study the progression of cracks and other defects from low to medium and to high severity levels. Most semi-automated equipment that are equipped to videotape the pavement surface or to measure the longitudinal and transverse profiles are equipped with GPS or GIS unit and are capable of recording positions every 6-nches along the pavement.	

2.9.2 PMS Data Linkage, Storage, Accessibility

To make all of the various types of data easily accessible they must be linked by a common reference system. The common reference system for a SHA is the location reference system (mile posts, GPS, GIS, etc.), which could serve to link all data within the SHA. Note that the data do not have to be stored in the same location, but must be able to be queried upon to bring all applicable data to the user (USDOT 2001).

Another obstacle facing the data integration process is storing and accessing data which comes in various formats, such as paper files, common computer programs such as Microsoft Excel, department specific data sheets, etc. Two types of database are available for this task, fused and interoperable. Fused databases convert all data sources to a common database with a single user interface. Once all data is integrated the old systems are discarded and all SHA personnel must learn the new system. On the other hand, the interoperable database does not centralize the data, but rather centralizes communication between existing databases. The user can retain control of their database and also have access to other SHA databases. The type of integrated database to be implemented by a SHA depends on the structure of the SHA, and the implementation process will likely encounter some or all of the following challenges (USDOT 2001):

1. Heterogeneous data should be reduced to single sets without duplication.
2. Poor quality data should be cleaned from the database.
3. Lack of storage capacity should be avoided to allow inclusion of all data sources.
4. Unexpected costs can occur in the data transformation process.
5. Inadequate cooperation from agency personnel.
6. Lack of data management expertise could prohibit the development of the database.

Nevertheless, the integration of all data within a SHA will promote intra communication and increase the cost-effectiveness of the SHA. Integration will allow SHA personnel to study the effectiveness of their work and how it relates to the SHA as a whole. Likely the largest obstacle is training the SHA staff and convincing them to use, accept, and share data with other divisions. This can be accomplished by training and good data management procedures (USDOT 2001).

CHAPTER 3

DATA MINING

3.1 Introduction

The pavement condition and distress, treatment, and cost data used in this research study were obtained from the pavement management system (PMS) units of the following State Highway Agencies (SHAs) and pavement studies.

- The Colorado Department of Transportation (CDOT)
- The Louisiana Department of Transportation and Development (LADOTD)
- The Michigan Department of Transportation (MDOT)
- The Washington State Department of Transportation (WSDOT)
- The Minnesota Road Research Project (MnROAD)
- The Long Term Pavement Performance (LTPP)

The geographical locations of the four SHAs, MnROAD, and the LTPP pavement sections are distributed throughout the USA with varying network size, traffic, climate, etc. Hence, the pavement data are somewhat representative of the entire pavement network in the USA.

At the onset of this research, pavement condition and distress data for only nine pavement sections were requested from each of the four SHAs. These data were received and analyzed; later the entire PMS databases were requested to broaden the extent of the analyses and to incorporate the entire sets of available data from each SHA. The databases were received in various formats (Microsoft Excel spreadsheets, Microsoft Access files, text documents, and so forth). Further, the pavement condition and distress data were expressed using various measurement units (SI and English) and various formats such as area, length, count, average, maximum, etc. The data were converted to Microsoft Excel files and the measurements were unified for compatible data mining and analyses.

3.2 Data Format, Restructuring, and Unification

The first step in the support of the pavement condition and distress data analyses, discussed in Chapter 4, was to convert each of the four PMS databases obtained from four SHAs, to one uniform format for analyses. The database conversions were conducted as listed below:

1. The pavement condition and distress data from Colorado, Louisiana, and Washington were received in Microsoft Access format. The data analyses, described in Chapter 4, were conducted using the Matlab computer program, which cannot read Microsoft Access files. Hence, the data were converted to Microsoft Excel files. The pavement condition and distress data received from MDOT were stored in several formats including Microsoft Excel and text files such as EI2, WI2, ER3, WR3, etc., and were left in their original formats.
2. The data from Washington and Louisiana were converted from Microsoft Access format to Excel spreadsheets and stored in separate Excel files, one file for each data collection year. This was accomplished by sorting the data by the data collection year and copying each year individually to a new Excel file. The data from Colorado were already stored by year.

As stated earlier, the pavement condition and distress data were reported using various units of measurements and measurement types as listed in Table 3.1. Since the data from each SHA were used in this study, and in order to conduct uniform analyses, some of the data elements were re-arranged and/or re-configured to produce uniform data structure using uniform units of measurement. The restructuring and re-configuration did not affect the accuracy and/or the variability of the data. Such restructuring and re-configuration were dependent on the pavement condition or distress type as detailed below.

Table 3.1 Pavement condition and distress data stored by various SHAs

Condition or distress	Units and type of measurements reported by SHAs				Units used in the analysis
	SI	English	Types of measurements		
Roughness	m/km	inch/ mile	Each wheel path	Average wheel path	Average IRI (inch/mile)
Rut depth	mm	inch	Maximum and average rut depth	Average rut depth	Average rut depth (inch)
Alligator cracks	m, m^2	ft, ft^2	Percent of length	Percent of area	Cumulative length along the pavement segment (ft)
Longitudinal cracks	m	ft	Cumulative length along the pavement segment		Cumulative length along the pavement segment (ft)
Transverse cracks	m	ft	Count	Cumulative length	Cumulative length along the pavement segment (ft)

- Roughness – Each of the four SHAs collects pavement roughness data in terms of the International Roughness Index (IRI). The data are recorded in either SI units (m/km) or English units (in/mi). In this study the data reported in SI units were converted to English units. In addition, the roughness data for each 0.1 mile long pavement segment were measured and reported, depending on the SHA, for each wheel path and/or the average IRI along the pavement segment in question. Hence, the average IRI measurements of the two wheel paths per pavement segment were used in this study.
- Rut Depth – The rut depth data are collected and stored in either SI units (mm) or English units (inch). In this study the data reported in SI units were converted to English units. In addition, for each 0.1 mile long pavement segment, the rut depth data were measured and reported, depending on the SHA, as the average of each wheel path, the average of the pavement segment, or the average and the maximum rut depth of the pavement segment in question. The average rut depth measurements per pavement segment were used in this study.

- Alligator Cracking - The alligator cracking data are collected and stored in terms of length (m or ft), area (m^2 or ft^2), percent of the pavement length, or percent of area. In addition, the alligator cracking data for each 0.1 mile long pavement segment were measured and reported, depending on the SHA, over the entire pavement width (12 feet) or in the wheel paths (3 foot width per wheel path) of the pavement segment in question. In this study the alligator cracking data were converted to linear feet by dividing the area by the width. For example, a pavement segment with 600 ft^2 of alligator cracking, and full lane width measurement criteria, was divided by 12 feet, yielding 50 linear feet of alligator cracking.
- Longitudinal Cracking - The cumulative length of longitudinal cracks are collected and stored in either SI units (m) or English units (ft). In this study the cumulative longitudinal crack length data along a pavement segment reported in SI units were converted to English units.
- Transverse Cracking - The transverse cracking data are collected and stored in either SI units (m), English units (ft), or by count. In this study the data reported in SI units were converted to English units. In addition, the transverse crack data for each 0.1 mile long pavement segment were measured and reported, depending on the SHA, by length categories (such as 0 to 1 foot, 2 to 3 feet, etc.), by the number of transverse cracks without crack lengths, or by the cumulative length of transverse cracks. The cumulative length of transverse cracks in feet was used in this study. The cumulative length of transverse cracks was calculated, when applicable, by multiplying the crack count by the average reported length. For example, one crack in the 0 to 1 foot long category was counted as 0.5 foot long, while a crack in the 6 to 12 feet category was counted as 9 feet long crack. Transverse crack data reported by count only (no crack lengths were included), were assumed to extend across the entire lane and

therefore their length is 12 feet.

- Severity levels – Most cracking data are stored under three separate severity levels, low, medium, and high. The problem of such data is that the crack rating is a function of the judgment of the surveyor who is reviewing and digitizing the electronic images. Such judgment is a function of the degree of training and experience of the surveyors. In addition, the same pavement segment may not be reviewed by the same surveyor each year or each data collection cycle. Thus, a crack may be labeled high severity in one year and medium next year and vice versa. Figures 3.1 and 3.2 depict the time-series data for each transverse crack severity level along a portion of highway 24 in Colorado. The data in the figures show that:

  1. The transverse crack length changes severity level from year to year without a pavement treatment. For example, the high severity transverse crack length is about 150 feet in year 2, then only about 10 feet in year 3, and is finally absent in year 4. Likewise, the medium severity transverse crack length is about 130 feet in year 2, 150 feet in year 3, and only about 10 feet in year 4. On the other hand, the low severity transverse crack length increases from about 60 feet, to 110 feet, and to 310 feet over the same time period. The reduction in the lengths of medium and high severity transverse cracks could be attributed to the rater assigning low severity to the same cracks which were previously assigned medium or high severity. This inconsistency could be addressed through enhanced crack rating quality control.
  2. The individual severity levels show high variability when modeled in time-series, as indicated by the exponential models determined for each transverse crack length severity level. In fact, the models indicate that the medium and high severity transverse crack

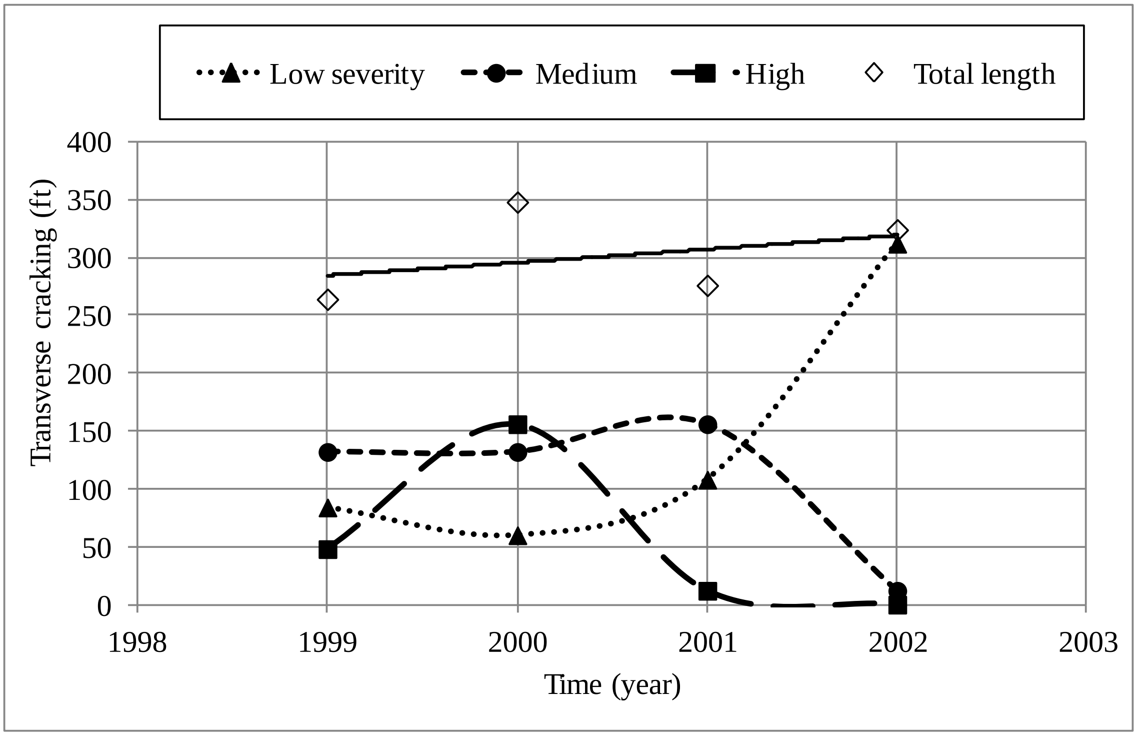


Figure 3.1 Time-series transverse cracking data for each severity level and the sum of all levels, Colorado, HWY 24, direction 2, BMP 329.9

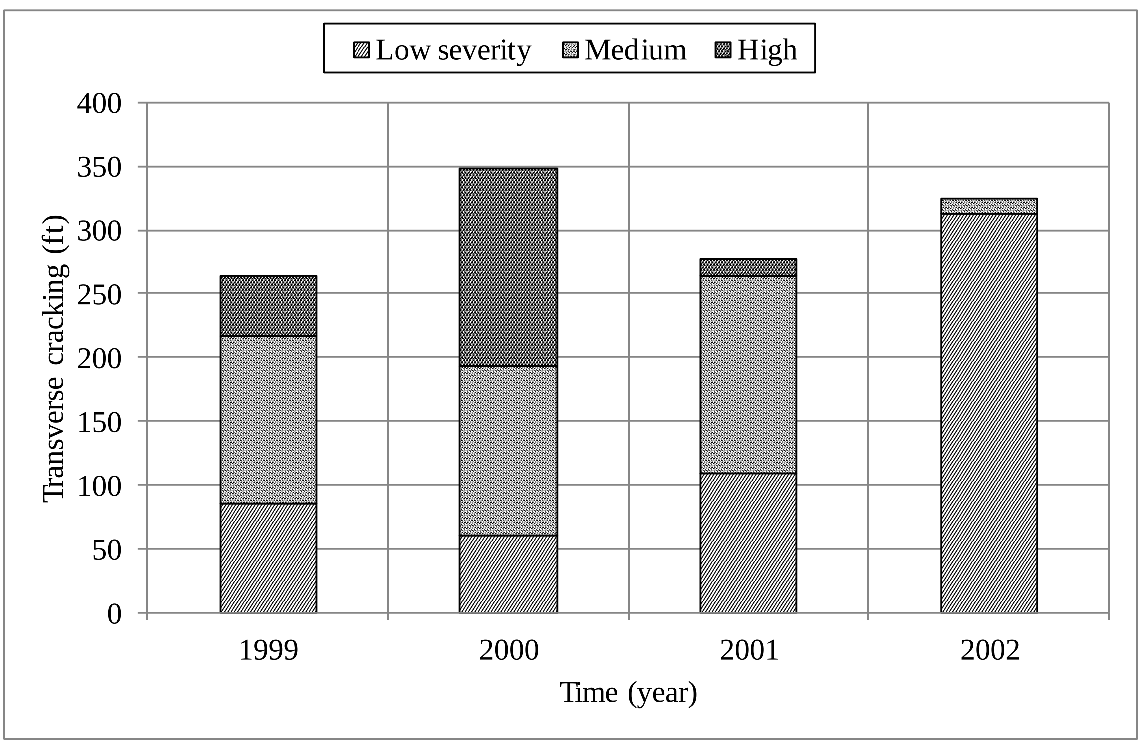


Figure 3.2 Cumulative time-series transverse cracking data showing individual transverse crack severity level and the sum of all severity levels, Colorado, HWY 24, direction 2, BMP 329.9

length is decreasing with time without a pavement treatment. However, it was found that expressing the time-series cracking data as the sum of the three severity levels yielded much less data variability, as evidenced by the model of the total transverse crack length.

Nevertheless, the crack severity levels could be used to roughly estimate the amount of work to be done. For example, so many feet of cracks in the medium and high severity levels need to be sealed or patched. Low severity cracks are typically not sealed or patched. Likewise, for concrete pavements, low severity transverse cracks may be subjected to dowel bar retrofit, while medium and high severity cracks are typically not. In this study, analysis of the time-series cracking data was based on the sum of all severity levels.

A summary of the data characteristics and information of each of the four SHAs and how they were re-arranged and re-structured to yield uniform set of data for analysis are detailed in the next few subsections.

3.2.1 Pavement Distress and Condition Data from Four State Highway Agencies (SHAs)

Table 3.2 lists the pavement condition and distress types, severity levels, and measurement units used by each of the four SHAs. For each SHA, some statistical information regarding the pavement network and a summary of the data characteristics and how they were re-formatted and re-structured to the uniform set of data are detailed in the next few subsections.

a) Colorado Department of Transportation (CDOT)

CDOT has 9,134 miles (22,912 lane-miles) of pavement and 3,406 bridges under its jurisdiction. The annual traffic is in excess of 48 billion vehicle-miles. Forty percent of that traffic is carried on the Interstate system (I-25, I-70, I-76, I-225, and I-270) which accounts for about ten percent of the network length. In 2009, 7.2 million miles of pavement were cleared of snow and 248,000 tons of asphalt materials and 178,800 gallons of liquid asphalt were used to repair damaged pavement surfaces (Hartgen et. al. 2009, CDOT 2010).

Pavement condition and distress data for three flexible, three rigid, and three composite pavement sections were originally requested and received from CDOT. Later, upon request CDOT provided their entire pavement condition and distress database covering the period from 1998 through 2009. The condition and distress data were collected every year, every other year, or every third year depending on the road and direction of travel. The data were collected on a continuous basis and recorded in the database for each 0.1 mile long pavement segment. The applicable data types, severity levels, and measurement units contained in the database are listed in Table 3.2. The data were adjusted where necessary to unify the measurement units and severity levels as discussed above. The alligator cracking data were provided as lane area ($feet^2$) and were divided by the lane width (12 feet) to obtain linear feet. The reported number of transverse cracks was multiplied by the lane width (12 feet) to obtain linear feet. The IRI, longitudinal cracking, and rut depth data were not adjusted. Note that no distress severity levels were reported with the full database. Examples of the measured and formatted data are listed in Tables A.1 and A.2 of Appendix A and the entire database is available upon request from the Department of Civil & Environmental Engineering at Michigan State University (MSU) (CDOT 2010).

b) Louisiana Department of Transportation and Development (LADOTD)

LADOTD has 15,987 miles (38,458 lane-miles) of pavement and 8,060 bridges under its jurisdiction. The big number of bridges is due to the large areas of land near the Mississippi River and Delta, which are near or below sea level. The Interstate system (I-10, I-12, I-20, I-49, I-55, I-59, I-110, I-210, I-220, I-310, I-510, I-610, and I-910) makes up about six percent of the

Table 3.2 The pavement condition and distress data received from the three SHAs

SHA	Pavement condition information		
	Condition or distress type	Severity level	Measurement unit
CDOT	Alligator cracking		$Feet^2$/ segment
	IRI		Inch/ mile
	Longitudinal cracking		Feet/ segment
	Rut depth		Inch
	Transverse cracking		Count/ segment
LADOTD	Alligator cracking	Low, medium, and high	$Feet^2$ (wheel paths)/ segment
	IRI		Inch/ mile
	Longitudinal cracking	Low, medium, and high	Feet/ segment
	Rut depth in right or left wheel path		Inch
	Transverse cracking	Low, medium, and high	Feet/ segment
MDOT	Alligator crack consists of two categories (right and/or left wheel path)	25 levels depending on the length and associated distress	Percent of length
	IRI		Inch/ mile
	Longitudinal crack consists of 3 categories for rigid and 5 for flexible & composite pavements depending on location in the lane	49 levels for flexible, rigid and composite pavements depending on crack length and its associated distress	Percent of length
	Rut depth		Inch
	Transverse crack consists of two categories for rigid, composite and flexible pavements depending on crack opening for rigid and composite, and crack irregularity for flexible pavements	Twelve levels for rigid and composite and 16 levels for flexible depending on length and width of the associated distresses	Count/ segment
WSDOT	Alligator cracking	Low, medium, and high	Percent of 2 Wheel paths of a segment
	IRI		Meter/ kilometer
	Longitudinal cracking	Low, medium, and high	% of segment
	Rut depth		Millimeter
	Transverse cracking	Low, medium, and high	Count/ segment

pavement network length. In 2006, about six percent of the network was maintained through sealing or resurfacing (LADOTD 2007, Hartgen et. al. 2009).

Pavement condition and distress data for three flexible, three rigid, and three composite pavement sections were originally requested and received from LADOTD. Later, upon request LADOTD provided their entire pavement condition and distress database covering the period from 1995 through 2009. The condition and distress data were recorded in 1995, 1997, 2000, 2003, 2005, 2007, and 2009. The data were collected on a continuous basis and stored in the database for each 0.1 mile long pavement segment. The applicable data types, severity levels, and measurement units contained in the database are listed in Table 3.2. The alligator cracking data were provided as wheel path area (feet^2) and were divided by the wheel path width (3 feet) to obtain linear feet. The rut depth data were provided as the average of each wheel path and the average of the averages was taken as the average rut depth. The IRI, longitudinal cracking, and transverse cracking data were not adjusted. Note that the low, medium, and high severity distresses were summed. Example of the formatted data is listed in Table A.3 of Appendix A and the entire database is available from the Department of Civil & Environmental Engineering at MSU (LADOTD 2010).

It should be noted that rut depth data were collected by ultra-sounding in 1993 and 1995, then by 3-point laser system from 1997 to 2005, then by continuous laser after 2005. There is no correlation between equipment. An error in the rut depth collection software was discovered by the staff of LADOTD and the rut depth data collected after 2005 were corrected and inserted to replace the original data. It should also be noted that sealed cracks are not counted as cracks in the database.

c) Michigan Department of Transportation (MDOT)

MDOT has about 9,700 miles (27,503 lane-miles) of pavement and 5,400 bridges under its jurisdiction. The Interstate system consists of I-69, I-75, I-94, I-96, I-194, I-275, I-475, I-496, I-675, and I-696. In 2006, about 400 miles of pavement and 300 bridges were improved (MDOT 2007, Hartgen et. al. 2009).

Pavement condition and distress data for three flexible, three rigid, and three composite pavement sections were originally requested and received from MDOT. Later, upon request MDOT provided their entire pavement condition and distress database covering the period from 1992 through 2009. The condition and distress data were typically recorded every other year, alternating annually for each direction of travel. The data were collected on a continuous basis and recorded for each 0.1 mile long pavement segment. The applicable data types, severity levels, and measurement units contained in the database are listed in Table 3.2. The alligator and longitudinal cracking data were provided as decimal percent of the length of each segment (528 feet) and were multiplied by the segment length to obtain linear feet. The transverse cracking data were provided as the count of cracks with certain range of length and were multiplied by the average length in each range to obtain linear feet. The IRI and rut depth data were provided in inch/mile, and inch, respectively and hence, they were not adjusted. Note that the cracking data are collected under several classifications and the length was summed. Example of the formatted data is listed in Table A.4 of Appendix A and the entire database is available from the Department of Civil & Environmental Engineering at MSU (MDOT 2010).

The MDOT pavement condition and distress data are stored in several formats scattered among thousands of individual files. Each file contains portions of the pavement condition and distress database. For example, the IRI, rut depth, and cracking data are each stored separately for each 0.1 mile long pavement segment. In addition, the majority of each data type is stored in different files for each control section. Further, for a given control section and a given pavement data collection cycle, the linear location reference system in the IRI and rut depth files often do not match that in the cracking files. For example, the beginning mile points (BMPs) for the sensor collected data (IRI and rut depth) and the videotaped data (cracking) are not consistent along portions of M-39, control section 82192, direction 1. In 2001 portion of the videotaped data were collected for the pavement segments with BMPs 0.0, 0.131, 0.256 and so on; while the sensor collected data were collected for pavement segments with BMPs 0.0, 0.1, 0.2 and so on. To further complicate the situation, similar problem exists in time-series where the BMPs of the pavement segments float back and forth from one data collection cycle to the next. Figure 3.3 shows the BMPs of the first 10 pavement segments along M-39, control section 82192, direction 1 from 5 data collection cycles (1999 to 2007). The data in the figure indicate that the BMPs of the pavement segments shift significantly back and forth from one year to the next. Further, few of the pavement segments are more or less than 0.1 mile long and some of the segments were not surveyed in few years. Countless attempts were made to adjust the BMPs using specially written Matlab based computer programs to match the data for each pavement segment within each data collection cycle (sensor and videotaped data) and between various data collection cycles. Some of the data were matched by shifting each BMP along the pavement to coincide with a reference BMP (the oldest data collection cycle). In the example, the data collected from 2001 to 2007 were shifted forward or backward to match the BMPs of the data collected in 1999. Various degrees of difficulties were encountered because the required shift in the BMPs for each 0.1 mile long pavement segment within a data collection cycle is not constant. Further, for the same 0.1 mile long pavement segment, the required shift in the BMP from one data collection cycle to the next is not constant. Such shifts were often greater than 0.1 mile. Nevertheless, Figure 3.4 shows the resulting time-series longitudinal crack data after adjusting the BMP of each 0.1 mile long pavement segment by hand. The data in the figure indicate that the shifted BMPs result in highly variable time-series cracking data. Such variability did not allow:

1. Modeling the data as a function of time.
2. Calculating consistent rate of deterioration.
3. Estimating the benefits of treatments.

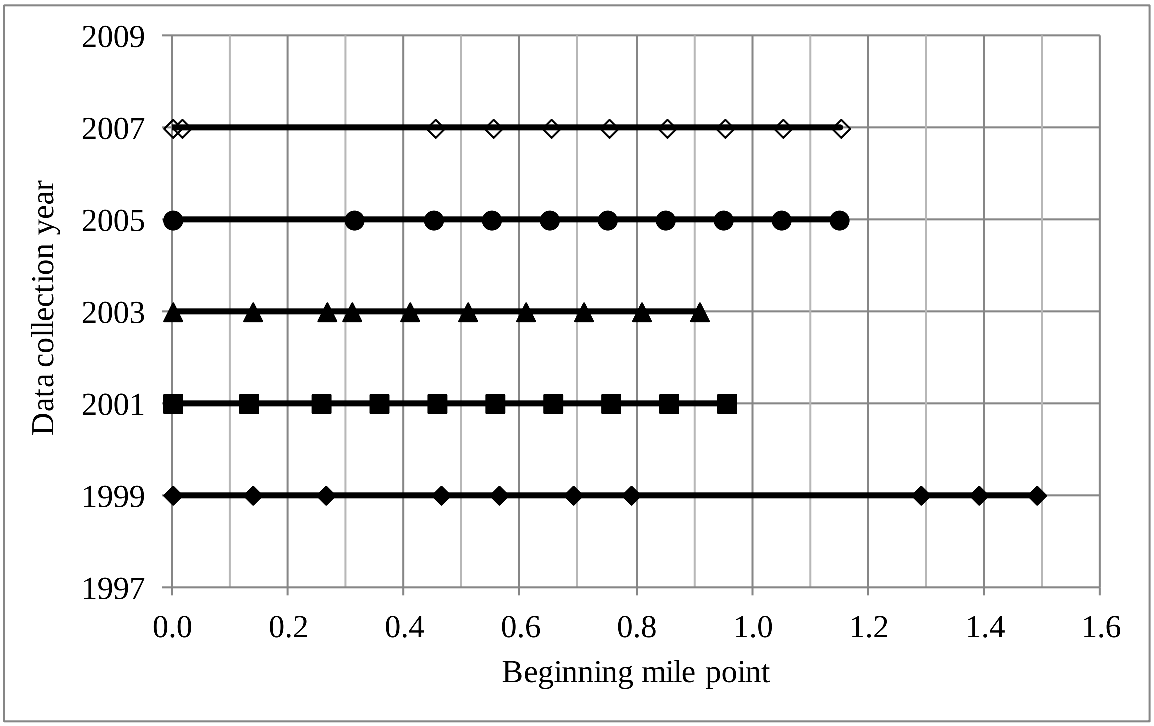


Figure 3.3 The BMP of the first ten 0.1 mile long pavement segments locations along M-39, control section 82192, direction 1, Michigan

The time-series pavement condition and distress data are spatially skewed and the confidence in the accuracy of the shifted BMPs was minimal. Once again, during about four month period, countless attempts were made to unify the BMP of each 0.1 mile long pavement segment between the various data collection cycles. Unfortunately, none of these attempts were

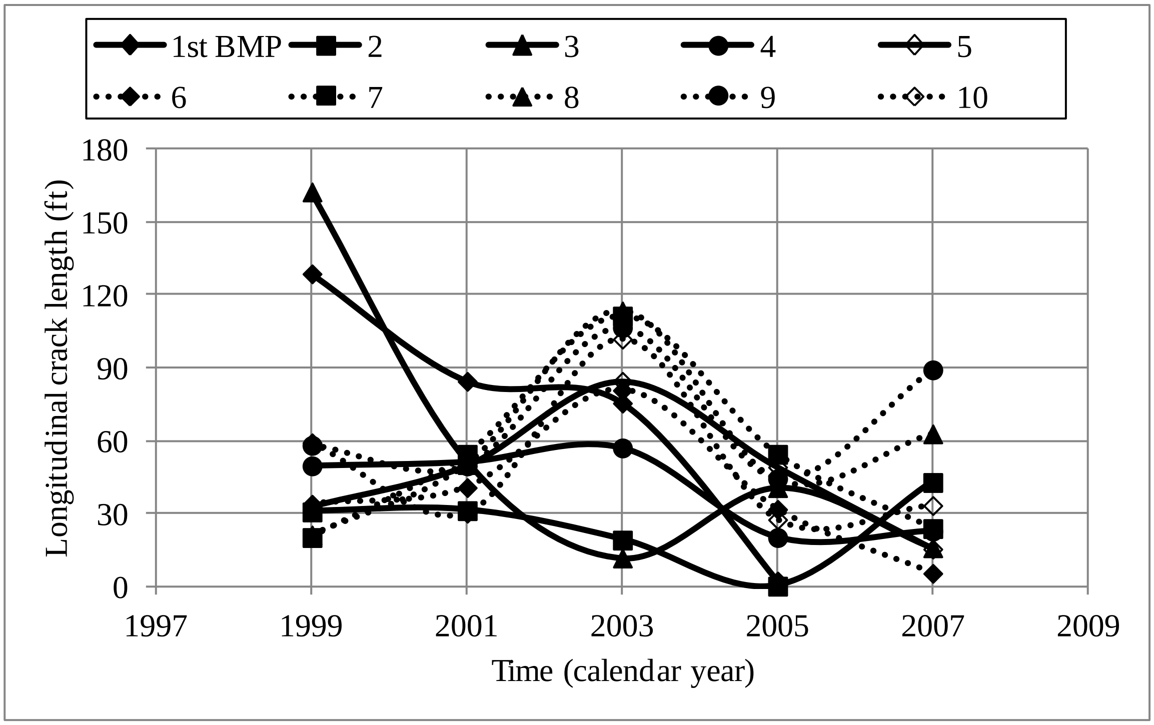


Figure 3.4 Longitudinal cracking data after adjusting the BMP for the first ten 0.1 mile long pavement segments along M-39, control section 82192, direction 1, Michigan

even partially successful. Therefore, it was decided to exclude the MDOT pavement condition and distress data from any further analysis in this study.

d) Washington State Department of Transportation (WSDOT)

WSDOT has about 7,000 miles (18,392 lane-miles) of pavement and 3,400 bridges under its jurisdiction. The Interstate system consists of I-5, I-10, I-82, I-90, I-405, and I-705 (Hartgen et. al. 2009, WSDOT 2010).

Similar to the other three states, pavement condition and distress data for three flexible, three rigid, and three composite pavement sections were originally requested and received from WSDOT. Later, upon request WSDOT provided their entire pavement condition and distress database covering the period from 1969 through 2007. The condition and distress data were collected and recorded primarily every other year from 1969 to 1988 and every year starting in 1989. The data were collected on a continuous basis and recorded for each 0.1 mile long pavement segment. The applicable data types, severity levels, and measurement units contained in the database are listed in Table 3.2. The alligator cracking data were provided as percent of the length of the two wheel paths in the segment (1,028 feet). In this study, the data were divided by 100 and multiplied by the segment wheel path length to obtain linear feet. The Longitudinal cracking data were provided as percent of the length of each segment (528 feet). Once again, the data were divided by 100 and multiplied by the segment length (528 feet) to obtain linear feet. The transverse cracking data were provided as the count of the number of cracks and were multiplied by the lane width (12 feet) to obtain linear feet. Note that, in the analysis of the cracking data, the sum of the low, medium, and high severity levels was used. The IRI and rut depth data, which are recorded in SI units (m/km and mm) were converted to English units (in/mile and inch), respectively. Example of the formatted data is listed in Table A.5 of Appendix A and the entire database is available from the Department of Civil & Environmental Engineering at MSU (WSDOT 2010).

3.2.2 Minnesota Road Research (MnROAD)

The MnROAD test facility is located near Albertville, Minnesota. The pavement testing facility was constructed between 1992 and 1994 through a joint venture between the Minnesota Department of Transportation (Mn/DOT) and the Minnesota Local Road Research Board (LRRB). The test facility consists of two road sections divided into fifty five 500-foot long segments or “cells”. One road section is a 3.5 mile mainline section along I-94 carrying about 28,500 vehicles per day. The other is a 2.5 mile long low volume road utilizing a controlled five-axle semi tractor-trailer (MnROAD 2008, MnDOT 2011). The pavement condition and distress types, severity levels, and measurement units contained in the database are listed in Table 3.3. The longitudinal, transverse, and alligator cracking were not restructured or reconfigured. Note that, in the analysis and for each cell, the sum of the low, medium, and high severity cracking data was used. The IRI and rut depth data were converted from SI to English units. Example of the formatted data is listed in Table A.6 of Appendix A and the entire database is available from the Department of Civil & Environmental Engineering at MSU.

Table 3.3 MnROAD database items

Condition or distress type	Severity level	Measurement unit
Alligator cracking	Low, medium, and high	Feet/ cell
IRI		Meter/ kilometer
Longitudinal cracking (wheel path or non wheel path and sealed or not sealed)	Low, medium, and high	Feet/ cell
Rut depth		Millimeter
Transverse cracking (sealed or not sealed)	Low, medium, and high	Feet/ cell

3.2.3 Long Term Pavement Performance (LTPP)

The LTPP program began in 1987 as part of the Strategic Highway Research Program (SHRP). The program consists of over 2,500 asphalt and Portland cement concrete (PCC) test segments across the US and Canada. Each site is divided into 500 foot segments. These segments are monitored and data are collected for analyses of the performance of different pavement designs, rehabilitations, and maintenance activities (FHWA 2010). The applicable data types, severity levels, and measurement units contained in the database are listed in Table 3.4. The alligator cracking data were provided as area (meter^2) and were converted to English units and then divided by the lane width (12 feet), to obtain linear feet. The IRI, longitudinal and transverse cracking, and rut depth data were converted from SI to English units. Note that, in the analysis, the sum of the low, medium, and high severity cracking data were used. The entire database is available at http://www.ltpp-products.com/.

Table 3.4 LTPP database items

Condition or distress type	Severity level	Measurement unit
Alligator cracking	Low, medium, and high	$Meter^2$/ cell
IRI		Meter/ kilometer
Longitudinal cracking (wheel path or non wheel path and sealed or not sealed)	Low, medium, and high	Feet/ cell
Rut depth		Millimeter
Transverse cracking (sealed or not sealed)	Low, medium, and high	Feet or count/ cell

3.3 Treated Pavement Section Identification

Following the conversion and restructuring of the pavement condition and distress databases, exhaustive searches were conducted of the CDOT, LADOTD, and WSDOT databases and data files to identify pavement sections that were subjected in the past to one of the following six most common treatment types:

A. Thin (< 2.5 inch) hot-mix asphalt (HMA) overlay of asphalt surfaced pavement

B. Thick (≥ 2.5 inch) HMA overlay of asphalt surfaced pavement

C. Chip seal

D. Double chip seal

E. Thin (< 2.5 inch) mill and fill of asphalt surfaced pavement

F. Thick (≥ 2.5 inch) mill and fill of asphalt surfaced pavement

It should be noted that, the pavement treatment files provided by each of the three SHAs were used in the identification of treated pavement sections. This identification consists of determining the acceptable time window of the treatment calendar year such that the database likely contains three or more data collection cycles before treatment (BT) and three or more data collection cycles after treatment (AT). For example, if the database contains data collection cycles for the calendar years 1994, 1996, 1999, 2000, 2002, 2004, 2006, and 2008, then the acceptable treatment time window is between the years 2000 and 2003. This increases the probabilities that every 0.1 mile long pavement segment has three or more data points BT and AT available. In general, the section identification implies that for those states that collect data every other year, an absolute minimum of 12 years of time-series data (6 years BT and 6 years AT) are required. Whereas for the State of Washington, who collect the data every year, an absolute minimum of 6 years of data are required (3 years BT and 3 years AT).

It is important to note that, in this study as well as in others, such data search is difficult, laborious, and time consuming. The difficulties that were, or could be, encountered include:

- For some pavement sections, the PMS database of some SHAs, especially those who collect pavement condition and distress data on a bi-annual basis, did not contain the minimum of three time-series data points BT and/or AT.
- Many pavement sections were subjected to treatment more frequently than the required time span listed above.
- For a significant number of pavement sections, accurate and consistent records regarding the applications of routine, reactive, and preventive maintenance treatments do not exist in the database.
- For those SHAs who collect pavement condition and distress data using a sampling technique, the availability of three time-series data points could be significantly reduced.

For each SHA, the number of pavement sections or projects that were identified for analysis, the cumulative length of these sections for each treatment type, and the total length of all projects of all SHAs identified for analyses are listed in Table 3.5. The databases and the treatment files of the three SHAs were also searched for other pavement treatment types (dowel bar retrofit, rigid pavement rehabilitation, crack sealing, and etc.). Unfortunately, the available number of pavement sections and the data were insufficient to support their inclusion in the analyses.

Table 3.5 The number of pavement projects and total length identified in each state

<table>
<tr><th rowspan="3">Treatment type</th><th colspan="6">States and frequency of treatment (number, miles)</th><th colspan="2" rowspan="2">Total, all SHAs</th></tr>
<tr><th colspan="2">Colorado</th><th colspan="2">Louisiana</th><th colspan="2">Washington</th></tr>
<tr><th>Projects</th><th>Length</th><th>Projects</th><th>Length</th><th>Projects</th><th>Length</th><th>Projects</th><th>Length</th></tr>
<tr><td>A</td><td rowspan="2">235</td><td rowspan="2">276.4</td><td rowspan="2">129</td><td rowspan="2">485.6</td><td rowspan="2">104</td><td rowspan="2">347.6</td><td rowspan="2">468</td><td rowspan="2">1,109.6</td></tr>
<tr><td>B</td></tr>
<tr><td>C</td><td rowspan="2">168</td><td rowspan="2">1,190.4</td><td rowspan="2">187</td><td rowspan="2">968.4</td><td rowspan="2">7</td><td rowspan="2">20.4</td><td rowspan="2">362</td><td rowspan="2">2,179.2</td></tr>
<tr><td>D</td></tr>
<tr><td>E</td><td rowspan="2">5</td><td rowspan="2">14.0</td><td rowspan="2">166</td><td rowspan="2">709.1</td><td rowspan="2">25</td><td rowspan="2">106.9</td><td rowspan="2">196</td><td rowspan="2">830.0</td></tr>
<tr><td>F</td></tr>
<tr><td>Total</td><td>408</td><td>1,480.8</td><td>482</td><td>2,163.1</td><td>136</td><td>474.9</td><td>1,026</td><td>4,118.8</td></tr>
<tr><td colspan="9">Treatment type: A = Thin (< 2.5 inch) HMA overlay of asphalt surfaced pavements;
B = Thick (≥ 2.5 inch) HMA overlay of asphalt surfaced pavements;
C = Single chip seal; D = Double chip seal;
E = Thin (< 2.5 inch) mill and fill of asphalt surfaced pavements;
F = Thick (≥ 2.5 inch) mill and fill of asphalt surfaced pavements.</td></tr>
</table>

3.4 Time-Series Data Restructuring

For each treated pavement section identified above, the pavement condition and distress data for each 0.1 mile long pavement segment along the section were restructured in a time-series format. A Matlab based computer program was written to search the database and retrieve the corresponding condition and distress data based on the user inputs of:

- The BMP and the ending mile point (EMP) of the pavement project.
- The route and control section number and the direction of traffic.

For each pavement section/project, the output of the Matlab computer program consists of one Microsoft Excel file containing six worksheets; one sheet per condition and distress type and one listing the treatment type, time, and project boundaries.

3.5 Cost Data

Limited cost data were received from the three SHAs. When cost data were available they were extremely limited in detail. For example, the treatment cost consists of the total contract cost. The cost data did not include the breakdown of cost relative to materials, pre-overlay activities, safety improvement such as guardrails, signing, mobilization charges, incentive/disincentives, and so forth. Detailed cost data are required to determine the cost-effectiveness of the treatment. Nevertheless, when available, only the material and labor costs data were used to determine the cost-effectiveness of the treatment while extraneous cost items in the contract such as guard-rail and sign replacement, when available, were not included in the analyses. Similarly, the equipment mobilization cost is a significant portion of the cost of short pavement projects and small portions of long projects. In most cases, these information were not available, they should be included in the database and in the analyses. Further, since the pavement conditions and distress levels along a given pavement project vary substantially, the amount of pre-treatment work and cost would also vary. Hence, a complete set of cost data should include the cost of the treatment along each 0.1 mile of the pavement section. This would allow analysis of the optimum time at which the pavement treatment is the most cost effective.

The lack of sufficient and detailed cost data did not allow comprehensive analysis of the treatment costs in this study. Therefore, analyses of treatment benefits (effectiveness) were undertaken. The analysis procedures and the results are detailed and discussed in Chapter 4. Note that, Baladi & Dean 2011 obtained, from MDOT, general cost data of various preventive maintenance treatment actions. The data are summarized in Tables 3.6 and 3.7 for the years 2009 and 2010, respectively. The data in the tables detail the entire range of project costs, which include project locations and possible provisions of the contracts. Table 3.8 summarizes the general material costs and the total costs including materials, engineering fees, culverts and slope work, etc. Finally, equipment mobilization is typically about $50,000. The cost data (listed in Tables 3.6 through 3.8) were used to conduct life cycle cost analysis (LCCA) of light (frequent) and heavy (less frequent) pavement treatments in urban and rural areas. The analyses procedures and results are presented and discussed elsewhere (Baladi et al. 2012, Dean 2012).

Table 3.6 Typical preventive maintenance treatment costs in 2009 for the State of Michigan (Baladi & Dean 2011)

Treatment type	Cost per unit in 2009		
	Minimum	Average	Maximum
Cold milling bituminous/HMA surface	$0.26/yd^2 $0.01/ton	$0.69/yd^2 $6.34/ton	$7.43/yd^2 $11.50/ton
Non structural bituminous/HMA overlay	$39.54/ton	$52.73/ton	$100.85/ton
Over band crack fill	$2,145/ rbmi	$4,191/ rbmi	$13,150/ rbmi
Bituminous crack treatment	$3,092/ rbmi	$4,210/ rbmi	$17,000/ rbmi
Chip seal	$1.24/yd^2	$1.38/yd^2	$1.66/ yd^2
Double chip seal	$2.57/ yd^2	$2.78/ yd^2	$2.98/ yd^2
Micro-surface with Warranty	$2.55/ yd^2	$3.09/ yd^2	$3.91/ yd^2
HMA ultra-thin with low volume warranty	$2.08/ yd^2	$2.35/ yd^2	$2.70/ yd^2
HMA ultra-thin with medium volume warranty	$2.72/ yd^2	$2.86/ yd^2	$3.07/ yd^2
Paver placed surface seal	$5.19/ yd^2	$5.74/ yd^2	$6.00/ yd^2
Diamond grinding concrete pavement	$2.90/ yd^2	$3.14/ yd^2	$3.18/ yd^2
Resealing transverse pavement joints with hot-poured rubber	$0.69/ft	$0.97/ft	$2.00/ft
Resealing longitudinal joints with hot-poured rubber	$0.59/ft	$0.92/ft	$1.50/ft
Crack sealing concrete pavement	$1.00/ft	$1.12/ft	$2.00/ft
Yd^2 = square yard with 1 inch depth, rbmi = roadbed mile, ft = foot			

Table 3.7 Typical preventive maintenance treatment costs in 2010 for the State of Michigan (Baladi & Dean 2011)

Treatment type	Cost per unit in 2010		
	Minimum	Average	Maximum
Cold milling bituminous/HMA surface	\$0.01/ yd^2 \$0.01/ton	\$0.72/ yd^2 \$0.45/ton	\$7.00/ yd^2 \$1.00/ton
Non structural bituminous/HMA overlay	\$45.00/ton	\$57.54/ton	\$110.00/ton
Over band crack fill	\$2,935/rbmi	\$4,418/rbmi	\$13,600/rbmi
Bituminous crack treatment	\$1,757/rbmi	\$4,447/rbmi	\$12,541/rbmi
Chip seal	\$1.28/ yd^2	\$1.51/ yd^2	\$1.65/ yd^2
Double chip seal	\$2.69/ yd^2	\$2.75/ yd^2	\$2.81/ yd^2
Micro-surface with Warranty	\$2.55/ yd^2	\$3.48/ yd^2	\$5.00/ yd^2
HMA ultra-thin with low volume warranty	\$2.50/ yd^2	\$2.58/ yd^2	\$3.00/ yd^2
HMA ultra-thin with medium volume warranty	\$2.55/ yd^2	\$2.58/ yd^2	\$2.60/ yd^2
Paver placed surface seal	\$5.19/ yd^2	\$5.74/ yd^2	\$6.00/ yd^2
Diamond grinding concrete pavement	\$2.90/ yd^2	\$3.14/ yd^2	\$3.18/ yd^2
Resealing transverse pavement joints with hot-poured rubber	\$0.95/ft	\$1.88/ft	\$4.00/ft
Resealing longitudinal joints with hot-poured rubber	\$0.96/ft	\$1.03/ft	\$2.40/ft
Crack sealing concrete pavement	\$0.10/ft	\$0.58/ft	\$2.47/ft
Yd^2 = square yard with 1 inch depth, rbmi = roadbed mile, ft = foot			

Table 3.8 Typical material and total treatment costs for the State of Michigan (Baladi & Dean 2011)

Treatment type	Road type	
	Freeway	Non-Freeway
	Material cost only	
HMA reconstruction	\$476,000/lane-mile	\$272,000/lane-mile
PCC reconstruction	\$359,000/lane-mile	\$325,000/lane-mile
Rubblization & HMA overlay	\$238,000/lane-mile	\$263,000/lane-mile
Unbonded PCC overlay	\$263,000/lane-mile	\$262,000/lane-mile
	Total cost including materials, engineering fees, culverts and slope work, etc.	
HMA reconstruction	\$1,000,000/mile	\$1,200,000/mile
PCC reconstruction	\$1,300,000/mile	-

CHAPTER 4

DATA ANALYSES & DISCUSSION

4.1 Introduction and Research Objectives

In this chapter, the analysis methods used in this study and the analysis results are presented and discussed. The analyses and discussions were based on the scope, research plan, and objectives of this study. The scope, objectives, and the flow chart of the research plan are presented in Chapter 1 and for convenience, they are included below. The hypotheses, which were established at the commencement of this study, are presented in Section 4.2 below.

As stated in Chapter 1, section 1.3, the scope of this research study is to determine the optimum set of pavement related data items that could be used to optimize and maximize the benefits of pavement management data collection and to determine the benefits of pavement treatments. The specific objectives of the study are repeated below for convenience.

- Determine the optimum set of data needed to comprehensively and systematically arrive at cost-effective pavement management decisions.
- Determine the impact of the current state-of-the-practice regarding the quality and accuracy of the time-series pavement condition and distress data.
- Review the advantages and shortcomings of existing methodologies and algorithms to express the benefits of pavement treatments, and recommend the most reliable methods.
- Analyze the state-of-the-practice to establish, for some treatment types, treatment transition matrices (T^2M) expressing the probability that the treatment causes gain, loss, or has no effect on the service life of the treated pavement segments. The matrices will be based on both the before treatment (BT) and the after treatment (AT) pavement conditions and distresses and rates of deterioration.

- Provide implementation steps to be used by State Highway Agencies (SHAs) to estimate the effectiveness of pavement treatments based on the BT conditions and distresses and rates of deterioration of planned pavement projects.
- Determine the most cost-effective pavement treatment type, time, and project boundaries based on the pavement conditions and distresses and rates of deterioration.
- Develop methodology to estimate the pavement conditions and distresses of multiple-lane facilities based on the time dependent conditions and distresses of the driving lane.
- Develop universal guidelines that can be implemented by SHAs to reliably determine the most cost-effective:
  a. Pavement treatment strategy at the network level within a given set of constraints (budget and political constraints).
  b. Treatment type, time, and project boundaries at the project level.

To accomplish the above objectives, a comprehensive research plan was drawn and is presented in Chapter 1 and the flow chart of the plan is shown in Figure 4.1.

It should be noted that, the time-series pavement condition and distress data for each 0.1 mile long pavement segment that were obtained from the four SHAs were used in the analyses. Dean et al. 2012 used the same data to simulate longer pavement segment lengths of 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, and 1 mile and analyzed the effects of pavement segment length on data modeling and the calculation of treatment benefits. They concluded that the pavement survey length does not affect the results and accuracy of the data modeled using the proper mathematical functions nor does it affect the calculated treatment benefits.

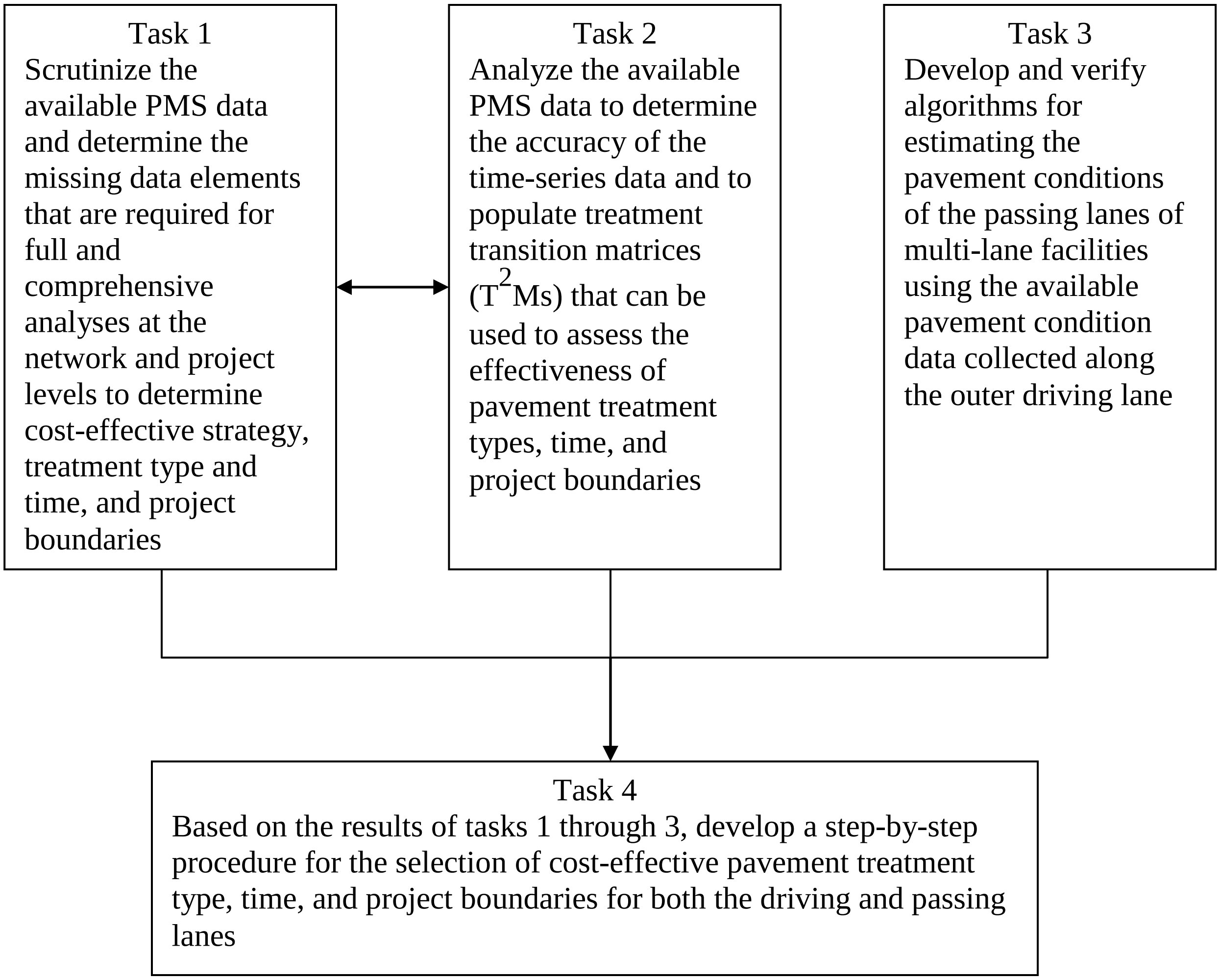


Figure 4.1 Flow chart and the four tasks of the research plan

4.2 Hypotheses

In this study, it is hypothesized that for a given pavement network and budget, there exists an optimum pavement treatment strategy consisting of treatment types, times, and project boundaries that would minimize the objective function stated in Equation 4.1. The conditions of the pavement network and its rates of deterioration before and after treatment significantly affect the agency and user costs and hence, the outcomes of the objective function.

$$\text{Objective Function} = \text{Minimum}\left(\frac{\text{Costs}}{\text{Benefits}}\right) \qquad \text{Equation 4.1}$$

Where, Costs are the agency costs ($) with consideration of the user costs;

Benefits are the treatment benefits in term of years of service (see section 4.5.2)

In this study, it was also hypothesized that for facilities with multiple lanes in one direction, the measured pavement conditions and distresses and the traffic lane distribution factor (LDF) can be used to estimate the pavement conditions and distresses of the other lanes.

4.3 Data Analysis Primer

After restructuring the pavement condition and distress data in Microsoft Excel spreadsheet/format (see Chapter 3), the data were divided into two groups; the Minnesota Road Research (MnROAD) data and the four SHA's data. The former were analyzed to develop rigorous models that can be used to assess the pavement conditions and distresses of multi-lane facilities using the time-series data collected along the driving lane. The methodology used in the analysis and the analysis results are presented and discussed in section 4.6 below. The data from three of the four SHA's were subjected to three tear analyses (data acceptance and modeling, the remaining service life (RSL), and treatment benefits). The analysis methodology used in each tear and the results are presented and discussed in Sections 4.4 and 4.5 below. It should be noted that various difficulties were encountered while restructuring the data from the Michigan Department of Transportation (MDOT) especially in the attempts to make the location reference system compatible from one year to the next (see Chapter 3). Unfortunately, none of the various attempts succeeded on a global scale. The reference location system of each 0.1 mile long pavement segment of each year must be changed manually without any specific rules so that it matches the location reference system of another year. Hence, the MDOT pavement condition and distress data were not subjected to any further analysis.

4.4 Data Acceptance Criteria and Modeling

The time-series pavement condition and distress data were subjected to robust quality control (QC) analysis to determine whether or not the time-series data could be analyzed. The QC analysis consists of the two data acceptance criteria presented in Sections 4.4.1 and 4.4.2. The two criteria were developed to assess whether or not the pavement condition and distress data for each 0.1 mile long pavement segment along the identified pavement projects could be included in the analyses. The criteria independently address the pavement condition and distress data collected BT and AT. Matlab computer programs were written and/or coded for the implementation of the acceptance criteria. The data for each condition and distress type of each 0.1 mile long pavement segment along each identified pavement project were scrutinized by the program to determine whether or not they satisfied the acceptance criteria. Those which did were included in the analyses; otherwise, they were excluded.

It is very important to note that sensor (rut depth and International Roughness Index (IRI)) and image (pavement surface distress) data of all pavement sections that were subjected to treatments were not measured immediately after construction. In almost all cases, the data were collected between few months and few years after construction. For most projects, this time window between the end of construction and data collection did not significantly affect the time-series IRI and surface distress data. However, it had significant impact on the time-series rut depth data. That is the first measured and available AT rut depth data ranges from almost zero inch to a significant rut depth depending on the pavement section and the time after construction at which the data were measured. This created a substantial problem when modeling the rut depth data as a function of time. The problem was partially solved by assuming that at 0.1 year after the completion of each of the four pavement treatments listed below, the rut depth is 0.01-inch (zero-inch cannot be used in power function). The four treatments are:

1. Thin, less than 2.5 inch hot-mix asphalt (HMA) overlay of asphalt surfaced pavement sections.
2. Thick, equal to or more than 2.5 inch HMA overlay of asphalt surfaced pavement sections.
3. Thin, less than 2.5 inch mill and fill of asphalt surfaced pavement sections.
4. Thick, equal to more than 2.5 inch mill and fill of asphalt surfaced pavement sections. Herein the term thin refers to non-structural HMA overlay and mill and fill treatments of less than 2.5 inches, while the term thick refers to treatments of 2.5 inches or more.

The above assumption is logical and reasonable because:

1. The smooth-drum rollers that are typically used in the compaction of the HMA overlays or mill and fill projects produce smooth and flat pavement surface.
2. The assumption is supported by the available databases (for most pavement sections subjected to one of the four treatments listed above, rut depth data measured within 2 months after construction varied from almost zero-inch to insignificant number).

Thus, one data point (0.01-inch at 0.1 year) was added to the AT rut depth data of each 0.1 mile long pavement segment subjected to any of the four treatments listed above. No data point was added to the rut depth data of any 0.1 mile long pavement segment that was subjected to single or double chip seal treatments. Such addition cannot be supported by engineering reasons or the available databases. Nevertheless, the addition of the rut depth data point enhanced the quality and increased the number of available AT rut depth data. This resulted in increases in the percent of the pavement segments accepted for analysis.

4.4.1 The First Acceptance Criterion - Three Data Points

After identifying the pavement sections/projects that were subjected to one treatment type in the past (see Chapter 3), the available time-series data for each 0.1 mile long pavement segment along the project and for each condition and distress type were tested to determine whether or not the data satisfy the first acceptance criterion, that is the data include:

1. Three or more consecutive data points BT.
2. Three or more consecutive data points AT.

This acceptance criterion is based on the fact that, for each pavement condition and distress type, a minimum of three data points are required BT and three data points AT to model the time-series pavement condition and distress data using the non-linear mathematical functions listed in Table 4.1. Two or less data points do not define the parameters of the non-linear mathematical functions representing the data. The implication of this acceptance criterion is that, for those states that collect data every other year, a minimum of 12 years of time-series data (6 years BT and 6 years AT) are required to develop before and after treatment performance models and hence analyze the benefits of the treatment. Whereas for the State of Washington, who collect the data every year, a minimum of 6 years of data are required (3 years BT and 3 years AT). For each 0.1 mile long pavement segment and for each condition and distress type where the data did not satisfy the acceptance criterion, the data were excluded from any further analysis. Thus, a given 0.1 mile long pavement segment could be analyzed if one or more sets of data such as IRI, transverse cracks, and rut depth are accepted and the other distress data (longitudinal and alligator cracks) are not. This implies that a 0.1 mile long pavement segment could be included in the analysis for one condition or distress type and excluded for others or it could be included in the analysis up to five times, once for each of the four distress types and

Table 4.1 Pavement condition and distress models

Pavement condition/ distress type	**International Roughness Index (IRI) (inch/mile or m/km)**	**Rut depth (RD) (inch or mm)**	**Cracking (length, area, or percent)**
Model form	Exponential	Power	Logistic (S-shaped)
Generic equation	$\mathrm{IRI} = \alpha \exp(\beta t)$	$\mathrm{RD} = \gamma\, t^{\omega}$	$\mathrm{Crack} = \dfrac{k}{1 + \exp[-\theta(t - \mu)]}$
Time when a threshold value is reached	$t = \dfrac{\ln\left(\dfrac{\mathrm{Threshold}_{IRI}}{\alpha}\right)}{\beta}$	$t = \exp\left[\dfrac{\ln\left(\dfrac{\mathrm{Threshold}_{RD}}{\gamma}\right)}{\omega}\right]$	$t = -\left[\dfrac{\log\left(\dfrac{k}{\mathrm{Threshold}_{Crack}} - 1\right) - \theta * \mu}{\theta}\right]$
Derivative (rate of deterioration)	$\dfrac{d\,\mathrm{IRI}}{dt} = \alpha\,\beta \exp(\beta t)$	$\dfrac{d\,\mathrm{RD}}{dt} = \gamma\omega t^{(\omega - 1)}$	$\dfrac{d\,\mathrm{Crack}}{dt} = -\dfrac{k * \theta * \exp(\theta(t + \mu))}{(\exp(\theta t) + \exp(\theta\mu))^2}$
Integral (area)	$A_{IRI} = \int_{t_1}^{t_2} = \left(\dfrac{\alpha}{\beta}\right)\exp(\beta t)$	$A_{RD} = \int_{t_1}^{t_2} = \dfrac{\gamma\, t^{(\omega+1)}}{(\omega + 1)}$	$A_{Crack} = \int_{t_1}^{t_2} = k\left[\mu - \dfrac{\log(\exp(\theta(\mu - t)) + 1)}{\theta}\right]$
Where, α, β, γ, ω, k, θ, and μ are regression parameters, RD is rut depth, Crack is alligator, longitudinal, or transverse crack length, area or percent, t is the elapsed time in years, and Threshold is the pre-specified condition or distress level indicating zero serviceability for any given pavement condition or distress type.			

once for IRI. The bottom line is that, for each pavement treatment type (thin and thick HMA overlays, single and double chip seal, and thin and thick mill and fill), the number of 0.1 mile long pavement segments that were included in the analysis varied from one condition or distress type to another.

Tables 4.2 through 4.4 provide a list of the results of each of the data acceptance criterion of the three SHA's data, and for each of the BT and AT pavement condition and distress types. The results include:

- The percent of the 0.1 mile long pavement segments where the AT and BT data, for each pavement condition and distress type, independently passed the first acceptance criterion.
- The percent of the 0.1 mile long pavement segments where the AT and BT data, for each pavement condition and distress type, independently passed the second acceptance criterion (see section 4.2.2).
- The number of the 0.1 mile long pavement segments accepted in the analysis for each pavement condition and distress type.
- The number of the 0.1 mile long pavement segments available in the database for each pavement condition and distress type.
- The overall number of the 0.1 mile long pavement segments accepted for further analysis. It should be noted that, if a pavement segment is accepted in 1 or more pavement condition or distress types it is counted one time only.

The data in Tables 4.2 through 4.4 indicate that the acceptance rate of the first criterion ranges from 37.8% to 100% and the majority have an acceptance rate of 90% or better. This is mainly due to the pavement section identification procedure used in the selection of the

Table 4.2 The BT and AT results of the 2 acceptance criteria, Colorado

TT	BT, AT, & the number of 0.1 mile long pavement segments	The percent of 0.1 mile long pavement segments passing each acceptance criterion for each condition and distress type, and the number of 0.1 mile long pavement segments accepted and available in the database										Number of segments accepted (*) and available
		IRI		RD		AC		LC		TC		
		1	2	1	2	1	2	1	2	1	2	
A	BT (%)	99.6	82.3	99.6	62.3	99.6	78.5	99.6	68.2	85.9	77.5	
	AT (%)	98.4	72.6	99.7	96.2	98.4	69.4	98.4	58.5	98.4	63.7	
	Accepted	557		559		506		384		385		878
	Available	968		968		968		968		968		968
B	BT (%)											
	AT (%)											
	Accepted											
	Available											
C	BT (%)	98.5	60.5	98.5	39.2	98.5	62.9	98.5	67.5	77.7	78.9	
	AT (%)	94.5	77.3	94.5	36.9	94.5	75.2	94.4	76.6	94.4	82.1	
	Accepted	2,281		399		2,228		2,440		2,163		4,033
	Available	4,958		4,958		4,958		4,958		4,958		4,958
D	BT (%)											
	AT (%)											
	Accepted											
	Available											
E	BT (%)	100	73.6	100	81.3	100	84.6	100	61.5	100	60.4	
	AT (%)	73.6	69.2	100	100	73.6	94.5	73.6	92.3	73.6	80.2	
	Accepted	28		74		49		38		24		83
	Available	91		91		91		91		91		91
F	BT (%)											
	AT (%)											
	Accepted											
	Available											

TT = Treatment type; RD = Rut Depth; AC = Alligator Cracks; LC = Longitudinal Cracks; TC = Transverse Cracks. Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

(*) If a pavement segment is accepted in 1 or more pavement condition or distress types it is counted one time only.

Table 4.3 The BT and AT results of the 2 acceptance criteria, Louisiana

TT	BT, AT, & the number of 0.1 mile long pavement segments	The percent of 0.1 mile long pavement segments passing each acceptance criterion for each condition and distress type, and the number of 0.1 mile long pavement segments accepted and available in the database										Number of segments accepted (*) and available
		IRI		RD		AC		LC		TC		
		1	2	1	2	1	2	1	2	1	2	
A	BT (%)	88.8	63.7	95.4	47.7	95.4	61.8	99.8	16.2	99.8	31.6	
	AT (%)	94.5	85.7	97.9	100	99.6	71.9	99.6	74.7	99.6	84.4	
	Accepted	219		224		202		71		134		439
	Available	526		526		526		526		526		526
B	BT (%)	97.5	69.2	97.5	52.4	97.8	64.0	98.7	31.1	98.7	47.3	
	AT (%)	94.5	87.8	99.4	100	99.8	71.5	99.8	72.2	99.8	79.5	
	Accepted	1,416		1,242		1,199		595		984		2,279
	Available	2,511		2,511		2,511		2,511		2,511		2,511
C	BT (%)	95.5	61.6	96.1	67.7	96.1	91.4	96.7	48.8	96.7	49.1	
	AT (%)	94.1	78.7	81.2	63.6	99.8	74.4	99.8	71.8	99.8	79.9	
	Accepted	1,089		574		1,605		772		819		2,131
	Available	2,421		2,421		2,421		2,421		2,421		2,421
D	BT (%)	98.8	60.5	67.9	75.8	99.0	87.2	99.5	35.3	99.8	24.2	
	AT (%)	97.8	88.9	71.9	54.3	99.8	51.6	99.8	58.5	99.8	65.7	
	Accepted	206		43		177		61		44		316
	Available	405		405		405		405		405		405
E	BT (%)	92.1	64.1	95.3	70.7	97.0	75.1	97.5	25.5	97.5	40.6	
	AT (%)	90.1	87.1	84.1	100	97.0	58.9	97.0	81.9	97.0	88.2	
	Accepted	163		191		146		80		135		311
	Available	365		365		365		365		365		365
F	BT (%)	96.6	74.4	91.4	77.8	92.0	80.3	99.4	33.3	99.4	40.5	
	AT (%)	92.7	82.8	99.3	100	99.6	56.4	99.6	69.8	99.6	74.5	
	Accepted	735		957		605		286		396		1,280
	Available	1,390		1,390		1,390		1,390		1,390		1,390

TT = Treatment type; RD = Rut Depth; AC = Alligator Cracks; LC = Longitudinal Cracks; TC = Transverse Cracks. Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

(*) If a pavement segment is accepted in 1 or more pavement condition or distress types it is counted one time only.

Table 4.4 The BT and AT results of the 2 acceptance criteria, Washington

TT	BT, AT, & the number of 0.1 mile long pavement segments	The percent of 0.1 mile long pavement segments passing each acceptance criterion for each condition and distress type, and the number of 0.1 mile long pavement segments accepted and available in the database										Number of segments accepted (*) and available
		IRI		RD		AC		LC		TC		
		1	2	1	2	1	2	1	2	1	2	
A	BT (%)	50.6	74.3	50.6	83.4	100	85.1	100	55.5	100	74.3	
	AT (%)	99.1	74.4	100	99.7	100	98.0	100	87.2	100	97.7	
	Accepted	349		709		1,746		1,000		1,538		2,059
	Available	2,100		2,100		2,100		2,100		2,100		2,100
B	BT (%)	37.8	65.6	37.9	88.4	100	87.3	100	79.6	100	49.5	
	AT (%)	100	80.2	100	99.8	100	99.1	100	84.5	100	96.6	
	Accepted	10		122		403		310		220		461
	Available	465		465		465		465		465		465
C	BT (%)	100	82.4	100	64.2	100	92.7	100	73.5	100	99.5	
	AT (%)	100	33.3	100	28.4	100	83.3	100	73.5	100	95.1	
	Accepted	52		38		156		111		194		203
	Available	204		204		204		204		204		204
D	BT (%)											
	AT (%)											
	Accepted											
	Available											
E	BT (%)	88.3	42.4	88.3	81.8	96.1	94.3	96.1	43.3	96.1	67.9	
	AT (%)	83.3	83.7	100	99.1	100	96.7	100	88.2	100	97.0	
	Accepted	123		701		886		357		633		946
	Available	1,013		1,013		1,013		1,013		1,013		1,013
F	BT (%)											
	AT (%)											
	Accepted											
	Available											

TT = Treatment type; RD = Rut Depth; AC = Alligator Cracks; LC = Longitudinal Cracks; TC = Transverse Cracks. Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

(*) If a pavement segment is accepted in 1 or more pavement condition or distress types it is counted one time only.

appropriate time window of calendar years when the treatments took place (see Chapter 3). This time window must be between a minimum of three data collection cycles BT and three AT.

Although the application of the first acceptance criterion presented and discussed above is necessary to properly model the BT and AT pavement conditions and distresses; its implementation resulted, for some condition and distress types along some pavement projects, in a significant reduction in the number of 0.1 mile long pavement segments having sufficient data for analyses. Although few methods exist to impute missing data points; no data were imputed in this study (Baladi et al. 2011).

4.4.2 The Second Acceptance Criterion – Rate of Deterioration

For some 0.1 mile long pavement segments, the BT and/or the AT time-series data indicate that the pavement condition is improving over time without the application of any treatment. In the absence of a pavement treatment, most pavement sections deteriorate over time. Having said that, pavement healing without treatment could be the results of:

- The applied treatments were not recorded in the pavement management system (PMS) database. In some cases the records do not exist, in others, the records are not included in the PMS database but could be requested and obtained from the pavement maintenance or rehabilitation divisions. In this last scenario, care must be taken to match the location reference system used in the maintenance file to that used in the PMS pavement condition and distress data files.
- Inaccuracy in the linear reference location system (such as beginning and ending mile points) used by most SHAs. Such inaccuracy was calculated in this study using the Global Positioning System (GPS) data reported parallel to the linear location referencing system by

the SHAs. It was found that the error ranges from about zero to more than 500 feet per 0.1 mile long pavement segment. For more detail please see section 4.5.10.

- Human error and/or inaccuracy while digitizing the electronic images. The subjectivity in digitizing the images and the rating of the severity levels or in estimating the extent of the distress could generate inaccuracies in the time-series data. On the other hand, the sensor measured data do not exhibit this problem as no human subjectivity is involved.
- Data inaccuracy due to equipment problems such as calibration, malfunction, or changing equipment type.
- Environmental conditions from one data collection cycle to the next. For example, the crack opening is wider on cold days than on warmer ones due to thermal expansion and contraction. This may change the assigned crack severity level and the observance of the length of the cracks. Likewise, temperature differential may cause curling on certain days which influence the measured pavement roughness in terms of IRI.

Nevertheless, the implementation of this acceptance criterion was accomplished using the following three steps:

Step 1 – Data Modeling – The time-series pavement condition and distress data were modeled using the proper mathematical function and least squares regression. The latter is commonly used to determine the statistical parameters of the selected mathematical functions. The least squares regression is based on minimizing the sum of the squared differences (error) between the calculated and the measured data. The selected mathematical functions, such as those listed in Table 4.1 and shown in Figure 4.2, are based on known trends in, and mechanism of, pavement deterioration (M-E PDG 2004, Meyer et al. 1999). For example, rutting typically occurs early in the service life of asphalt pavements and its progression slows over time as the pavement materials become denser due to loading. Hence, a power function would model the data. However, pavement roughness typically increases exponentially as the pavement ages, cracks, and becomes uneven thereby causing increases in the dynamic traffic load. Hence, an exponential function is typically used to model the IRI data. Finally, the propagation and deterioration of pavement surface cracks typically follows three stages. In the first stage, few cracks appear in the early pavement service life; their number and length increase following a slowly rising exponential function (see Figure 4.3). In the second stage, the crack number and their lengths increase almost linearly over time as they approach the crack saturation point. In the third stage (crack saturation stage), few, if any, new cracks are initiated and most existing cracks approach their maximum possible lengths (lane width or the pavement segment length). During this third stage period, a power function could be used to model the cracks as a function of time. Given the above description of crack initiation and propagation, a logistic function could be used to model the three stages as a function of time. However, the accuracy of such modeling is dependent on the availability of cracking data in all three stages and the definition of the crack saturation point. For transverse cracks, the crack saturation point could be defined by the crack spacing along the pavement segment (such as 2 to 4 feet). Likewise, longitudinal crack saturation point could be defined by the number of cracks across the lane (such as 2, 3, or 4 longitudinal cracks) and their maximum lengths (such as the length of the pavement segment). Finally, alligator crack saturation point could be considered as the length of one or both wheel paths. The exact definition of crack saturation point should be established by each SHA based on the agency's state-of-the-practice, pavement conditions and distresses, and the agency's experience.

Once again, logistic (S-shaped curve) is typically used to model cracking data as a function of time when sufficient cracking data are available to statistically determine the

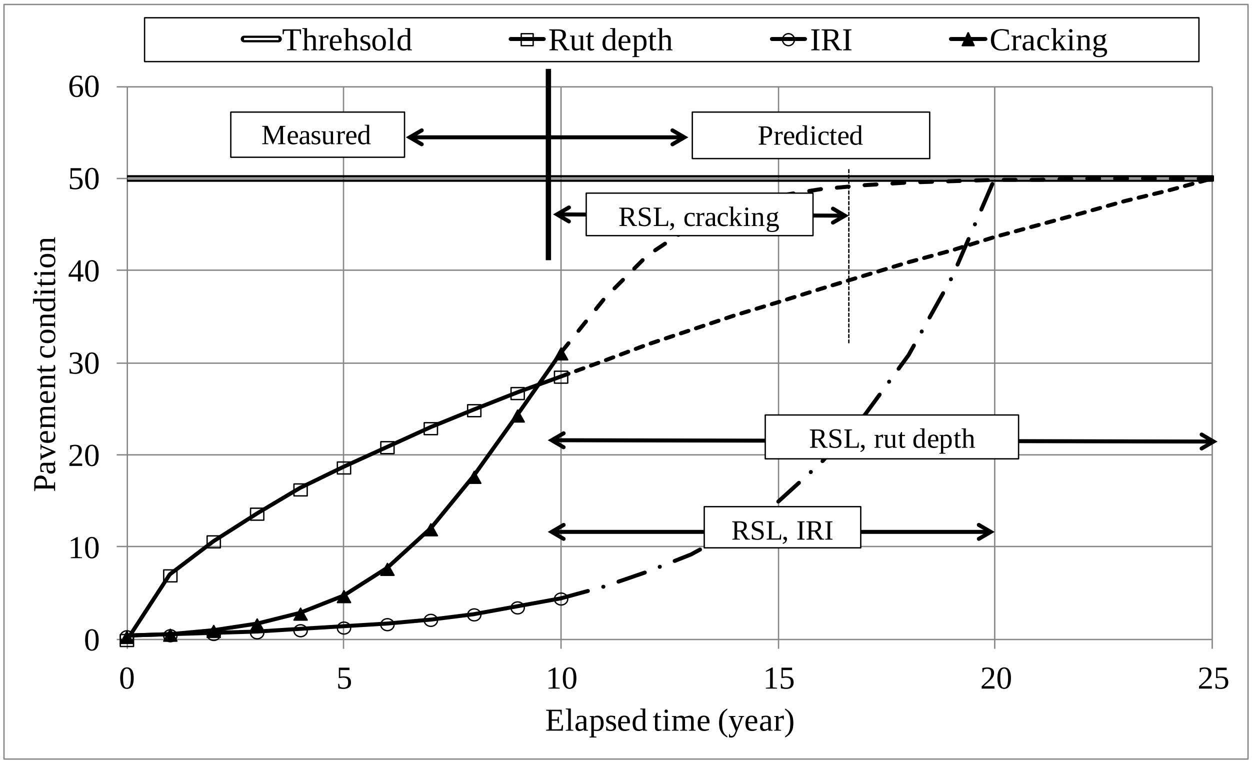


Figure 4.2 Exponential, power, and logistic (S-shaped) curves

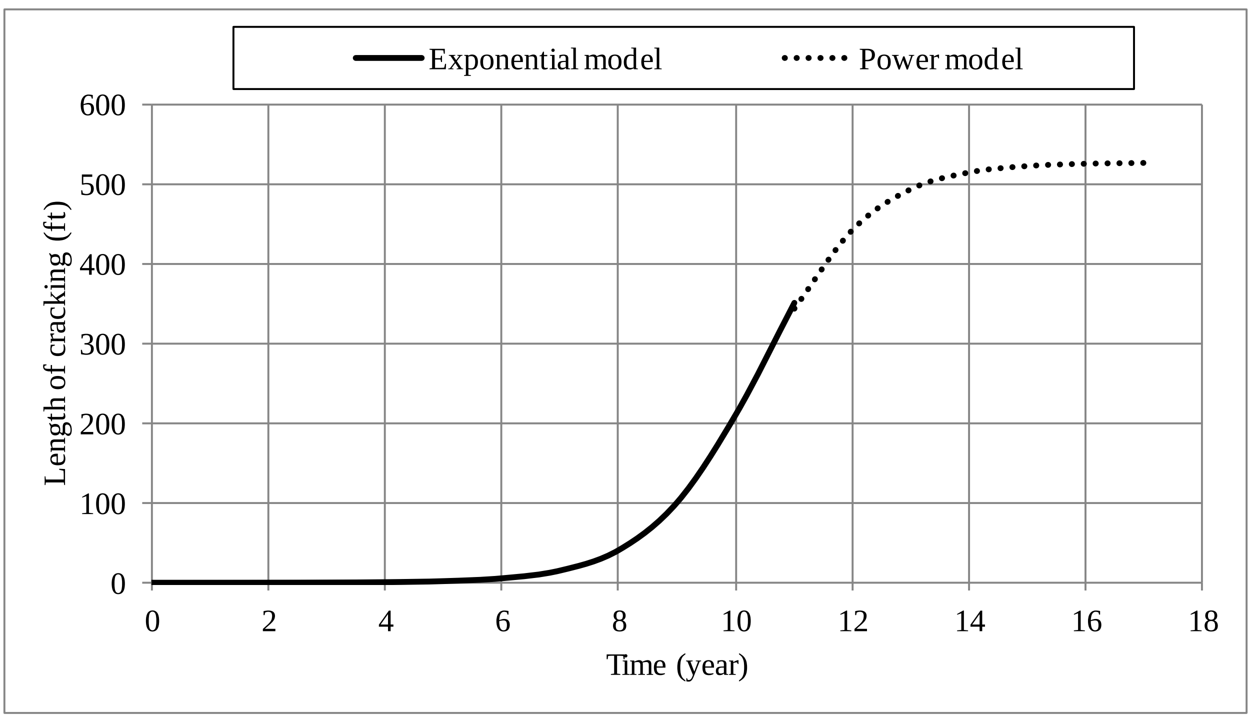


Figure 4.3 Logistic (S-shaped) function represented by an exponential and a power model

parameters of the s-shaped curve. In this study, most of the available cracking data belong to the first stage of cracking (6 to 8 years of AT data) and hence they were modeled using an exponential function. Note that any mathematical function that models the progression of pavement condition or distress could also be used in the analyses.

The time values used to model the pavement conditions and distresses have significant impact on the regression parameters. The pavement conditions and distresses should be modeled with respect to the pavement segment surface age and not the given data collection calendar years. The pavement surface age is defined as the number of years between the data collection year in question and the last treatment/action year. Therefore, the calendar year of each data collection cycle was used to calculate the pavement surface age at the time of data collection. For AT scenarios and for most 0.1 mile long pavement segments, their pavement surface age was easily calculated using Equation 4.2. However, if the pavement treatment was applied during the same calendar year and before the data were collected, the surface age of the first data point would be zero and could not be modeled using the functions listed in Table 4.1. Hence, in this scenario, the surface age was calculated using Equation 4.3.

$$SA_{AT} = (\text{data collection year} - \text{treatment year}) \qquad \text{Equation 4.2}$$

Where, SA_{AT} is the pavement surface age after treatment in years;

Data collection year is the calendar year of data collection such as 2001 and 2003;

Treatment year is the treatment calendar year such as 1997, 1998, and so on

$$SA_{AT} = 1 + (\text{data collection year} - \text{treatment year}) \qquad \text{Equation 4.3}$$

The BT pavement surface age calculation, on the other hand, was not as straight forward as the AT one. The reason is that, for good number of pavement sections, the pavement treatment files do not include the time at which the previous treatment or two consecutive treatments were done. Hence, two separate scenarios were used as detailed below.

1. The pavement treatment files contain the time for two or more consecutive treatments. In this case, equation 4.4 could be and was used to calculate the pavement surface age.

$$SA_{BT} = 1 + (\text{data collection year - the past treatment year}) \qquad \text{Equation 4.4}$$

Where, SA_{BT} is the BT pavement surface age in years;

Data collection year is the calendar year of data collection such as 2001 and 2003;

The past treatment year is the treatment calendar year such as 1997, 1998, and so on

2. The pavement treatment files contain the time and type of only the last treatment; that is no past treatment data are available (which is the scenario for most pavement sections included in this study). In these cases, the pavement surface age of all 0.1 mile long pavement segments along a given pavement project could be, and was, calculated using Equation 4.5.

$$SA_{BT} = 1 + (\text{data collection year} - \text{first available data year}) \qquad \text{Equation 4.5}$$

Where, first available data year is the calendar year of the oldest data point and all others are the same as before

Step 2 – Examination of the Pavement Rate of Deterioration – As stated in Section 4.4.2, the regression parameters of the mathematical function obtained in step 1 were examined to determine if the modeled pavement conditions and distresses were increasing or decreasing over time. For the exponential and power functions listed in Table 4.1, negative beta or omega parameters, respectively, indicate that the pavement conditions are improving over time without the application of treatment. Such scenario yields infinite RSL and treatment benefits and cannot be used to study pavement performance. The negative parameters could be the results of the various reasons listed in section 4.4.2.

It is very important to note that some SHAs calculate condition indices (such as distress index or pavement condition index) using certain numerical scale where the high end of the scale implies good pavement conditions. Thus decreasing distress index values implies pavement deterioration. In this scenario, negative regression parameters should be accepted.

Step 3 – Combination Thereof –The data for each 0.1 mile long pavement segment which resulted in positive regression parameters, and thereby satisfied the second acceptance criterion, were accepted for further analyses. For each 0.1 mile long pavement segment and for each condition and distress type where the data did not satisfy the acceptance criterion, the data were excluded from any further analysis. That is, when the statistically determined parameters of the mathematical function of any given pavement segment are negative, the BT and AT data of the segment in question were excluded from further analysis.

Tables 4.2 through 4.4 provide a list of the results of each of the acceptance criterion of the three SHA's data, and for each of the BT and AT pavement condition and distress types. The data in the tables indicate that, for the 0.1 mile long pavement segments, the acceptance rate of the second criterion ranges from 16.2% to 100% and the majority have an acceptance rate of 70% or better. It is interesting to note that the AT acceptance rate is generally higher than the BT rate. This is likely due to the age and condition of the pavement segments. The older pavement segments typically have higher and more variable levels of distress, while the AT data are generally more consistent and less variable. In addition, the BT surface age estimation is much less reliable than the AT one, yielding more uncertainty in the BT data.

4.5 Data Analyses & Discussion

It is important herein to make the distinction between the terms "treatment effectiveness" and "cost-effective treatment". The former could be expressed in terms of the treatment benefits (discussed below in subsection 4.5.2) such as the treatment life (TL), service life extension (SLE), or AT RSL which indicates the longevity of the treated pavement segments. The latter "cost-effective treatment" is typically expressed in terms of dollar per year per lane mile. Thus cost-effective treatment implies the cost of the treatment divided by the number of lane-mile and the treatment benefit, which is also called the cost to benefit ratio. Although the two terms; treatment effectiveness and cost-effective treatment are related, they have different meaning. For example, pavement treatment A which covers 2 lane-miles, costs $400,000, and results in a SLE of 10 years, is considered more effective than treatment B which covers 2 lane-miles, costs $300,000, and results in 8 years of SLE. However, the latter is more cost-effective because its cost-effectiveness is $300,000 divided by 2 lane-mile and by 8 years SLE or $18,750 per lane-mile per year versus treatment A with $20,000 per lane-mile per year. In this study, the first hypothesis in section 4.2 states that the pavement treatment cost-effectiveness is the key to selecting pavement treatment type, time, and project boundaries. Unfortunately, very limited cost data are available in the PMS databases. Therefore, in this study, treatment effectiveness is calculated and discussed. Given this, the results of the analysis discussed in the following subsections should be viewed carefully and should not be used to cast judgments on the SHAs and their cost-effectiveness.

Analyses of the time-series pavement condition and distress data are presented and discussed in the next few subsections. First the pavement condition and distress thresholds and their uses for estimating pavement treatment benefits are discussed. Then the formulation and population of treatment transition matrices (T^2Ms) are presented and discussed. The results presented in the T^2Ms were then used to compare the states-of-the-practices of the three SHAs. Further, alternative T^2Ms based on pavement condition and distress data without using any modeling or prediction techniques are presented and their advantages and shortcomings are discussed. Finally, methodologies are provided to better populate the T^2Ms.

4.5.1 Threshold Values

The pavement condition and distress threshold values (defined in section 2.2.2 of Chapter 2) consist of certain levels of pavement conditions or distress signifying the need for pavement treatment. Pavement sections having worse condition or distress levels than the zero RSL threshold value are considered to provide sub-standard levels of service. A SHA could specify various threshold values to trigger preventive maintenance, rehabilitation, or re-construction actions. For each condition and distress type, multiple threshold values could also be specified to designate good, fair, and poor pavement conditions as shown in Figures 4.4 and 4.5 for increasing and decreasing distress index (worsening conditions in both), respectively. Hence, the concept of threshold values could be used at:

1. The project level to trigger the need for treatment actions.
2. The project and network levels to express descriptive terminologies regarding the health or the conditions of the pavement network such as good, fair, poor, etc.
3. The user level to describe the ride quality.
4. The safety level to express the safety of the pavement as in skid resistance.
5. The structural level to express the structural capacity of the pavement using nondestructive deflection test data.

The threshold values to be established by a SHA should be based on the conditions of the pavement network, treatment costs, the experience of its staff, and the policy, objectives, and scope of the agency, in general, and the PMS.

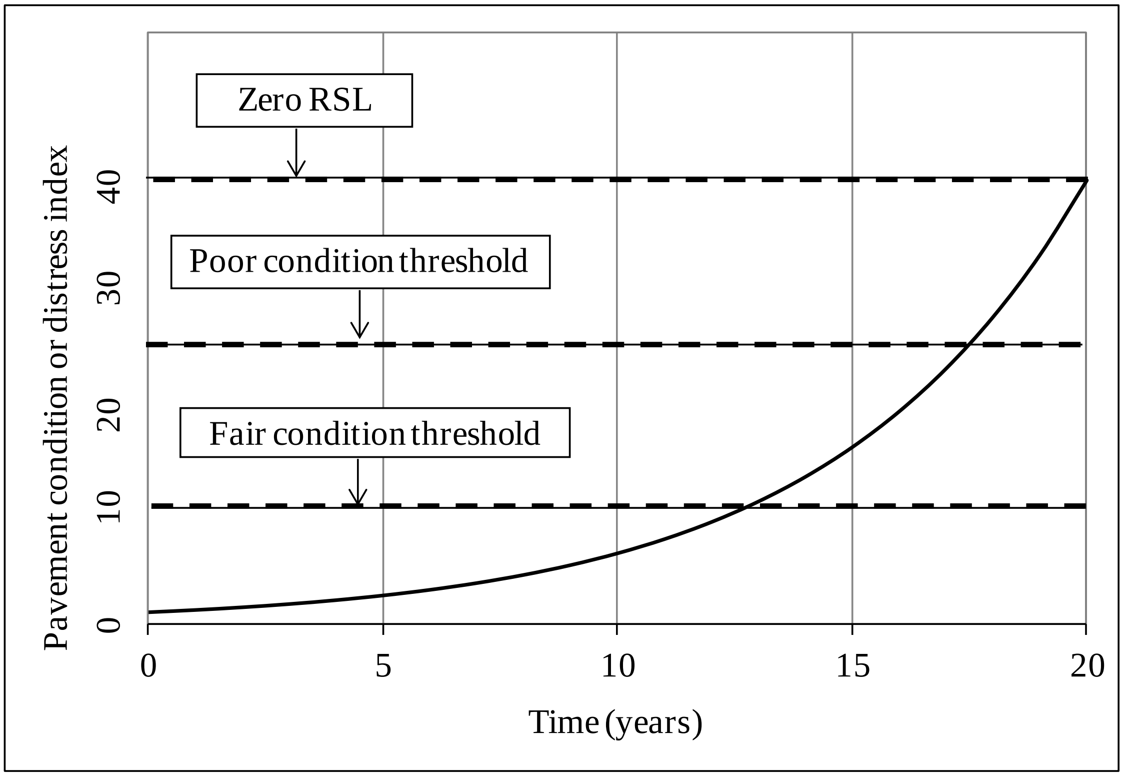


Figure 4.4 Pavement condition thresholds (increasing function)

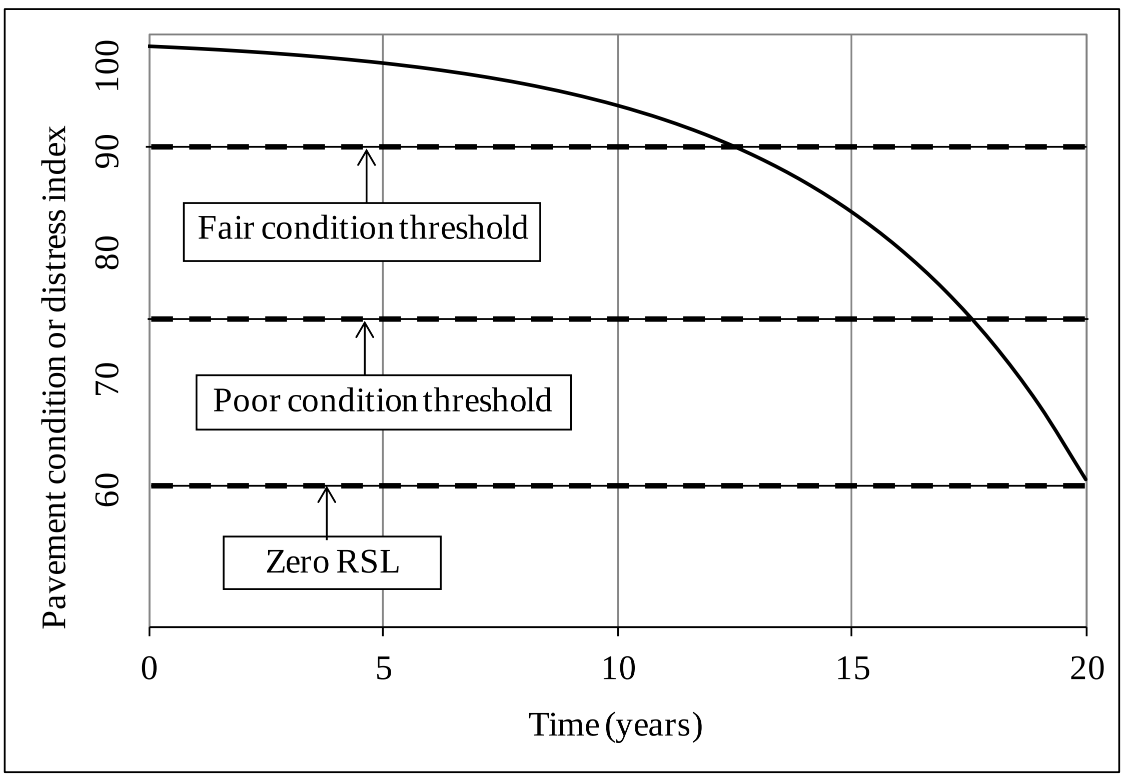


Figure 4.5 Pavement condition thresholds (decreasing function)

As stated earlier, the pavement condition threshold values shown in Figures 4.4 and 4.5 could be used to identify certain descriptive range of the pavement health. For example, a SHA may label a pavement segment in good condition if the IRI value is less than 100 in/mi, fair from 100 to150 in/mi, and poor for values larger than 150 in/mi. Another or the same SHA may specify a maximum IRI threshold value of 200 in/mi when a pavement segment is no longer providing the minimum level of service. The descriptive terms are typically used to express the ride quality in terms more easily understood by the users and they express the current conditions of a pavement segment or the pavement network; they do not indicate the rate of deterioration of the pavement segment.

To overcome the limitations of the descriptive pavement terms or a snapshot value of the pavement condition or distress, the RSL was used in this study to describe the pavement condition and distress and its time rates of deterioration. For each pavement condition and distress type, the RSL was calculated utilizing the proper mathematical model listed in Table 4.1 to estimate the time remaining for the pavement conditions or distresses to reach the specified threshold value. For both BT and AT conditions, the threshold values used in this study are summarized in Table 4.5 (M-E PDG 2004). For each pavement condition or distress type, the threshold values were used to calculate the BT and AT RSL values and hence the treatment benefits as discussed in section 4.5.2. The pavement condition and distress threshold values should be determined based on the various issues discussed below.

1. The Minimum Acceptable Level of Service – The threshold values that constitute zero RSL of a pavement segment should be based on the condition and distress levels when the pavement segment starts to provide the minimum acceptable level of safety, serviceability, or ride quality.

Table 4.5 Pavement condition and distress threshold values used in this study

Pavement condition and distress types	**BT and AT threshold values constituting zero RSL value**
IRI	200 (in/mi)
Rut depth	0.5 (in)
Alligator (fatigue) cracking	10 percent of each wheel path or 105.6 ft per 0.1 mile
Longitudinal cracking	7,000 ft per mile
Transverse cracking	7,000 ft per mile

a. Safety based threshold - The pavement should provide a reasonably safe ride to the users. Although traffic accidents cannot always be prevented or eliminated, their number could be reduced through the design and implementation of balanced pavement maintenance and preservation programs. For example, the maximum acceptable rut depth should be based on the hydroplaning potential when the rut channels are full with water. The hydroplaning potential is a function of the water depth, the vehicle speed, and the tire wear and tire treads. Thus, the maximum acceptable rut depth should be limited to less than the depth of water which could lead to hydroplaning and loss of control of the vehicle. A maximum rut depth of 0.5 inches could be set for roads with speed limits of 55 mph or more, while 0.75 inches rut depth could be specified for local roads having lower speed limits (WSDOT 1999). Lower speeds would allow more time for the tires to dissipate water pressure and hence, remain in contact with the pavement surface. This implies that the rut depth threshold is a function of the pavement class. In this study, one rut depth threshold value of 0.5-inch was used to analyze the RSL of each pavement segment based on the measured rut depth data (M-E PDG 2004).

b. Serviceability or ride quality based threshold - The minimum acceptable level of ride quality or serviceability, which is typically expressed by the IRI, should constitute the threshold value for zero RSL. Such minimum level is a function of the posted speed limit. Higher speed requires smoother road surfaces. Smooth road surfaces and good ride quality are not new concepts. Indeed, they are an integral part of the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures. The AASHTO guide recommends that the minimum acceptable levels of the Pavement Serviceability Index (PSI) are 2.5 for Interstates and major arterials and 2.0 for local roads (AASHTO 1993). In this study, one IRI threshold value of 200 inch per mile was used to analyze the RSL of each pavement segment based on the measured IRI data (M-E PDG 2004).

2. The Lowest Level of Structural Integrity and/or Capacity – The threshold values for cracking should be based on the structural capacity and integrity of the pavement segment in question. Cracking also affects the ride quality to different degrees depending on the shape, geometry, and crack locations. For example, if the two sides of a crack are exactly at the same elevation, the ride quality is only slightly affected depending on the width of the crack. For the majority of the cases, the area of the pavement near the cracks is:
   a. Depressed relative to the rest of the pavement surface as is the case for most temperature cracks in asphalt pavements.
   b. Elevated relative to the rest of the pavement surface as is the case for most reflective transverse cracks in composite pavements.
   c. Faulted, the two sides of the crack are at different elevation as is the case in most transverse cracks and joints in rigid pavements where the load transfer efficiency is low.

Further, high severity and wide open cracks significantly affect the ride quality. The effect of cracks on the ride quality is also a function of the location of the cracks (in the wheel paths, longitudinal, or transverse) and their severity levels and extent. For example, a longitudinal crack at the center of the lane or a transverse crack between the wheel paths do not influence the ride quality. Given the above scenarios, establishing threshold values for cracking based on their effects on the ride quality is not an easy task. Therefore, the cracking threshold values are mainly based on the structural capacity and integrity of the pavement structure. If the cracks affect the ride quality, the IRI threshold would control the decision.

Establishing threshold values for cracking based upon the pavement structural capacity is also a difficult task. The reason is that the threshold value is a function of the cost of the treatment, the treatment benefits, and the severity and extent of the distress. Typical cracking data consists of three severity levels, low, medium, and high. The problem of such data is that the severity level and hence the crack rating is subjective and a function of the judgment of the surveyor who is reviewing the electronic images. His/her judgment is a function of the degree of their training and experience and the pavement temperature at the time when the electronic image was obtained. In addition, the same pavement segment may not be reviewed by the same surveyor each year or data collection cycle. Thus, a crack may be labeled high severity in one year and could be labeled medium or low severity next year and vice versa. Therefore, in this study, the sum of all crack severity levels was used in the analysis (for more information, please reference the discussion in section 3.2 of Chapter 3).

4.5.2 Pavement Treatment Benefits

For each 0.1 mile long pavement segment, five pavement condition and distress data (IRI, rut depth, and alligator, transverse, and longitudinal cracks) were tested against the two acceptance criteria and then modeled as a function of time. Each of the resulting models was then used to calculate the pavement treatment benefits. Extensive literature review was conducted regarding methodologies for estimating pavement treatment benefits. Unfortunately no universally accepted method was found. In summary, the three potential methods were found (RSL, service life extension (SLE), and total benefit (TB)). Because of the shortcomings (see subsection C below) the TB method was modified to modified total benefit (MTB). In addition, one new method was developed, the treatment life (TL). The five methods are presented and discussed below.

A) After Treatment RSL

For a given 0.1 mile long pavement segment, the AT RSL value (see Figure 4.6) was calculated using the available AT time-series pavement condition and distress data, the resulting best fit mathematical model, and the pre-specified threshold values listed in Table 4.5. The AT RSL does not directly account for the BT RSL or condition. Hence, the method does not address the net gain in the pavement service life due to the treatment; it simply estimates the AT longevity of the pavement segment in question.

For some 0.1 mile long pavement segments, the estimated RSL (longevity) was unreasonably large due mainly to two reasons:

1. The high variability of the time-series pavement condition and distress data, which produced high uncertainty in the performance model and hence in the estimated RSL.
2. The available time-series pavement condition or distress data were measured during the early deterioration stage. They do not represent the later pavement rate of deterioration accurately. To illustrate, the pavement condition data of a pavement segment that were collected over seven year period are shown by the open triangles in Figure 4.7. Using the seven data points and the best fit exponential function, the RSL was estimated at 7 years. If the first three data points (indicated by the solid triangles) of the same pavement segment were used to obtain the best fit

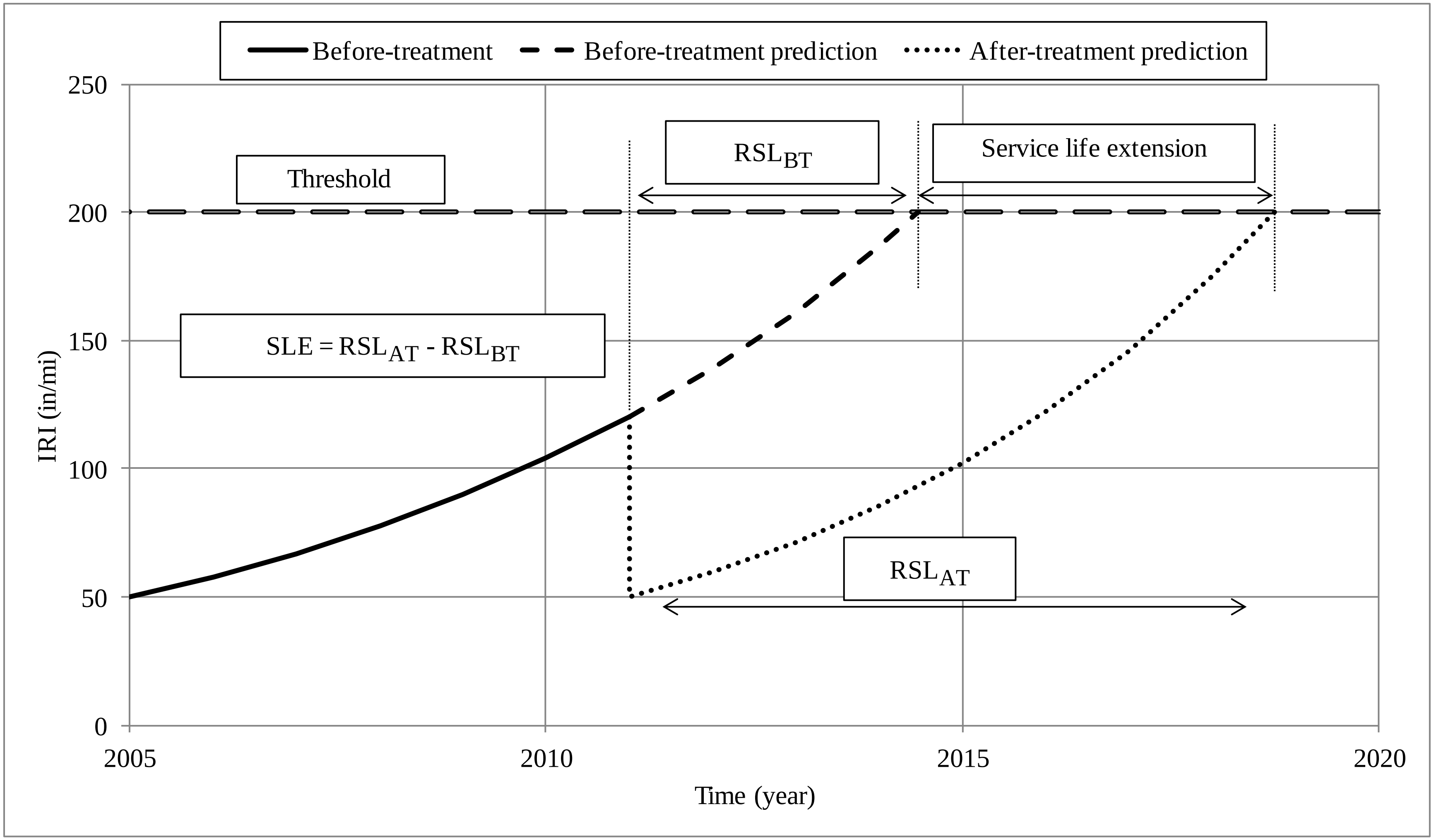


Figure 4.6 Schematic of the definition of AT RSL and SLE

exponential function, then the estimated RSL would be 32 years. Another illustrative example is shown in Figure 4.8 where the early three data points yield very short estimate of the RSL of 4 years relative to the estimated RSL of 12 years using the seven data points. Hence, the estimation of the RSL at early ages where the rate of deterioration is not well defined yet may cause over, under, or the correct estimation of the RSL value. While there is no solid engineering scenario that can be used to increase the under estimated AT RSL values, the overestimated values were reasonably limited to the assumed maximum treatment design service life of 20 years.

Nevertheless, in this study, this method or the longevity of the treated pavement segment was used along with two other methods to assess the effectiveness of the treatment.

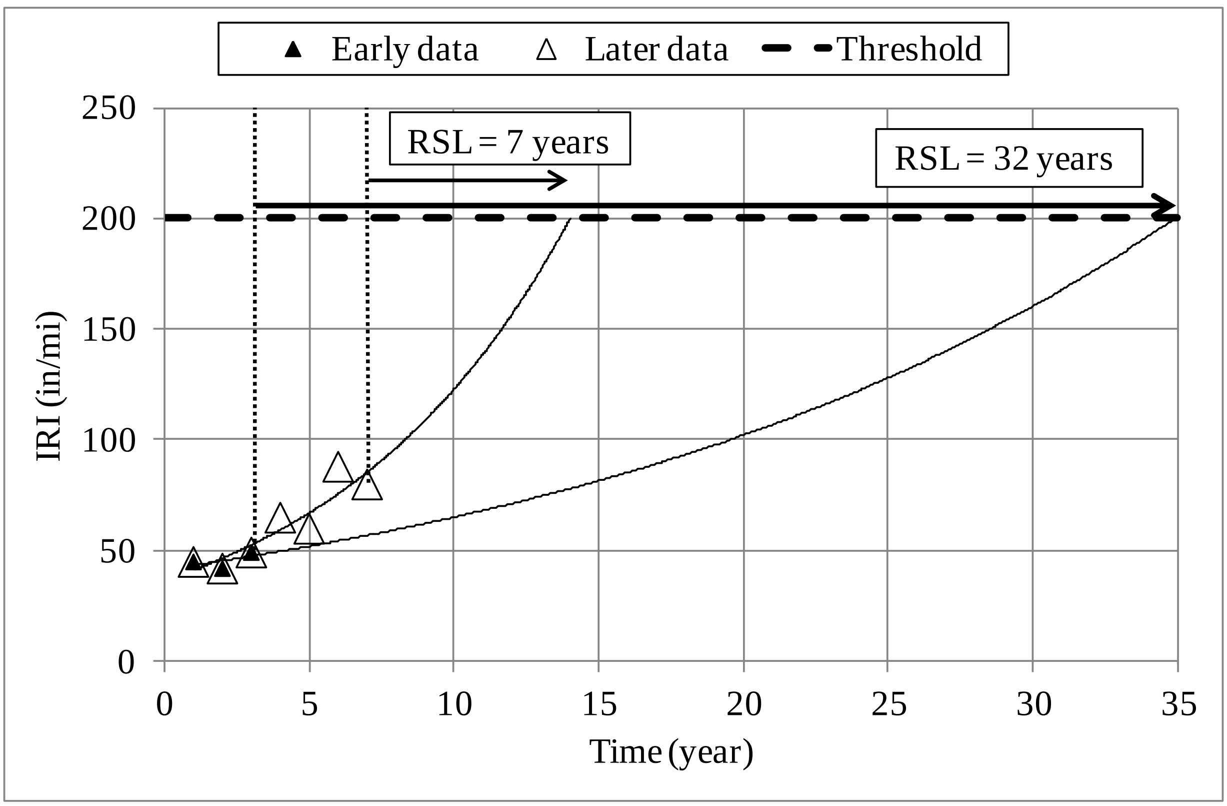


Figure 4.7 Overestimating RSL using three early measured IRI data points

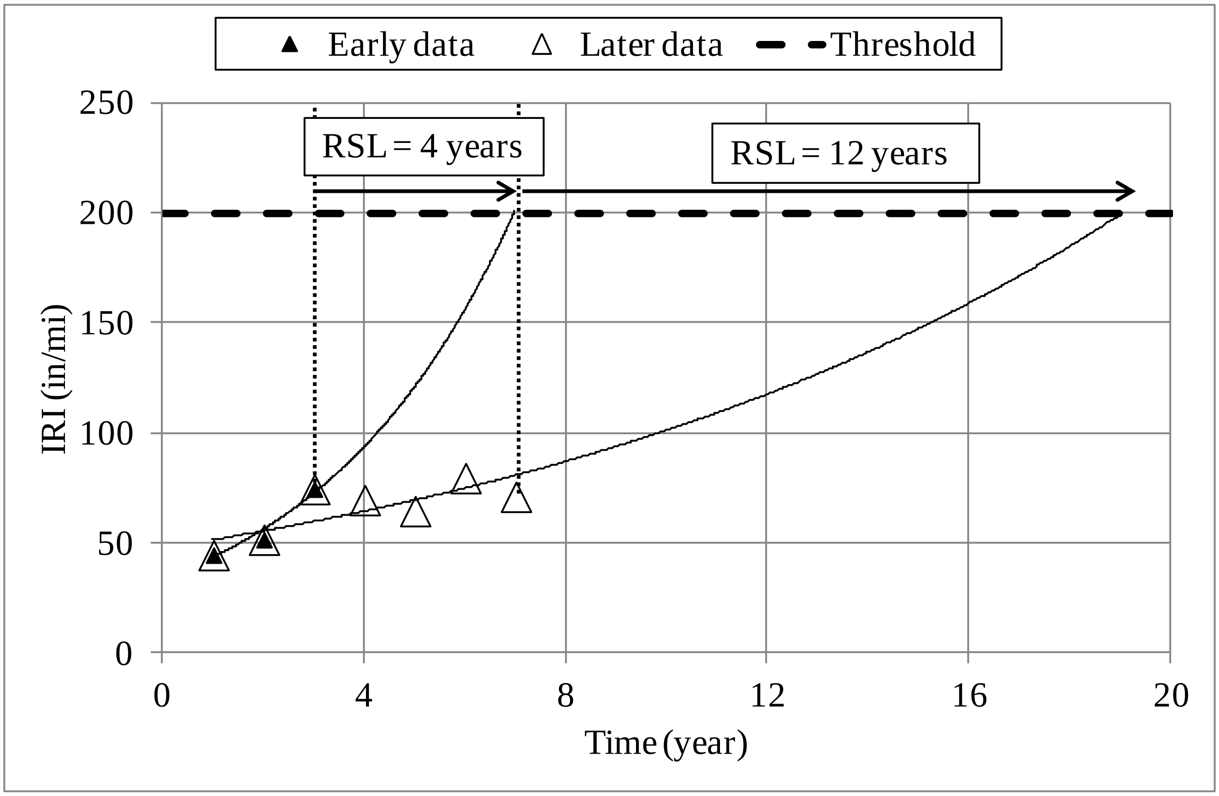


Figure 4.8 Underestimating RSL using three early measured IRI data points

B) Service Life Extension (SLE)

The SLE is the gain in the pavement service life due to the treatment. For each 0.1 mile long pavement segment, the SLE was calculated as the difference in years between the estimated BT and AT RSL values as shown in Figure 4.6 above. It should be noted that since in most cases only three BT data points were available, the estimated BT RSL values are susceptible to the same shortcomings shown in Figures 4.7 and 4.8. Therefore, the BT RSL values were limited to the assumed maximum BT design service life (DSL) of 25 years. In this study, the SLE of the treated pavement segment was also used to assess the effectiveness of the pavement treatment.

C) Total Benefit (TB)

The TB shown in Figure 4.9 is calculated as the ratio of the area bounded by the BT and AT pavement performance curves and the threshold value (the benefit area) to the area bounded by the BT pavement performance curve and the threshold value (the do-nothing area) (Peshkin et al. 2004). In this study, the pre-specified threshold values listed in Table 4.5 were used and the TB values were calculated for several 0.1 mile long pavement segments. It was found that:

1. The TB is susceptible to similar shortcomings as those shown in Figures 4.7 and 4.8 and discussed in part A above. The estimated benefit and do-nothing areas could be overestimated, underestimated, or correctly estimated.
2. The do-nothing area is nearly always underestimated due to the lack of available BT data; for most cases, only three BT data points are available.
3. The calculated TB for various 0.1 mile long pavement segments could be almost the same and yet the available pavement condition and distress data are drastically different.
4. For a significant number of 0.1 mile long pavement segments, the pavement surface age at the time of the treatment is not known and hence, the do-nothing area cannot be calculated.

Because of the above listed reasons, the TB method as reported by Peshkin was not used in this study. Further, the method was modified to overcome some of its shortcomings. The modified method is presented below.

D) Modified Total Benefit (MTB)

The MTB is similar to the TB method except that the BT and AT threshold values for each 0.1 mile long pavement segment were taken the same as the pavement condition or distress at the time of the treatment as shown in Figure 4.10. This modification decreased the time period over which the data are extrapolated to calculate the do-nothing and the benefit areas. The MTB values of several 0.1 mile long pavement segments were calculated using Equation 4.6 and the results were scrutinized. It was found that the shortcomings of the MTB method are more or less similar to those of the TB method. Therefore, the MTB was not used in this study.

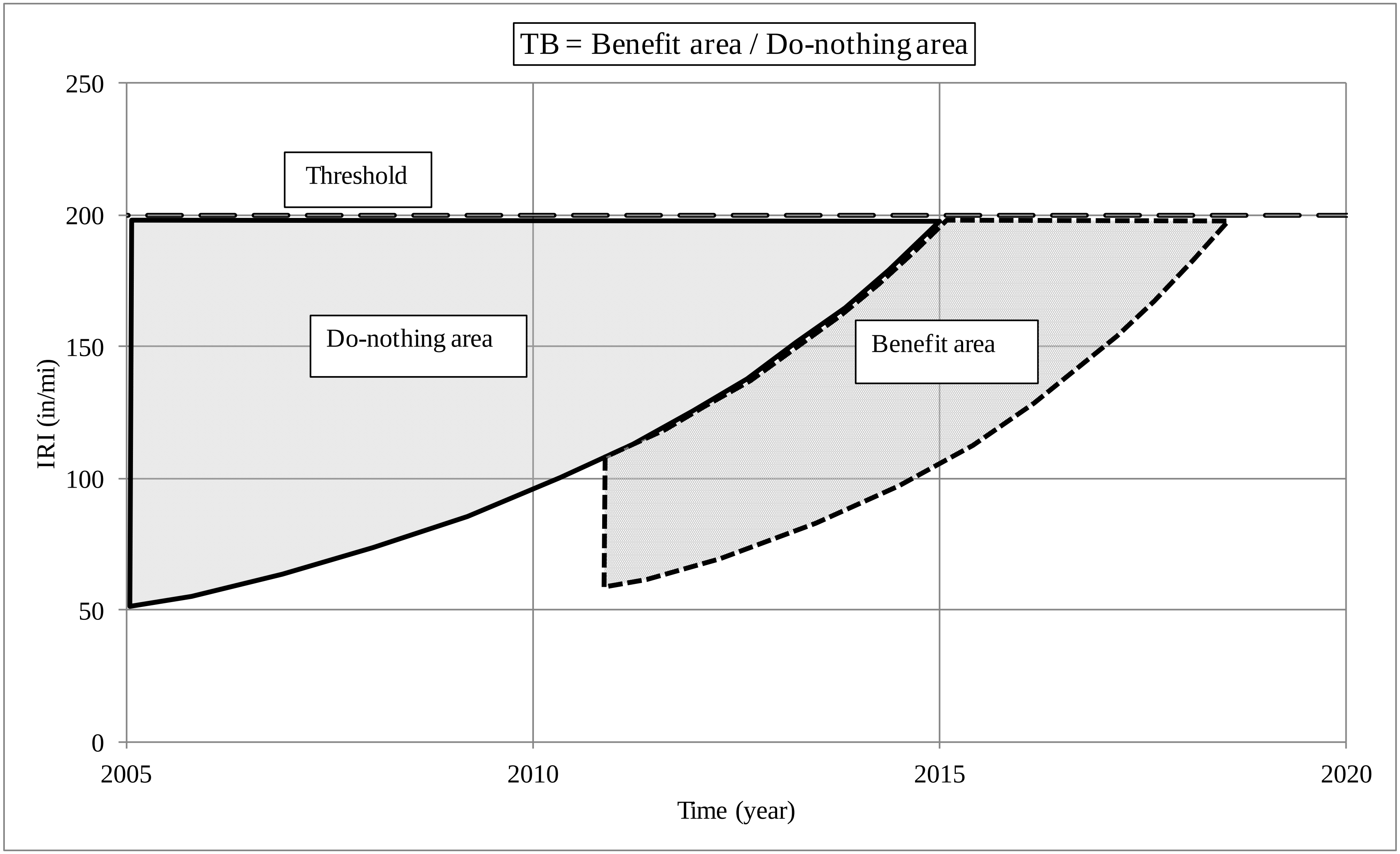


Figure 4.9 Schematic of the definition of total benefit (TB)

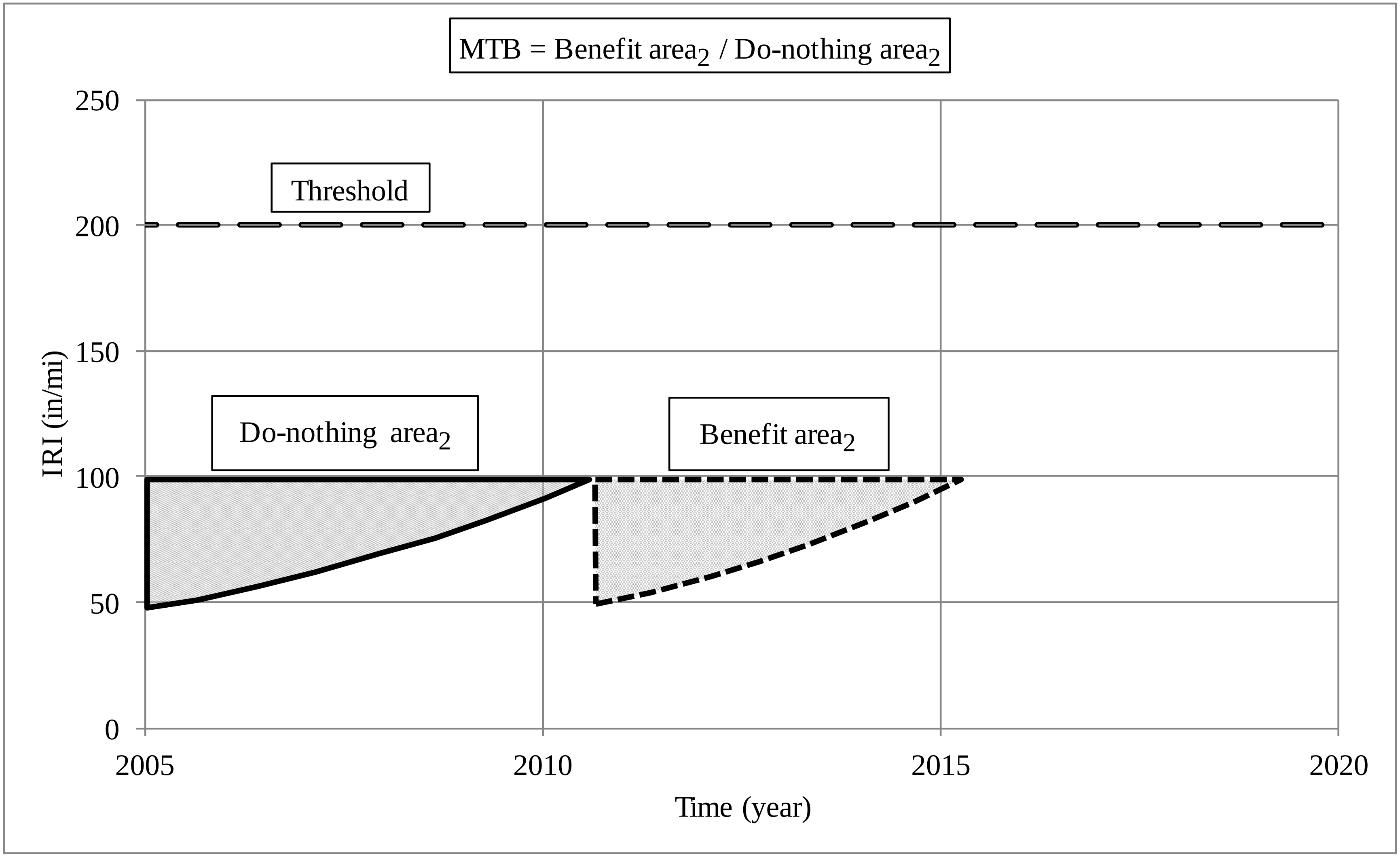


Figure 4.10 Schematic of the definition of modified total benefit (MTB)

$$\text{MODIFIEDTOTALBENEFIT} = \frac{\text{Benefit Area}_2}{\text{Do} - \text{NothingArea}_2}$$

Equation 4.6

E) Treatment Life (TL)

The TL is a new method that was developed during the course of this study to calculate the treatment benefits. The TL is defined herein as the estimated time in years between the treatment year and the year when the pavement conditions or distresses reach the lesser of the threshold value or the BT pavement condition or distress as shown in Figure 4.11. Stated differently, for those pavement segments where the pavement condition or distress are better than the threshold value, the TL is the time in years for the AT condition to reach the BT conditions. On the other hand, for those pavement segments where the pavement condition or distress are worse than the threshold value, the TL is the time in years for the AT pavement conditions to reach the threshold value. For each pavement condition and distress type, the estimation of TL requires 5 pieces of information; the last measured pavement condition or distress BT, a minimum of three measured AT pavement condition or distress data and the threshold value of the condition or distress in question. For each pavement treatment type and for each 0.1 mile long pavement segment, the maximum TL value is assumed to be equal to the maximum treatment life reported in the literature and listed in Table 4.6.

Figure 4.12 illustrates the TL concept using the measured IRI data along 4.8 mile long project along US-165, in Louisiana. Each of the solid diamonds in the figure represents the measured IRI data BT, whereas each of the open squares represents the first measured IRI data AT. Note that in few locations (beginning mile points (BMPs) 1.6, 2, and 2.2) the BT or the AT data are missing. It is common for few data points to be missing along each pavement project, as indicated in Tables 4.2 through 4.4. The data could be missing for several reasons, such as obstruction of the data collection lane or malfunction in the equipment during data collection or storage. Nevertheless,

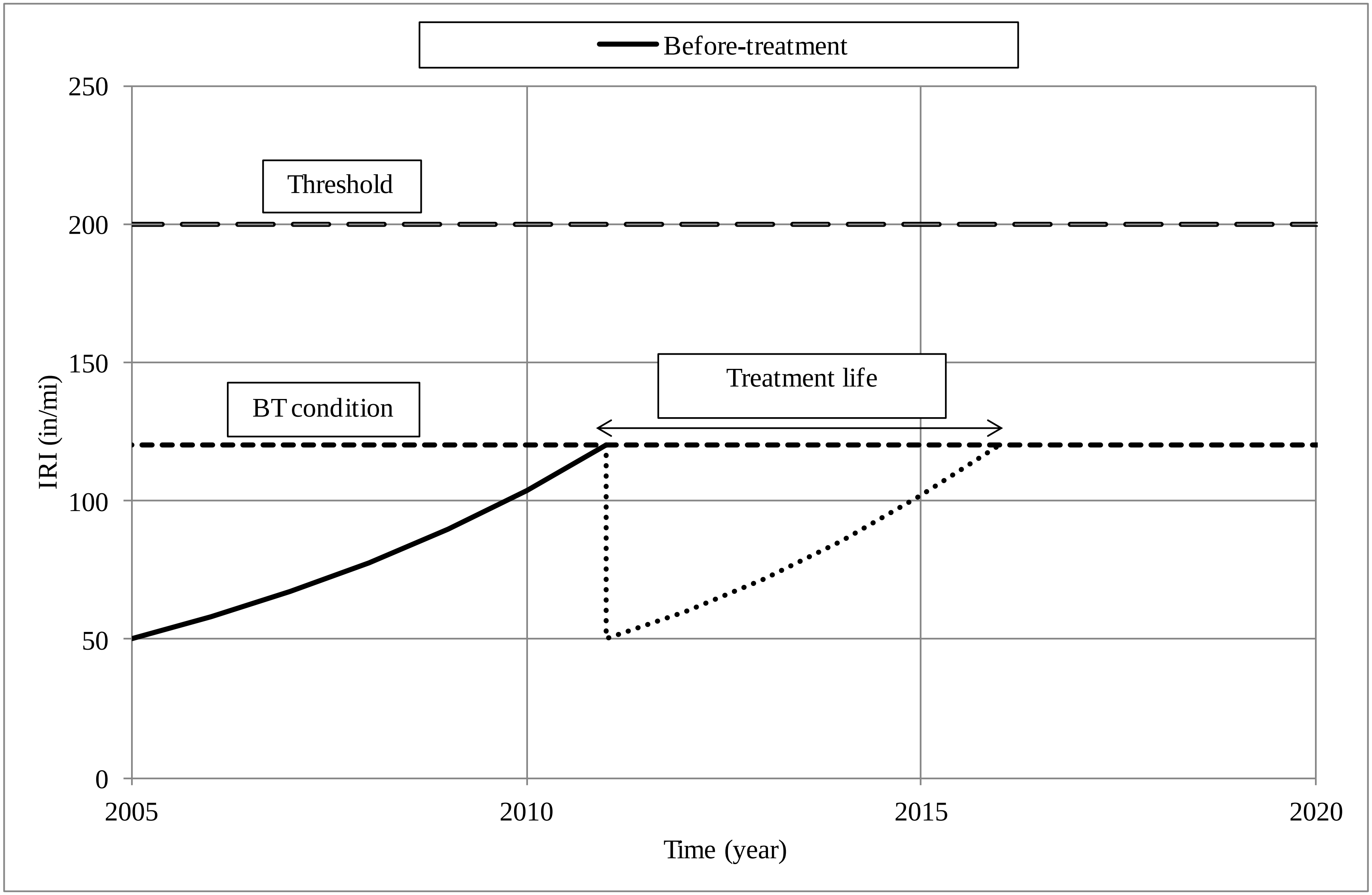


Figure 4.11 Schematic of the definition of treatment life (TL)

Table 4.6 Maximum treatment life values reported in the literature and used in the TL analyses

Treatment type	References	Maximum treatment life (year)
Thin HMA overlay	Geoffroy 1996, Hicks et al. 2000, Johnson 2000, ODOT 2001, Wade et al. 2001, MDOT 2001, Peshkin et al. 2004	8
Thick HMA overlay	FHWA 2010	10
Single chip seal	Geoffroy 1996, Hicks et al. 2000, Johnson 2000, Bolander 2005, Gransberg & James 2005	6
Double chip seal	Hicks et al. 2000, Johnson 2000, MDOT 2001, Bolander 2005, Maher et al. 2005	9
Thin mill & fill	MDOT 2001, FHWA 2010	8
Thick mill & fill	FHWA 2010	10

for each 0.1 mile long pavement segment, the TL is the required time for the AT condition or distress to mirror the BT condition or distress or to reach the threshold value whatever is shorter. As indicated in the figure, for few 0.1 mile long pavement segments (BMPs 0.3, 0.6, 0.9, and 2.0), the treatment was applied when the IRI was above the threshold value. In this case the TL is the time to reach the threshold value indicating loss of serviceability. On the other hand, in certain scenarios (BMP 4.8) and for multiple reasons, the AT pavement conditions or distresses immediately after the application of the treatment, are worse than the BT conditions or distresses (the treatment caused negative PJ). This could have been a result of poor construction practices where the HMA overlay did not smooth the pavement surface. The HMA overlay may have mirrored the existing areas of roughness and increased the roughness in other areas by applying too thick or thin asphalt layer or by compacting the asphalt mix in discontinuous movements or at improper temperatures. In these scenarios, the TL is calculated as the negative time between the treatment application and the estimated time when the BT pavement conditions or distresses would reach the AT conditions or distresses assuming the do-nothing alternative. Negative TL values represent the “loss” in the pavement life in years due to the treatment. The negative TL value is further illustrated in Figure

4.13, where the AT IRI, shown by the dotted line, is about 50 in/mi higher than the BT condition, and the estimated TL is -3 years. Finally, the absolute value of the negative TL has the same limitations as the BT RSL.

The TL method is used in this study as a measure of the treatment benefits or effectiveness.

A summary of the advantages and shortcomings of the five treatment benefits presented above are listed in Table 4.7. The AT RSL, SLE, and TL methods are practical and can be easily understood. The benefit (in years) could be expressed to engineers, management, legislators, and the public. The main shortcoming of the AT RSL is that it does not reference the do-nothing alternative. In some scenarios, the AT RSL could be shorter than the BT RSL implying negative net gain in the pavement service life. To express the gain or loss in the pavement service life, the SLE is calculated. The main shortcoming of the SLE is that the times required for the pavement conditions to reach the threshold value BT and AT have to be predicted. The error in each or both predictions could be significant depending on the variability of the data and the number of available data points. Hence, the RSL values could be over or underestimated. To reduce the amount of prediction, the TL method was developed where no BT prediction has to be made and shorter AT time prediction is required. Nevertheless, for each 0.1 mile long pavement segment, the AT RSL, SLE, and TL were calculated and discussed in this study. Further, although the BT data required to properly define the do-nothing areas are typically not available, the treatment benefits expressed in terms of the TB and the MTB were calculated and are tabulated in Appendix B.

Finally, Matlab computer program was written to calculate the pavement treatment benefits. The outputs of the program (the treatment benefits) for each 0.1 mile long pavement segment were saved in Microsoft Excel file and used to populate the treatment transition matrices, which are presented and discussed in the next few subsections.

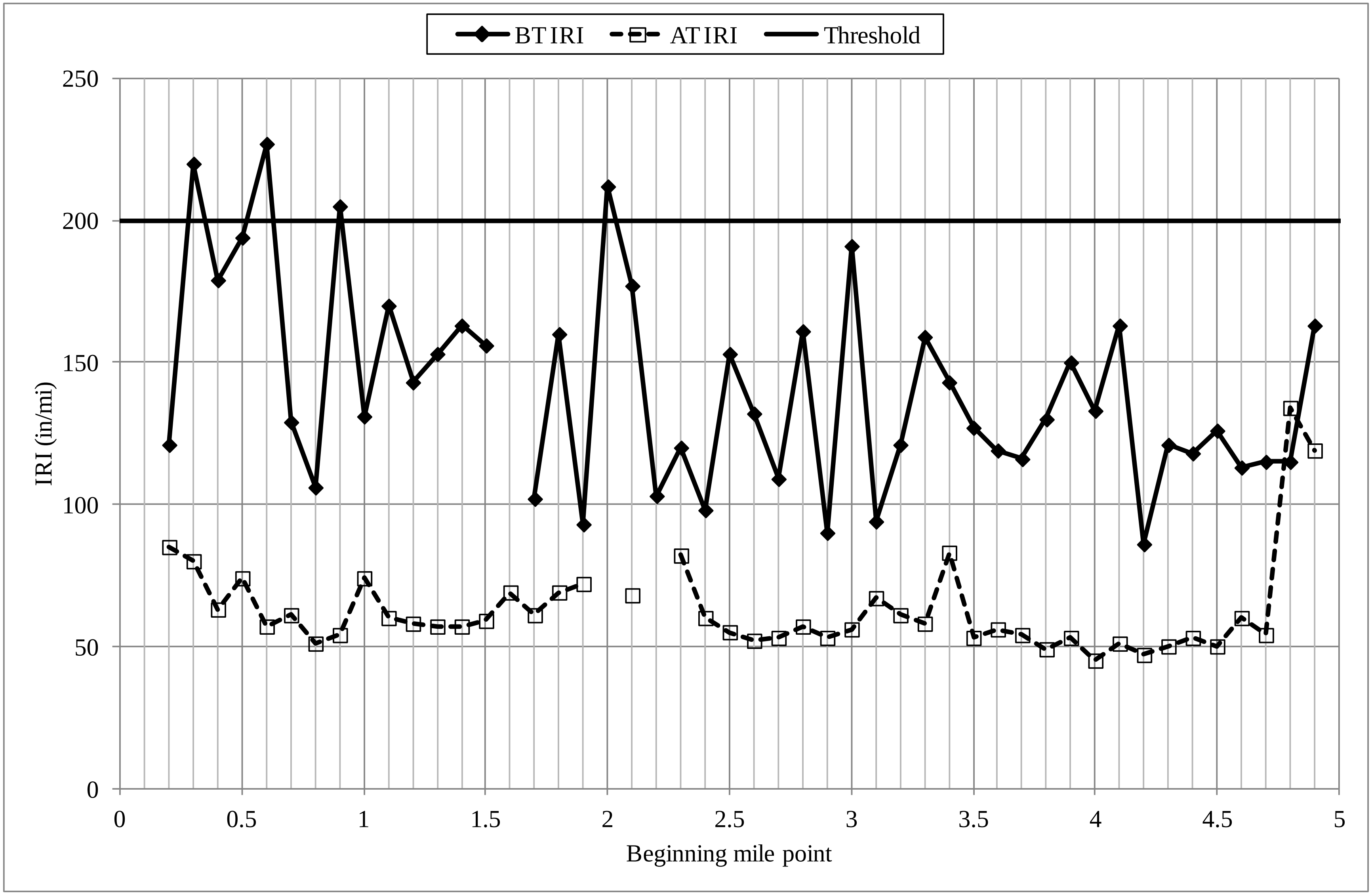


Figure 4.12 Before and after treatment pavement condition along 4.8 mile long pavement project along US-165, Louisiana

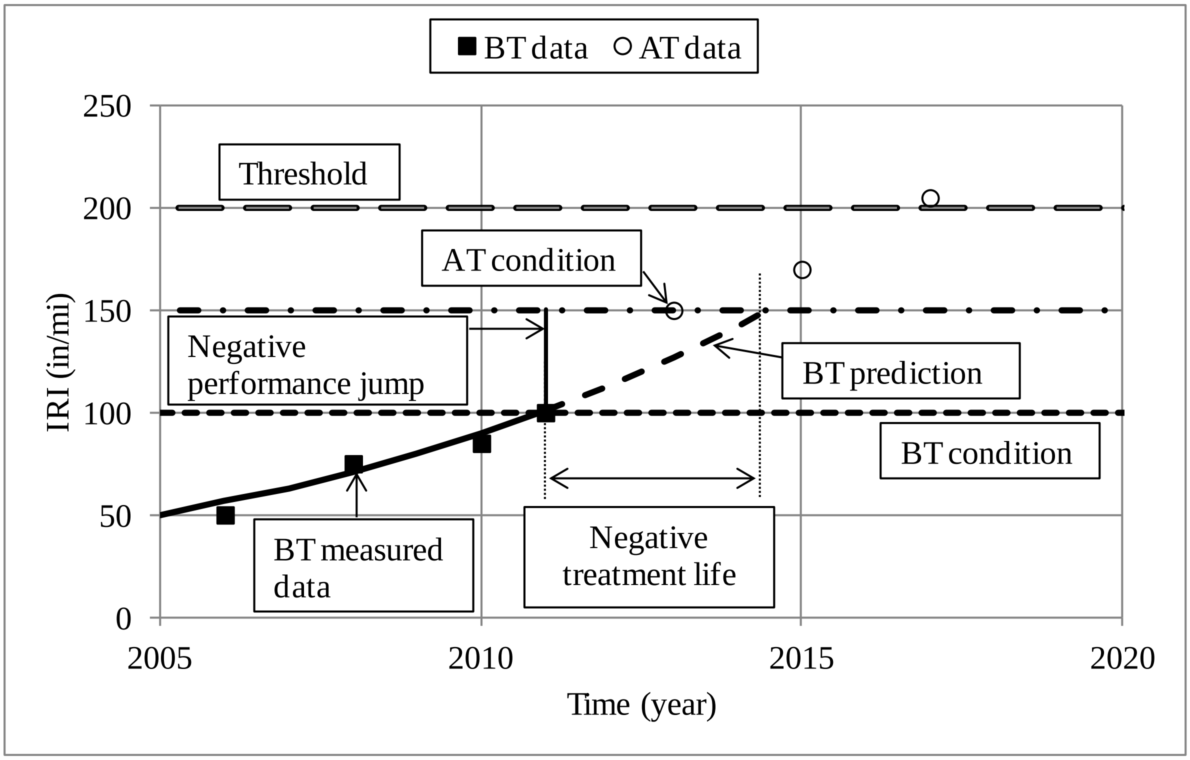


Figure 4.13 Schematic of the definition of negative TL

4.5.3 Formation of Treatment Transition Matrices (T^2Ms)

As stated earlier in this study, analysis of the pavement condition and distress data is based in parts on the BT and AT RSL of each 0.1 mile long pavement segment. At the onset, the BT and AT RSL values of each 0.1 mile long pavement segment were grouped in five brackets and referred to herein as “Condition States (CS)” or “RSL brackets”. The five RSL brackets or CSs are:

- CS 1 or RSL bracket 1 with RSL range from 0 to 2 years and average of 1 year.
- CS 2 or RSL bracket 2 with RSL range from 3 to 5 years and average of 4 years.
- CS 3 or RSL bracket 3 with RSL range from 6 to 10 years and average of 8 years.
- CS 4 or RSL bracket 4 with RSL range from 11 to 15 years and average of 13 years.
- CS 5 or RSL bracket 5 with RSL range from 16 to 25 years and average of 20.5 years.

Table 4.7 Advantages and shortcomings of five pavement treatment benefit methods

Treatment benefit methods	Advantages	Shortcomings
AT RSL	• Expresses the remaining years of service (longevity) • Expresses all pavement condition and distress types with the same benefit unit (year) • Judges all pavement segments or sections on the same threshold values	• Reference to the do-nothing scenario is not included • Requires condition predictions to AT threshold value
SLE	• Expresses the number of years of service gained or lost • Expresses all pavement condition and distress types with the same benefit unit (year) • Judges all pavement segments or sections on the same threshold values	• Requires predictions of the AT and BT pavement conditions to the threshold value
TB (Not used in the analysis)	• Judges all pavement segments or sections on the same threshold values	• Requires predictions of the AT and BT pavement conditions to the threshold value • Requires entire time-series of the BT and AT pavement conditions or distresses
MTB (Not used in the analysis)	• Requires minimal prediction of condition AT and none for BT	• Requires entire time-series of the BT and AT pavement conditions or distresses
TL	• Expresses the number of years until BT conditions return • Requires minimal prediction of condition AT and none for BT • Expresses all pavement condition and distress types with the same benefit unit (year) • Expresses the number of years gained or lost due to the treatment	

Note that higher CS or RSL bracket has wider RSL range; this is due to the increasing uncertainty associated with longer prediction of RSL into the future.

The before and after treatment CSs were then used to study the effects of the state-of-the-practice on the treatment benefits discussed in the previous section. The state-of-the-practice includes the selection of the treatment time, type, and project boundaries. The volume of the analysis results was overwhelming and requires thousands of pages to present the more than 10,000 tables in a conventional way. Hence an innovative matrix format called herein “Treatment Transition Matrix (T^2M)” was developed to:

- Tabulate the results of the analyses.
- Show the distribution of the CSs before and after treatment.
- Tabulate the standard error (SE) of the estimates of the pavement condition and distress models before and after treatment.
- Show the transitions of the CSs from BT to AT due to the treatment.
- List the treatment benefits based on the estimated TL, SLE, and AT RSL.
- Provide a “snap-shot in time” of the results of the pavement treatment.

Table 4.8 shows a generic T^2M that can be used for any treatment type and any type of pavement condition or distress. The cells of the T^2M display the above listed information in a convenient way. The T^2M format shown in Table 4.8 is used throughout this research to tabulate the results of the pavement treatment analyses. For the 0.1 mile long pavement segments along a pavement project that were subjected to certain treatment type by each district and each of the three SHAs included in this study, and for each pavement condition and distress type, a T^2M was generated that lists the project information before and after treatment and the treatment benefits. Nevertheless, the information included in the various columns of Table 4.8 are detailed below.

Table 4.8 A generic treatment transition matrix (T^2M)

Row designation	Column designation												
	A	B	C	D	E	F	G	H	I	J	K	L	M
	Before treatment (BT) data					After treatment (AT) data							
						CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
	CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE)	1	2	3	4	5	TL	SLE	RSL
						0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
			Number	Percent		SE of each CS or RSL bracket							
A	1	0 to 2											
B	2	3 to 5											
C	3	6 to 10											
D	4	11 to 15											
E	5	16 to 25											
F	Total/average												

- Columns A through E of Table 4.8 list the following BT information: the CS (the RSL bracket numbers), the RSL ranges, the number of 0.1 mile long pavement segments in each BT CS, the percentages of the 0.1 mile long pavement segments in each CS, and the SE of the estimates for the pavement condition or distress models for each CS. The SE was calculated using Equation 4.7.

$$SE = \sqrt{\frac{\sum_{i=1}^{n}\left(y_{m_i} - y_{pi}\right)^2}{(n - df)}} \qquad \text{Equation 4.7}$$

Where, SE is the standard error;

n is the number of measured data points;

y_m are the measured distress data;

y_p are the predicted distress data;

df is the degrees of freedom (2 for the power and exponential functions)

- Columns F through J of Table 4.8 list the following AT information: the CS (the RSL bracket numbers), the RSL ranges, the SE of the estimates for the pavement condition or distress models for each CS, and the number or the percent of 0.1 mile long pavement segments transitioned from the given BT CS to each AT CS and the total number or percent of 0.1 mile long pavement segments transitioned to each AT CS.
- Columns K through M of Table 4.8 list the following pavement treatment benefits: the average TL, SLE, and AT RSL of all 0.1 mile long pavement segments transitioned from a given BT CS to all AT CSs, and the overall average TL, SLE, and AT RSL.

The data analyses and the population of the T^2Ms were accomplished using the following general steps:

Step 1 - For each 0.1 mile long pavement segment along a given treated pavement project, the BT and AT time-series pavement condition and distress data were tested using the first acceptance criterion.

Step 2 – For each 0.1 mile long pavement segment along a given treated pavement project that passed the first acceptance criterion, the BT and AT time-series pavement condition and distress data were modeled using the proper mathematical function (power function for rut depth and exponential function for IRI and cracking).

Step 3 – For each 0.1 mile long pavement segment along a given treated pavement project that passed the first acceptance criterion, test the regression parameters obtained in Step 2 using the second acceptance criterion.

Step 4 – For each 0.1 mile long pavement segment along a given treated pavement project that passed the two acceptance criteria, use the appropriated mathematical function (model) to calculate the BT and AT RSL values and the treatment benefits.

Step 5 – Place each 0.1 mile long pavement segment in the proper BT and AT CS.

Step 6 – List the number and/or percent of 0.1 mile long pavement segments in each BT and AT CS and the treatment benefits in the T^2Ms.

Step 7 – Populate the various tables and figures listing and showing the criteria acceptance rates, the SEs, and the controlling pavement condition and distresses.

To illustrate the implementation of each of the above steps, an example of an asphalt surfaced pavement section that was subjected in 2004 to 1.5-inch HMA overlay is presented below and discussed in the next section. The example addresses the 0.1 mile long pavement segment at the BMP 202.8 of a 9.5 mile long pavement project along US-385, control section 385C, direction 1, in Colorado.

Step 1 – First acceptance criterion

- Arrange the BT and AT time-series pavement condition and distress data as shown in Table 4.9.

Table 4.9 Time-series data, US-385, control section 385C, direction 1, BMP 202.8, Colorado

	Before treatment (BT) data			After treatment (AT) data			
Data collection year	2000	2001	2003	2004	2005	2007	2009
Elapsed time (year)	1	2	4	0.1	1	3	5
IRI (in/mi)	114	121	159	-	136	178	200
Rut depth (in)	0.06	0.12	0.12	0.01	0.18	0.15	0.3
Alligator cracking (ft)	6	5	35	-	1	15	42
Longitudinal cracking (ft)	41	66	48	-	22	15	1
Transverse cracking (ft)	276	288	312	-	324	384	300

- Calculate the elapsed time in years using the data collection year and the pavement surface age if known. In this study, the pavement surface age is typically unknown, so the first BT data collection year 2000 was assigned an elapsed time of 1 year. Since the overlay was constructed in 2004, an elapsed time of 1 year was assigned to the data collection year in 2005.
- Subject the BT and AT time-series data to the first acceptance criterion. All pavement condition and distress data of this 0.1 mile long pavement segment satisfied the first acceptance criterion, as indicated in Table 4.10.

Table 4.10 Pavement condition models and treatment benefits, US-385, control section 385C, direction 1, BMP 202.8, Colorado

Pavement condition type	BT pavement condition model parameters		AT pavement condition model parameters		Pass acceptance criteria		RSL (year)		Treatment benefits (year)	
	Alpha	Beta	Alpha	Beta	1	2	BT	AT	TL	SLE
IRI	99	0.11	127	0.1	Yes	Yes	1	5	3	4
Rut depth	0.07	0.5	0.09	0.82	Yes	Yes	20	8	2	-12
Alligator cracking	2.21	0.64	0.66	0.88	Yes	Yes	1	6	5	5
Longitudinal cracking	48.08	0.02	70.20	-0.77	Yes	No	-	-	-	-
Transverse cracking	265.17	0.04	354.05	-0.019	Yes	No	-	-	-	-
Overall	-	-	-	-	-	-	1	5	2	4

Step 2 – Modeling the time-series data

- Model each of the BT and AT time-series pavement condition and distress data using the appropriate mathematical functions, discussed in section 4.4. The regression parameters of each model are listed in Table 4.10. The BT and AT IRI data are shown in Figure 4.14.
- Calculate the standard error of the estimates for the BT and AT models of each pavement condition and distress type.

Step 3 – Second acceptance criterion

- Test the regression parameters of the pavement models of each pavement condition and distress type using the second acceptance criterion. For the data listed in Table 4.9, Table 4.10 provides a list of the results. As can be seen, the longitudinal and transverse cracking data produced negative regression parameters. Hence, the data failed the second acceptance criterion and they were not included in any further analyses.

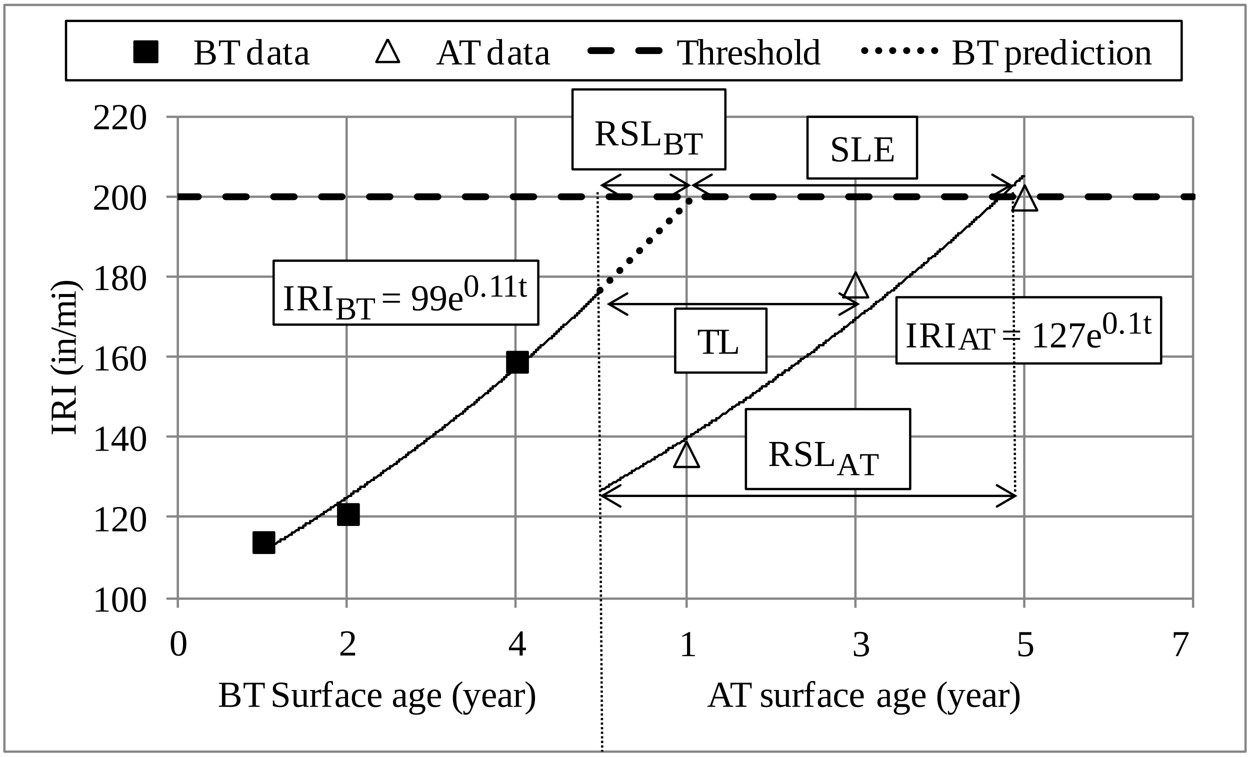


Figure 4.14 BT and AT IRI data, condition models, RSL, and treatment benefits, US-385, control section 385C, direction 1, BMP 202.8, Colorado

Step 4 – Calculation of RSL and treatment benefits

- Use the resulting regression model and the pre-specified threshold values to calculate the BT and AT RSL values for each pavement condition and distress type which passed the two acceptance criteria. The BT and AT RSL are schematically shown in Figure 4.14 and are listed in Table 4.10 for each pavement condition and distress type.
- Determine the minimum of the BT and AT RSL values of all pavement condition and distress types. In this example, the IRI and alligator cracking BT RSL of 1 year and AT RSL of 5 years are the minimum values and hence control the RSL (see Table 4.10).
- Use the pavement models to calculate the TL (the predicted time for each of the AT pavement conditions and distresses to reach the BT values). The estimated TL values for all pavement condition and distress types are listed in Table 4.10.

- Determine the minimum TL value, for this example the minimum TL is 2 years for rutting (see Table 4.10). This does not imply that a pavement treatment will be required in 2 years to address the rut depth and to restore pavement serviceability. In fact, 2 years AT the rut depth will be 0.12-inch only. The minimum TL value indicates the shortest time for the AT conditions or distresses to reach the BT condition or distress level.
- Calculate the SLE by subtracting the BT RSL from the AT RSL for each pavement condition and distress type. The estimated SLE values for all pavement condition and distress types are listed in Table 4.10. Note that, the SLE for rutting is negative 12 years. That is the 1.5-inch HMA overlay caused a loss in the pavement service life due to rutting. This could be the result of a soft HMA overlay.
- Determine the minimum SLE value, for this example the minimum SLE is negative 12 years for rutting (see Table 4.10). The data in the table indicate that the BT RSL of the 0.1 mile long pavement segment is 20 years, and 8 years AT the pavement segment will require an action to solve the anticipated rut problem. However, the pavement segment needs treatment for IRI in only 5 years. Thus, the IRI not the rut depth controls the timing of the future treatment.

Step 5 – Condition state (CS) or RSL brackets

- Place the BT and AT RSL into the appropriate CS (RSL bracket). In this example, the IRI BT CS was 1 and AT CS was 2.

Step 6 – Treatment transition matrices (T^2Ms)

- Populate a T^2M by listing the number of 0.1 mile long pavement segments in each BT CS and the number transitioned to each AT CS. Table 4.11 is the T^2M for the IRI data for the 70 accepted pavement segments of the 9.5 mile long example project.

- Populate another T^2M by listing the percent of the pavement segments transitioned from each individual BT CS to each AT CS. Table 4.12 is the T^2M for all of the IRI data for the 70 accepted pavement segments of the 9.5 mile long example project.
- Populate a third T^2M by listing the percent of the total treated pavement segments transitioned to each AT CS. Table 4.13 is the T^2M for the IRI data for the 70 accepted pavement segments of the 9.5 mile long example project.

Step 7 – Summary tables

- List the results of the acceptance criteria for the entire project in a format similar to that shown in Table 4.14.
- List the SE of the estimates of all pavement condition and distress models for each BT and AT CS. The SE results for the IRI data of the example pavement project are listed in Table 4.15.
- Draw figures showing the pavement condition and distress types controlling the BT and AT RSL values as shown in Figures 4.15 and 4.16.

4.5.4 Treatment Transition Matrices (T^2Ms)

For each of the three SHAs; Colorado, Louisiana, and Washington, T^2Ms were populated for each of the six pavement treatment types listed below:

- Thin HMA overlay of asphalt surfaced pavement
- Thick HMA overlay of asphalt surfaced pavement
- Single chip seal

Table 4.11 T^{2}M based on IRI for 1 pavement project along US-385, control section 385C, direction 1, Colorado, subjected to thin HMA overlay (The AT section is listing the number of 0.1 mile long pavement segments in each CS or RSL bracket)

Condition/distress type: IRI												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (in/mi)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each CS or RSL bracket (in/mi)							
						11	9	6	6			
1	0 to 2	8	11	8	**0**	8	0	0	0	2	3	4
2	3 to 5	29	41	12	0	**7**	18	2	2	2	4	8
3	6 to 10	19	27	10	0	1	**12**	2	4	1	3	11
4	11 to 15	7	10	7	0	0	3	**1**	3	0	1	14
5	16 to 25	7	10	4	0	0	4	2	**1**	1	-9	11
Total/average		70/	100/		0/	16/	37/	7/	10/	/2	/2	/9

Table 4.12 T^2M based on IRI for 1 pavement project along US-385, control section 385C, direction 1, Colorado, subjected to thin HMA overlay (The AT section is listing the percent of the BT 0.1 mile long pavement segments transitioned from each BT CS or RSL bracket)

Condition/distress type: IRI												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the percent of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (in/mi)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each CS or RSL bracket (in/mi)							
						11	9	6	6			
1	0 to 2	8	11	8	**0**	100	0	0	0	2	3	4
2	3 to 5	29	41	12	0	**24**	62	7	7	2	4	8
3	6 to 10	19	27	10	0	5	**63**	11	21	1	3	11
4	11 to 15	7	10	7	0	0	43	**14**	43	0	1	14
5	16 to 25	7	10	4	0	0	57	29	**14**	1	-9	11
Total/average		70/	100/		0/	23/	53/	10/	14/	/2	/2	/9

Table 4.13 T^{2}M based on IRI for 1 pavement project along US-385, control section 385C, direction 1, Colorado subjected to thin HMA overlay (The AT section is listing the percent of the project transitioned to each CS or RSL bracket)

Condition/distress type: IRI												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the percent of the 0.1 mile long pavement segments transitioned from the project to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (in/mi)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each CS or RSL bracket (in/mi)							
						11	9	6	6			
1	0 to 2	8	11	8	**0.00**	11.43	0.00	0.00	0.00	2	3	4
2	3 to 5	29	41	12	0.00	**10.00**	25.71	2.86	2.86	2	4	8
3	6 to 10	19	27	10	0.00	1.43	**17.14**	2.86	5.71	1	3	11
4	11 to 15	7	10	7	0.00	0.00	4.29	**1.43**	4.29	0	1	14
5	16 to 25	7	10	4	0.00	0.00	5.71	2.86	**1.43**	1	-9	11
Total/average		70/	100/		0/	23/	53/	10/	14/	/2	/2	/9

Table 4.14 The number and/or percent of 0.1 mile long pavement segments passed the before and after treatment acceptance criteria, US-385, control section 385C, direction 1, Colorado

Pavement condition/ distress types	Total number of 0.1 mile long pavement segments	Percent of 0.1 mile long pavement segments passing the acceptance criteria before and after treatment				Statistics of the 0.1 mile long pavement segments passing the acceptance criteria BT and AT	
		Before treatment (BT)		After treatment (AT)			
		3 data points	Positive regression parameters	3 data points	Positive regression parameters	Number	Percent
IRI	95	100.00	82.11	100.00	90.53	70	73.68
RD	95	100.00	56.84	100.00	100.00	54	56.84
AC	95	100.00	67.37	100.00	65.26	43	45.26
LC	95	100.00	60.00	100.00	34.74	18	18.95
TC	95	100.00	92.63	100.00	51.58	43	45.26
Percent of 0.1 mile long pavement segments passing each acceptance criteria		100.00	71.79	100.00	68.42	Overall acceptance (%) 48.00	
RD = rut depth; AC = alligator cracking; LC = longitudinal cracking; TC = transverse cracking							

- Double chip seal
- Thin mill and fill of asphalt surfaced pavement
- Thick mill and fill of asphalt surfaced pavement

Note that other pavement treatment types, such as crack sealing, dowel bar retrofit, and Portland cement concrete rehabilitation were also conducted by each state. However, the number of pavement sections and the available data were insufficient to support their inclusion in the analyses. Nevertheless, the number of projects and the total length in miles for each pavement treatment type in each state, and the total number of projects and miles included in the analyses are listed in Table 4.16. The detailed state by state results are presented and discussed below.

Table 4.15 The BT and AT pavement model SE of the estimates, US-385, control section 385C, direction 1, Colorado

Condition/distress type: IRI								
Before treatment (BT) data				CS or RSL bracket number and range in years, the number of the 0.1 mile long pavement segments transitioned from each BT RSL bracket to the indicated AT RSL brackets, and the BT and AT SE for the indicated number of 0.1 mile pavement segments per RSL bracket (in/mi)				
CS or RSL bracket number	RSL bracket range (year)	Number of 0.1 mile pavement segments	Standard error (SE) of each CS or RSL bracket (in/mi)	1	2	3	4	5
				0 to 2	3 to 5	6 to 10	11 to 15	16 to 25
1	0 to 2	8	8	***0***	***8***	***0***	***0***	***0***
					8			
					15			
2	3 to 5	29	12	***0***	***7***	***18***	***2***	***2***
					10	9	24	29
					7	7	5	6
3	6 to 10	19	10	***0***	***1***	***12***	***2***	***4***
					18	8	8	14
					1	10	4	8
4	11 to 15	7	7	***0***	***0***	***3***	***1***	***3***
						7	6	7
						14	15	4
5	16 to 25	7	4	***0***	***0***	***4***	***2***	***1***
						6	3	0
						12	4	1
Total		70						
AT SE of each AT CS or RSL bracket (in/mi)					11	9	6	6
Bold italic figures are the number of AT 0.1 mile long pavement segments				Dotted area designates the BT SE of the indicated number of 0.1 mile long pavement segments			Shaded area designates the AT SE of the indicated number of 0.1 mile long pavement segments	

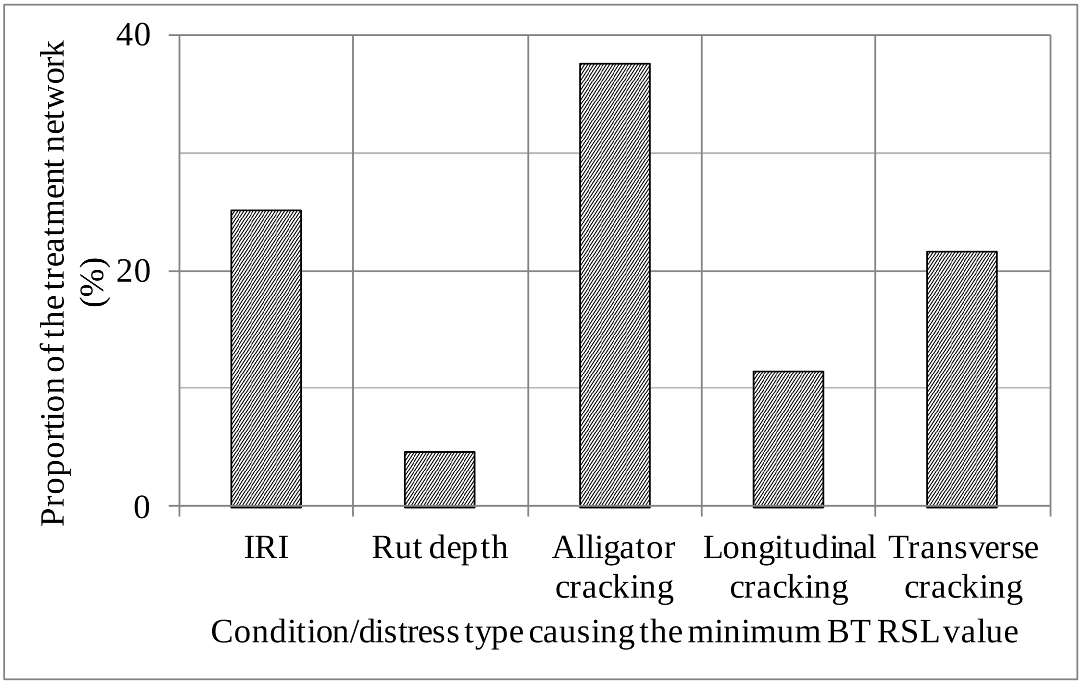


Figure 4.15 The percent of the treatment network having the minimum BT RSL value based on the indicated controlling pavement condition and distress type, US-385, control section 385C, direction 1, Colorado

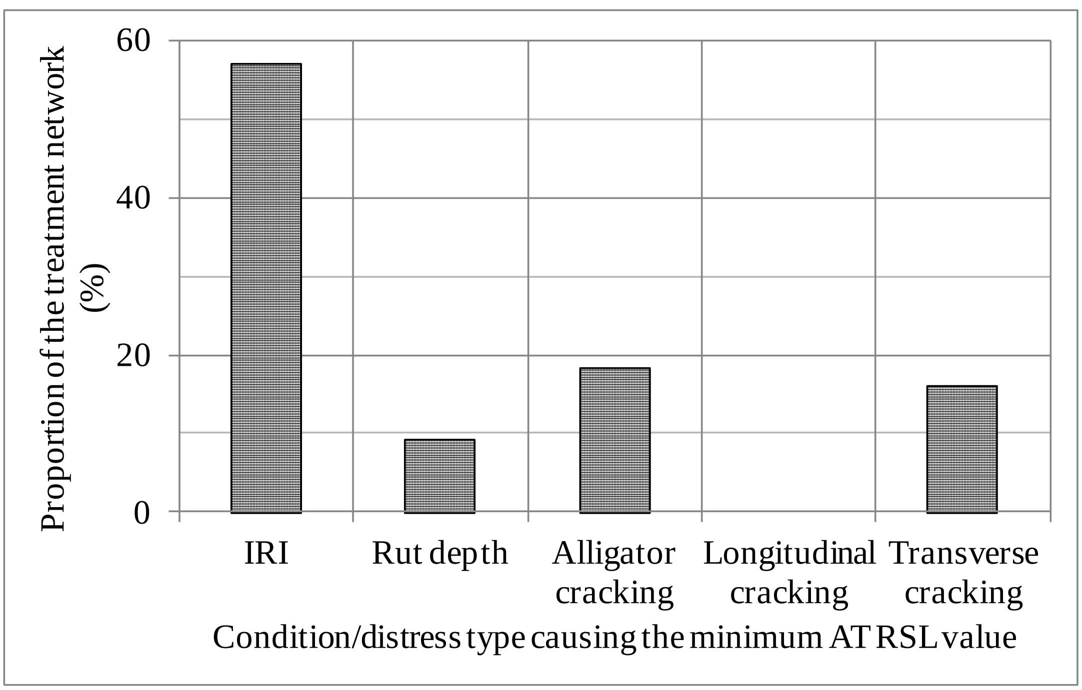


Figure 4.16 The percent of the treatment network having the minimum AT RSL value based on the indicated controlling pavement condition and distress type, US-385, control section 385C, direction 1, Colorado

Table 4.16 The number of pavement projects and total length analyzed in each state

Treatment type	The number of pavement projects, and the lengths in miles						Total	
	Colorado		Louisiana		Washington			
	Projects	Miles	Projects	Miles	Projects	Miles	Projects	Miles
A	57	87.8	18	43.9	71	205.9	146	337.6
B			58	227.9	11	46.1	69	274.0
C	69	403.3	46	213.1	7	20.3	122	636.7
D			12	31.6			12	31.6
E	2	8.3	11	31.1	22	94.6	35	134.0
F			32	128.0			32	128.0
Total	128	499.4	177	675.6	111	366.9	416	1,541.9
Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.								

a) Colorado DOT (CDOT)

The time-series pavement condition and distress data from the CDOT were analyzed for thin HMA overlay of asphalt surfaced pavement, single chip seal, and thin mill and fill treatments of asphalt surfaced pavement. Various T^2Ms were populated for each project (as detailed in the previous section), for each district, and for the state. The analyses results for thin HMA overlay of asphalt surfaced pavements are detailed and discussed below. Similar results were obtained of the other treatment types and are on a DVD in Appendix B.

1) Standard Error (SE) of the Estimates

Table 4.11 lists the T^2M for the IRI data from an asphalt surfaced pavement section that was subjected in 2004 to 1.5-inch HMA overlay. The 9.5 mile long pavement project is located along US-385, control section 385C, direction 1, in Colorado. Please note that 7 miles of the 9.5 miles (74%) passed the two acceptance criteria and were accepted for analysis. Table 4.11 lists, among other results, the SE of the estimates for each BT and AT CS (RSL bracket). The data in the table indicate that the SE ranges from 4 to 12 in/mi. This SE is only 10% or less of the average condition of the last data collection year BT of 120 in/mi, which is very reasonable and indicates a good fit of the model to the measured data. Similarly, the AT SE ranges from 6 to 11 in/mile, which is only 6% or less of the average AT condition of the last data collection year AT of 171 in/mile. T^2Ms were also populated at the district and state levels. Table 4.17 lists the T^2M for the IRI data for all thin HMA overlay of asphalt surfaced pavement in District 1 of Colorado, while Table 4.18 lists the T^2M for thin HMA overlays for all districts in the State of Colorado (the state-of-the-practice in Colorado). The SE ranges from 6 to 21 in/mi at the district and state levels. This range of SE is slightly larger than that of the estimates at the project level, listed in Table 4.11. Larger SE of the estimates implies larger variability in the time-series pavement condition data and/or higher measured pavement condition and distress data. Nevertheless, the maximum SE is still within 15% of the average condition of the last data collection year BT of 140 in/mi, which is very reasonable. Table 4.19 summarizes the SE values of the estimates of the fitted models for each pavement condition and distress type and for each pavement treatment type in Colorado. The SE could be misleading because the pavement conditions and distresses are increasing with time and vary from one 0.1 mile long pavement segment to the next. To illustrate, for a 0.1 mile long pavement segment with 50 in/mi measured IRI, an SE of 20 in/mi could be very high; whereas the same SE is insignificant for an adjacent pavement segment with 300 in/mi measured IRI. In addition, the units of measurement are different, such as in/mi for IRI, inch for rut depth, and linear feet for cracking. Hence, the SE of the estimates was compared to the average condition of the last data collection year BT for each pavement condition and distress type. The data in Table 4.19 indicate that the SE of the estimates, for each pavement condition and distress type, are reasonable both before and after treatment when compared to the average BT conditions and distresses for IRI and rut depth. On the other hand, the alligator, longitudinal, and transverse cracking standard errors of the estimates are relatively high when compared to the average cracking data of the last data collection year BT. Higher SE values of the estimates are likely due to the large amount of variability in the cracking data resulting from the subjectivity involved in the process of crack identification and length and severity rating, as discussed in section 3.2 of Chapter 3. Note that the alligator cracking SE of the estimates are low, however they are a relatively high when compared to the magnitude of the measured alligator cracking data. Nevertheless, the resulting mathematical models express the limited time-series pavement condition and distress data well and could fit better (lower SE) if time-series data were available for a longer time period and/or if the variability in the data were reduced through better technology and/or quality control.

2) Distribution of 0.1 Mile Long Pavement Segments in each Condition State (CS)

The BT and AT distribution of the 0.1 mile long pavement segments in each CS or RSL bracket are listed in the T^2M of Table 4.11 for the 7 miles, which were accepted for analysis, of the 9.5 mile long pavement project identified in the table. The data in the table indicate that the majority (41 and 27 percent) of the 0.1 mile long pavement segments were in BT CSs 2 and 3, respectively, with smaller percentages in BT CSs 1, 4, and 5. This distribution indicates that the RSL of about two thirds of the 7 accepted miles varies from 3 to 10 years before the 1.5-inch HMA overlay was applied. The remaining one third of the accepted mileage is almost equally distributed among the other three CSs with only 11 percent of the project in CS 1 (RSL of 0.0 to 2.0 years). Such BT distribution was expected since, as it is well documented in the literature, pavement treatments applied to pavement segments in better condition typically last longer. This distribution of the 0.1 mile long pavement segments in the BT CSs implies that the project was

Table 4.17 T^2M based on IRI for thin HMA overlay of asphalt surfaced pavements, District 1, Colorado

Condition/distress type: IRI												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (in/mi)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each CS or RSL bracket (in/mi)							
					21	15	15	11	6			
1	0 to 2	127	29	17	**10**	20	22	12	63	6	12	13
2	3 to 5	95	22	13	3	**19**	41	6	26	4	7	11
3	6 to 10	87	20	11	1	12	**26**	17	31	3	5	13
4	11 to 15	50	11	10	0	9	9	**6**	26	2	1	14
5	16 to 25	82	19	12	0	2	10	14	**56**	0	-3	17
Total/average		441/	100/		14/	62/	108/	55/	202/	/3	/5	/13

Table 4.18 T^2M based on IRI for thin HMA overlay of asphalt surfaced pavements, State of Colorado

Condition/distress type: IRI												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (in/mi)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each CS or RSL bracket (in/mi)							
					21	15	15	11	6			
1	0 to 2	168	30	19	**10**	21	29	26	82	7	13	14
2	3 to 5	120	22	14	3	**21**	46	13	37	4	7	11
3	6 to 10	106	19	12	1	12	**28**	19	46	4	6	14
4	11 to 15	61	11	11	0	9	10	**8**	34	3	2	15
5	16 to 25	102	18	11	0	2	12	18	**70**	1	-3	17
Total/average		557/	100/		14/	65/	125/	84/	269/	/4	/6	/14

Table 4.19 Model SE of the estimates summary in the State of Colorado

Treatment type	Pavement condition and distress types and measurement units	Number of 0.1 mile long pavement segments modeled	Average condition or distress of the last data collection year BT	Model standard error of the estimates	
				Before treatment (BT)	After treatment (AT)
A	IRI (in/mi)	557	141	14	10
	RD (in)	559	0.25	0.05	0.10
	AC (ft)	506	36	14	23
	LC (ft)	384	156	124	102
	TC (ft)	385	318	96	88
B					
C	IRI (in/mi)	2,281	113	13	8
	RD (in)	399	0.16	0.04	0.05
	AC (ft)	2,228	31	14	17
	LC (ft)	2,440	175	120	124
	TC (ft)	2,163	179	50	87
D					
E	IRI (in/mi)	28	202	55	17
	RD (in)	74	0.27	0.08	0.04
	AC (ft)	49	32	20	11
	LC (ft)	38	129	129	194
	TC (ft)	24	87	75	144
F					

Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

IRI (in/mi); RD = rut depth (in); AC = alligator cracking (ft); LC = longitudinal cracking (ft); TC = transverse cracking (ft)

likely selected as a preventive maintenance action to extend the service life of pavement in relatively good condition. Figure 4.17 depicts the BT and AT distribution of the CSs listed in Table 4.11. It can be clearly seen that the distribution has shifted to the right due to the treatment and fifty three percent of the project transitioned to CS 3 and CS 1 was evacuated. The detailed BT and AT distributions of the number of 0.1 mile long pavement segments in the various CSs are listed in Table 4.11. To illustrate, consider the AT section of Table 4.11. The numbers of the 0.1 mile long pavement segments listed along the diagonal indicates no gain in the CS of these segments due to treatment. The numbers in the cells above the diagonal indicate gain in the service life; whereas the cells below the diagonal indicate losses in service life. Thus, for this project, the 1.5 inch HMA overlay caused the following changes in the CSs along the project:

- The CS of thirty nine 0.1 mile long pavement segments or 56 percent of the project improved because of the 1.5-inch HMA overlay. It should be noted that the BT RSL of 28 of the 39 segments varied from 3 to 10 years.
- The CS of twenty one segments or about 30 percent of the project was not affected by the 1.5-inch HMA overlay.
- The CS of ten 0.1 mile long pavement segments or about 14 percent of the project worsened due to the 1.5-inch HMA overlay. Note that the BT RSL of 9 of these ten 0.1 mile long pavement segments was higher than 11years.

This AT distribution of the pavement segments in the various CSs is typical as the construction quality varies along the pavement project. The distribution of the 0.1 mile long pavement segments in the various AT CSs could result from the differences in the BT conditions and distresses, the structural capacity of the treated pavement, and/or the quality of the treatment, such as material quality and good quality control during application of the treatment. Loss in the RSL implies that the pavement treatment did not improve the pavement conditions and distresses and/or increased the pavement rate of deterioration. Also note that, pavement segments with high BT RSL are not likely to gain much service life, if any, as a result of the treatment. In fact, breaking-even is the best case scenario for pavement segments from BT CS 5.

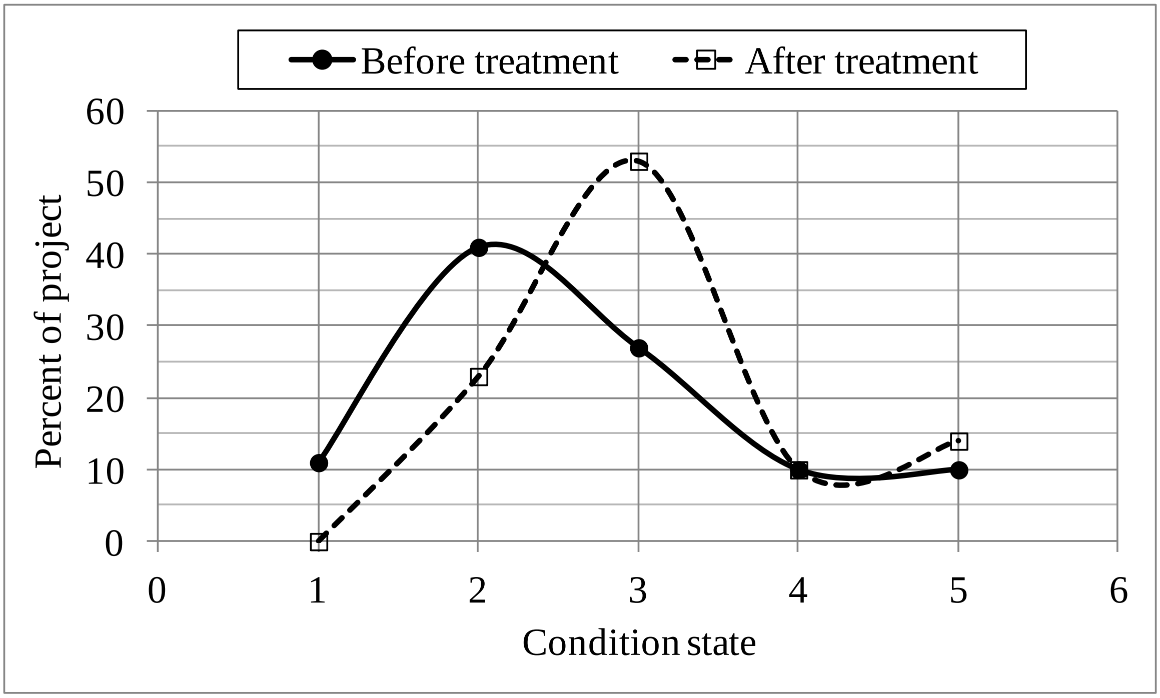


Figure 4.17 Before and after treatment distribution of the condition states of the pavement project listed in Table 4.11

The above noted transition of the CSs of the various 0.1 mile long pavement segments suggest that the 1.5-inch HMA overlay (a preservation treatment) is highly effective when applied to pavement segments having RSL between 3 and 10 years. It is not as effective for pavement segments in lower or higher condition states. The implication is that performing the treatment on pavement segments in CSs 2 and 3 is the most effective treatment timing. Note that, the information under the treatment benefits portion of the T^2M are designed to clearly display the benefits versus the BT CSs. Hence, the most effective treatment timing could be determined.

T^2Ms were also populated at the district and state levels. Table 4.17 lists the T^2M for the IRI data for all thin HMA overlay of asphalt surfaced pavement in District 1 of Colorado. The data in the table indicate almost uniform distribution of the 0.1 mile long pavement segments in most BT CSs with the highest percentage in CS 1. This implies that, at the district level, the selected project boundaries include almost equal pavement segments in all CSs. The data also indicate higher percentage in AT CS 5, which implies higher AT RSL compared to the single pavement project listed in Table 4.11. The distribution of the pavement projects in the various AT CSs is the direct result of variation in construction quality and perhaps, variations in the BT CSs. Further, Table 4.18 lists the T^2M for the IRI data for all thin HMA overlay of asphalt surfaced pavement in the State of Colorado. The data in this figure are similar to those listed in Table 4.17. This implies that the practice of selecting project boundaries and the variability in construction quality in District 1 are more or less representative of those of the other districts.

Additionally, several statistical tests were assessed to evaluate the association of the BT and AT CSs (RSL brackets) in the T^2Ms. It was determined that the Kendall's (or Stuart's) Tau-c test was most appropriate for the situation. The test is specifically designed for sets of ordinal variables such as the RSL brackets. Note that ordinal variables do not establish the numeric difference between data points; they indicate only that one data point is ranked higher or lower than another (Runyon & Haber 1991). Nevertheless, the Kendall's Tau-c test examines the existence of statistical associations between the concordant and discordant pairs in the T^2M. Concordant implies that as the BT RSL bracket increases the AT RSL bracket increases. Conversely, discordant implies that increasing BT RSL bracket has decreasing AT RSL bracket. The tau-c term is calculated using Equation 4.8, where the outcome is between 1 and -1. Tau-c value of 0 implies no association, or independence of the BT and AT rankings. Tau-c value of 1 implies perfect agreement, the BT and AT frequency are the same. Tau-c value of -1 implies perfect disagreement, the AT frequency is the opposite of that BT. Stated differently, positive or negative results that are significant indicate a correlation between the BT and AT RSL bracket

(Bolboaca & Jantschi 2006, SAS 2008). The Kendall's Tau-c test is performed under the null hypothesis of no association (tau-c value of 0) and the p value indicates the significance of the result. Bonferroni correction was applied to account for multiple comparisons (various condition and distress types and the minimum RSL values), such that P values of less than 0.0083 (0.05 familywise error rate, 95% confidence level) provide evidence against the null hypothesis.

$$\mathrm{Tau}_C = (P - Q)\left[\frac{(2m)}{n^2(m-1)}\right] \qquad \text{Equation 4.8}$$

Where, P is the number of concordant pairs;

Q is the number of discordant pairs;

m is the minimum number of rows or columns in the table;

n is the sample size

To this end, results of the analysis of the pavement performance before and after treatment, which are listed in T^2Ms such as those provided in Table 4.20 for single chip seal treatment and the controlling RSL values in Colorado, were subjected to statistical tests using the Kendall's Tau-c test. Results of the statistical tests for thin HMA overlay of asphalt surfaced pavements, single chip seal, and thin mill and fill of asphalt surfaced pavements in Colorado are listed in Table 4.21. The data in the table indicate that the BT RSL bracket is associated with the AT RSL bracket in most cases. For example, for thin HMA overlay of asphalt surfaced pavements, the BT and AT RSL brackets for IRI, rut depth, alligator, and transverse cracking, statistically associated, but are statistically independent for longitudinal cracking. The statistical association implies that the IRI, rut depth, and alligator and transverse cracking were not completely eliminated by the thin HMA overlay; they returned and will reach the threshold values within certain range of time. Hence the AT RSL was deemed a function of the BT RSL.

Table 4.20 T^2M for single chip seal, State of Colorado

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	2329	58		**125**	453	1230	267	254	4	8	9
2	3 to 5	746	18		3	**88**	379	121	155	3	7	11
3	6 to 10	365	9		1	52	**157**	55	100	2	4	12
4	11 to 15	141	3		0	8	52	**27**	54	2	1	14
5	16 to 25	452	11		1	24	128	55	**244**	1	-5	15
Total/average		4033/	100/		130/	625/	1946/	525/	807/	/3	/6	/10

Table 4.21 Results of statistical tests for association of the BT and AT 0.1 mile long pavement segments in the RSL brackets, State of Colorado

Pavement treatment types, the number of 0.1 mile long pavement segments, and the tau-c value		Pavement condition and distress types					
		IRI	Rut depth	Alligator cracking	Longitudinal cracking	Transverse cracking	Controlling RSL
A	Number of segments	557	559	506	384	385	878
	Tau-c value	0.1341	0.2134	0.1132	-0.0048*	0.0999	0.1223
B	Number of segments						
	Tau-c value						
C	Number of segments	2,281	399	2,228	2,440	2,163	4,033
	Tau-c value	0.2425	0.1080	0.2728	0.0550	0.0949	0.2346
D	Number of segments						
	Tau-c value						
E	Number of segments	28	74	49	38	24	83
	Tau-c value	0.2007*	-0.0453*	-0.2666	0.0997*	-0.1157*	0.4870
F	Number of segments						
	Tau-c value						

Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.
* indicates insignificant results

On the other hand, the statistical independence of the longitudinal cracks implies that the measured AT longitudinal cracks were not influenced by the BT longitudinal cracks. That is, for the time span where the AT data were measured and are available for the analysis; the longitudinal cracks did not propagate through the thin HMA overlay yet. This scenario could be reversed over time when more time-series data are collected and become available.

One important point could be deduced from the two results stated above; the rate of propagation of longitudinal cracks on pavement projects subjected to thin HMA overlay is slower than the rate of propagation of alligator and transverse cracks. This makes perfect sense because alligator and transverse cracks in the original asphalt mat are subjected to more movements as the wheel loads cross the cracks than longitudinal cracks, which are mainly parallel to the wheel paths.

Once again, the presence of statistical association between the BT and AT RSL variables implies that the AT longevity of the pavement segment is, in part, a function of the BT RSL. On the other hand, insignificant statistical association implies that the AT longevity of the pavement segment is not a function of the BT RSL. It is affected by other variables such as construction quality. In general, the effectiveness of a given pavement treatment is a function of many variables including the BT RSL. These variables include materials quality, the environmental conditions, construction variables such as layer thicknesses, compaction, particle and temperature segregations, and traffic load and repetitions. Unfortunately, the information regarding these variables are mostly not available in the PMS database and hence all pavement projects were grouped by treatment type (six types were analyzed) and by the pavement conditions and distress types.

Finally, the data in Table 4.21 indicate, for the single chip seal treatment, the BT and AT RSL brackets are statistically associated. This was expected because the single chip seal treatment does not eliminate or even treat the pavement condition or distress types included in the analyses. The treatment is intended to restore friction and/or protect the pavement surface from aging. Hence, all pavement condition and distress types would reflect through the chip seal layer is a short time period. This finding agrees with the expected relationships. On the other hand, the BT and AT RSL brackets for thin mill and fill are statistically associated for some pavement condition and distress types, and independent for others. These results are likely due to two reasons:

1. Small sample size as reflected by the low number of 0.1 mile long pavement segments that were accepted and included in the analysis. Hence, the results may or may not be representative of the results of mill and fill treatment in Colorado.
2. The nature of the mill and fill treatment, which removes the majority of the pavement surface defects such as rut channels and early stages of top-down cracks. The treatment does not completely remove alligator cracks, which are bottom-up cracks or pavement roughness caused by the lower pavement layers.

The inconsistent results regarding the association of the BT and AT RSL brackets could become more consistent if the number of 0.1 mile long pavement segments is increased and hence increasing the sample size. Nevertheless, it is very important to consider the known physical relationships between the treatment type and the treatment benefits before conducting statistical analyses.

3) Pavement Treatment Benefits

The T^2M for the 7 accepted miles of the 9.5 mile long project identified in Table 4.11, for District 1, and for the State of Colorado are listed in Tables 4.11, 4.17, and 4.18 respectively. The three tables also list (in the right three columns) the treatment benefits. It is important to recall that, because of lack of cost data, the analyses are focused on pavement treatment benefits and not treatment cost effectiveness, as discussed in section 4.5. The data in the tables indicate that:

- The TL generally decreases as the BT CS increases. This was expected and is mainly due to the definition of TL (the time between the treatment and when the AT pavement condition and distress reach the BT condition and distress levels). The definition of TL implies that the time required for the AT condition or distress to reach higher BT condition or distress levels is generally longer and depends on the performance jump (PJ) and the AT rate of deterioration.
- The SLE generally decreases as the BT CS increases. This trend was also expected because the algorithm of SLE (SLE is the difference between the AT and BT RSL values). Higher BT RSL values yield lower SLE. For example, the SLE values of two pavement sections having BT RSL of 6 and 10 years and AT RSL of 12 years are 6 and 2 years, respectively. Given the SLE definition, negative SLE implies that the AT RSL is shorter than the BT RSL. This could be caused by negative PJ and/or increasing rate of deterioration. The negative PJ and/or increasing rate of deterioration could be caused by soft HMA, inadequate compaction, or early opening of the pavement to traffic.
- The AT RSL generally increases as the BT RSL increases. This trend was also anticipated because higher BT RSL values generally imply better condition, which remains in better

condition if the overlay is properly constructed. In other words, pavement treatments applied to pavement segments in better condition (higher CS) are typically more effective.

The above observations indicate that each treatment benefit type generally follow similar trend in each of Tables 4.11, 4.17, and 4.18. Note that, T^2Ms for each project within a district, for each district, and for each state, showed similar results to the ones detailed above and are provided on DVD in Appendix B. For convenience, the treatment benefits of thin HMA overlay of asphalt surfaced pavement for all 0.1 mile long pavement segments in the State of Colorado are summarized in Table 4.22. Similar results were also found for single chip seal and for thin mill and fill of asphalt surfaced pavements. The benefits of the latter treatments are summarized in Tables 4.23 and 4.24.

4) Controlling Pavement Condition and Distress Types

The pavement condition and distress types which controlled the RSL values of the 0.1 mile long pavement segments subjected to thin HMA overlay are provided in Figures 4.18 and 4.19. The data in the figures indicate the percent of the 0.1 mile long pavement segments controlled by each pavement condition and distress type BT as well as AT. The data in Figure 4.18 indicate that, in the State of Colorado, thin HMA overlay of asphalt surfaced pavement is almost equally applied to pavements controlled by any of the pavement condition and distress types with the highest percentages from alligator cracking, IRI, and transverse cracking. This implies that, in general, the RSL dictates the selection of thin HMA overlay of asphalt surfaced pavement and that the controlling conditions do not have significant influence in the selection of the treatment. In other words, the thin HMA overlay of asphalt surfaced pavement is selected to address each of the pavement condition and distress types. The data in Figure 4.19 indicate that the alligator cracking controls the AT RSL most often. This is mostly due to the reflection of the

Table 4.22 A summary of treatment benefits for thin HMA overlay of asphalt surfaced pavement in the State of Colorado

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		557			559			506			384			385		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	7	13	14	5	7	8	5	8	9	6	10	11	7	11	12
2	3 to 5	4	7	11	4	6	10	3	5	9	4	7	11	5	10	14
3	6 to 10	4	6	14	4	3	11	3	2	10	3	3	11	4	6	14
4	11 to 15	3	2	15	4	-3	10	3	-1	12	4	-1	12	3	-1	12
5	16 to 25	1	-3	17	4	-5	15	1	-9	11	0	-9	11	0	-5	15
Overall		**4**	**6**	**14**	**4**	**13**	**14**	**3**	**2**	**10**	**4**	**3**	**11**	**5**	**8**	**13**

Table 4.23 A summary of treatment benefits for single chip seal in the State of Colorado

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		2,281			399			2,228			2,440			2,163		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	4	10	11	6	18	19	5	9	10	5	10	11	4	11	12
2	3 to 5	4	12	16	3	10	14	4	8	12	4	7	11	3	9	13
3	6 to 10	3	8	16	2	6	14	3	4	12	4	3	11	3	5	13
4	11 to 15	2	4	17	3	3	16	2	-1	12	4	-1	12	2	0	13
5	16 to 25	1	-2	18	2	-1	19	1	-5	15	2	-7	13	1	-6	14
Overall		**2**	**5**	**16**	**2**	**0**	**18**	**3**	**3**	**12**	**4**	**5**	**11**	**3**	**6**	**13**

Table 4.24 A summary of treatment benefits for thin mill and fill of asphalt surfaced pavement in the State of Colorado

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		28			74			49			38			24		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	8	11	12	8	19	20	6	11	12	5	6	7	8	10	11
2	3 to 5	-	-	-	8	16	20	5	5	9	3	0	4	4	3	7
3	6 to 10	5	4	12	8	12	20	4	-4	4	3	-3	5	2	2	10
4	11 to 15	8	7	20	8	7	20	4	-6	7	3	-8	5	-3	-5	8
5	16 to 25	-3	-3	17	7	-1	19	3	-13	7	3	-12	8	0	-12	8
Overall		**5**	**8**	**14**	**7**	**6**	**20**	**5**	**1**	**9**	**4**	**-3**	**6**	**1**	**-4**	**8**

alligator cracks through the overlay. This was expected as thin HMA overlay typically does not prevent the alligator cracks from propagating through the overlay to the new pavement surface in a short time. The alligator crack is a bottom-up crack, implying stress concentration in the tip of the crack at top of the old pavement surface and bottom of the overlay. Hence, the crack quickly propagates through the thin overlay to the pavement surface. Note that the data in Figures 4.18 and 4.19 are not linked from BT to AT and one must be careful when analyzing the data. For example, the data in the figures indicate that about 6% less 0.1 mile long pavement segments are controlled by IRI AT than BT, but the data do not imply that the treatment has completely corrected roughness for those 6%. The pavement segments could still have high AT roughness but will reach the threshold value of another pavement condition or distress type sooner than for IRI. Further, the distribution of the 0.1 mile long pavement segments whose RSL is controlled by each of the pavement condition and distress types are independent BT and AT. In other words, any 0.1 mile long pavement segment could have a RSL value controlled by one pavement condition or distress type BT and another AT. The data in the figures are only intended to be used to assess the controlling pavement conditions and distresses BT and AT. Similar results were found for the other pavement treatment types, and similar figures are provided in Appendix B.

b) Louisiana DOT & Development (LADOTD)

Similar results to those found in Colorado were found in Louisiana. The standard errors of the estimates of the pavement condition and distress data models for all six of the analyzed pavement treatment types are listed in Table 4.25. The data in the table indicate that the SE of the estimates are lower, relative to the average condition and distresses of the last data collection year BT, for the sensor measured data (IRI and rut depth), and higher for the cracking data. This

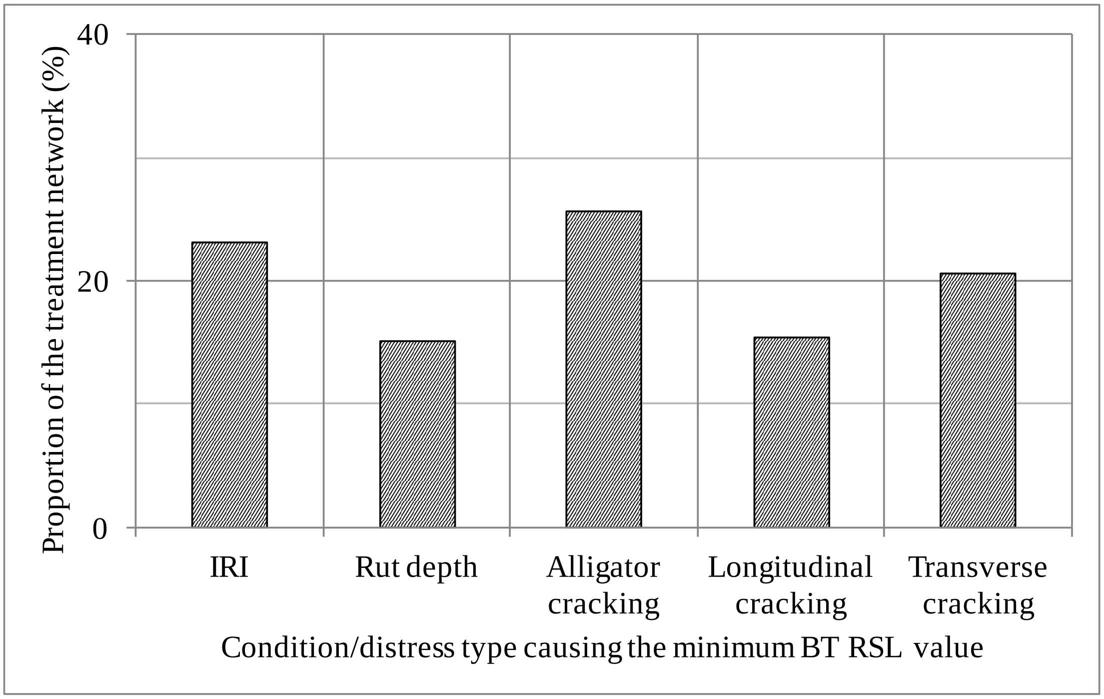


Figure 4.18 The percent of the treatment network having the minimum BT RSL value based on the indicated pavement condition or distress type, for thin HMA overlay of asphalt surfaced pavements in the State of Colorado

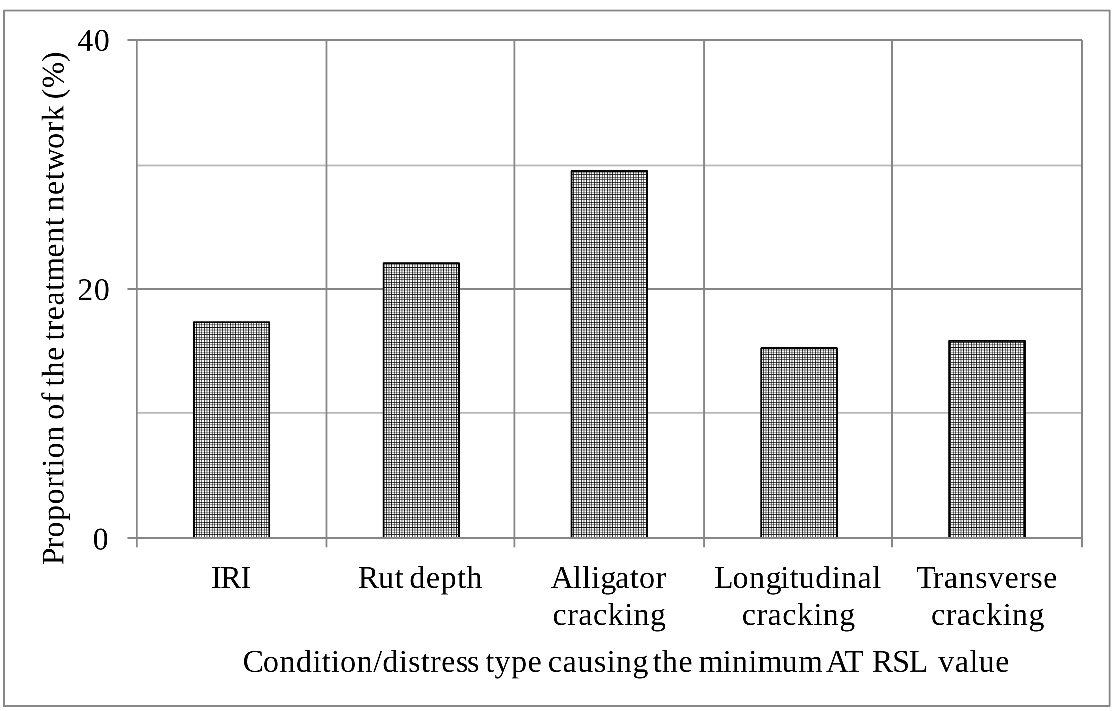


Figure 4.19 The percent of the treatment network having the minimum AT RSL value based on the indicated pavement condition or distress type, for thin HMA overlay of asphalt surfaced pavements in the State of Colorado

is expected, due to the data variability resulting from the subjectivity involved in the process of crack identification and length and severity rating, as discussed in section 3.2 of Chapter 3. The summarized treatment benefits for all six of the analyzed pavement treatment types are listed in Tables 4.26 through 4.31 respectively. Finally, the associations between the BT and AT RSL brackets for each treatment type and each pavement condition and distress type and the controlling RSL are listed in Table 4.32. The results are similar to those found in Colorado, where most scenarios are statistically associated and on few occasions the results are insignificant. The BT 0.1 mile long pavement segment distributions and the pavement treatment benefits in the various districts and the state comparisons are provided in sections 4.5.6 and 4.5.7.

c) Washington State DOT (WSDOT)

Similar results to those found in Colorado and Louisiana were found in Washington. The standard errors of the estimates of the pavement condition and distress data models for each of the analyzed pavement treatment types are listed in Table 4.33. The data in the table indicate that the SE of the estimates for IRI and rut depth are similar to those in Colorado and Louisiana. However, the SE of the estimates for cracking is lower for all pavement distress. This is likely due to the lower levels of BT conditions and distresses in Washington and possibly less variable measured pavement condition and distress data. The summarized treatment benefits for each of the analyzed pavement treatment types are listed in Tables 4.34 through 4.37 respectively. Finally, the associations between the BT and AT RSL brackets for each treatment type and each pavement condition and distress type and the controlling RSL are listed in Table 4.38. The results are similar to those found in Colorado and Louisiana, where most scenarios are statistically associated and on few occasions the results are insignificant. The BT 0.1 mile long

Table 4.25 Model SE of the estimates summary in the State of Louisiana

Treatment type	Pavement condition and distress types and measurement units	Number of 0.1 mile long pavement segments modeled	Average condition or distress of the last data collection year BT	Model standard error of the estimates	
				Before treatment (BT)	After treatment (AT)
A	IRI (in/mi)	219	178	19	5
	RD (in)	224	0.30	0.06	0.03
	AC (ft)	202	230	173	99
	LC (ft)	71	83	349	17
	TC (ft)	134	90	285	44
B	IRI (in/mi)	1,416	206	20	5
	RD (in)	1,242	0.39	0.07	0.03
	AC (ft)	1,199	256	171	39
	LC (ft)	595	97	169	27
	TC (ft)	984	116	280	36
C	IRI (in/mi)	1,089	125	9	12
	RD (in)	574	0.22	0.04	0.03
	AC (ft)	1,605	238	124	95
	LC (ft)	772	106	172	38
	TC (ft)	819	147	319	100
D	IRI (in/mi)	206	142	11	11
	RD (in)	43	0.24	0.02	0.04
	AC (ft)	177	222	102	59
	LC (ft)	61	83	143	16
	TC (ft)	44	106	463	17
E	IRI (in/mi)	163	166	15	6
	RD (in)	191	0.55	0.06	0.03
	AC (ft)	146	128	79	27
	LC (ft)	80	153	107	14
	TC (ft)	135	125	158	38

Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

IRI (in/mi); RD = rut depth (in); AC = alligator cracking (ft); LC = longitudinal cracking (ft); TC = transverse cracking (ft)

Table 4.25 Cont'd

Treatment type	Pavement condition and distress types and measurement units	Number of 0.1 mile long pavement segments modeled	Average condition or distress of the last data collection year BT	Model standard error of the estimates	
				Before treatment (BT)	Before treatment (BT)
F	IRI (in/mi)	735	173	16	5
	RD (in)	957	0.41	0.06	0.02
	AC (ft)	605	294	224	20
	LC (ft)	286	196	211	30
	TC (ft)	396	172	239	21

Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

IRI (in/mi); RD = rut depth (in); AC = alligator cracking (ft); LC = longitudinal cracking (ft); TC = transverse cracking (ft)

pavement segment distributions and the pavement treatment benefits in the various districts and the state comparisons are provided in sections 4.5.6 and 4.5.7.

4.5.5 State-of-the-Practice

The term "state-of-the-practice" is used herein to indicate the collective activities conducted by a given SHA, or semi-autonomous region or district within a SHA, and their outcomes. For a given SHA, the performance of the pavement network under its jurisdiction is, in general, a function of the comprehensiveness, balance, and design of its state-of-the-practice. A comprehensive and balanced state-of-the-practice includes all the activities listed below.

1. The pavement monitoring programs and their components includes the pavement condition and distress survey, the types of pavement condition and distress data to be collected, including the selection of data collection equipment, the data collection frequency, the

Table 4.26 A summary of treatment benefits for thin HMA overlay of asphalt surfaced pavement in the State of Louisiana

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		219			224			202			71			134		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	8	17	18	8	15	16	7	9	10	8	12	13	7	8	9
2	3 to 5	8	15	19	8	16	20	5	5	9	8	11	15	7	8	12
3	6 to 10	8	12	20	8	12	20	5	1	9	7	5	13	6	6	14
4	11 to 15	8	5	18	8	6	19	5	-4	9	7	3	16	5	-1	12
5	16 to 25	8	-1	19	8	0	20	3	-10	10	4	-6	14	6	-6	14
Overall		**8**	**13**	**18**	**8**	**5**	**19**	**6**	**7**	**10**	**7**	**6**	**14**	**7**	**3**	**12**

Table 4.27 A summary of treatment benefits for thick HMA overlay of asphalt surfaced pavement in the State of Louisiana

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		1,416			1,242			1,199			595			984		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	10	17	18	10	18	19	9	13	14	9	13	14	9	11	12
2	3 to 5	10	14	18	10	16	20	9	13	17	9	12	16	8	10	14
3	6 to 10	10	10	18	10	12	20	9	9	17	8	7	15	7	5	13
4	11 to 15	10	6	19	9	6	19	8	2	15	8	4	17	6	0	13
5	16 to 25	10	-1	19	10	0	20	3	-9	11	4	-3	17	3	-6	14
Overall		**10**	**13**	**18**	**10**	**7**	**19**	**9**	**12**	**14**	**7**	**6**	**16**	**7**	**6**	**13**

Table 4.28 A summary of treatment benefits for single chip seal in the State of Louisiana

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		1,089			574			1,605			772			819		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	4	4	5	5	14	15	6	7	8	6	14	15	6	11	12
2	3 to 5	3	8	12	5	14	18	4	9	13	6	11	15	5	10	14
3	6 to 10	3	6	14	5	10	18	3	2	10	6	8	16	4	6	14
4	11 to 15	-2	-1	12	5	7	20	3	-1	12	5	2	15	4	2	15
5	16 to 25	-2	-5	15	3	0	20	0	-7	13	3	-4	16	2	-4	16
Overall		**0**	**-1**	**13**	**4**	**2**	**19**	**5**	**6**	**9**	**5**	**8**	**15**	**5**	**8**	**13**

Table 4.29 A summary of treatment benefits for double chip seal in the State of Louisiana

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		206			43			177			61			44		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	8	13	14	1	0	1	8	11	12	9	17	18	9	15	16
2	3 to 5	3	5	9	5	6	10	7	11	15	9	14	18	9	14	18
3	6 to 10	1	-2	6	3	-4	4	5	5	13	8	10	18	9	12	20
4	11 to 15	0	-2	11	9	7	20	-	-	-	9	7	20	-	-	-
5	16 to 25	-3	-5	15	4	-2	18	-4	-7	14	6	-1	19	8	-2	18
Overall		**1**	**2**	**14**	**4**	**-2**	**16**	**8**	**10**	**13**	**8**	**11**	**18**	**9**	**12**	**17**

Table 4.30 A summary of treatment benefits for thin mill & fill of asphalt surfaced pavement in the State of Louisiana

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		163			191			146			80			135		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	8	15	16	8	18	19	8	18	19	8	17	18	8	10	11
2	3 to 5	8	16	20	8	13	17	7	13	17	8	14	18	8	13	17
3	6 to 10	8	9	17	8	12	20	7	7	15	7	7	15	8	12	20
4	11 to 15	8	6	19	8	7	20	8	7	20	6	3	16	7	1	14
5	16 to 25	7	-1	19	8	0	20	3	-7	13	0	-1	19	7	-2	18
Overall		**8**	**8**	**18**	**8**	**12**	**19**	**6**	**7**	**16**	**6**	**10**	**18**	**8**	**7**	**14**

Table 4.31 A summary of treatment benefits for thick mill & fill of asphalt surfaced pavement in the State of Louisiana

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		735			957			605			286			396		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	10	15	16	10	15	16	10	16	17	9	11	12	10	16	17
2	3 to 5	10	13	17	10	14	18	9	13	17	9	12	16	8	9	13
3	6 to 10	10	9	17	10	11	19	9	10	18	7	6	14	8	7	15
4	11 to 15	9	5	18	8	3	16	9	4	17	8	4	17	9	4	17
5	16 to 25	9	-1	19	10	0	20	2	-12	8	7	-3	17	5	-7	13
Overall		**10**	**10**	**17**	**9**	**7**	**17**	**9**	**14**	**17**	**8**	**6**	**15**	**9**	**10**	**16**

Table 4.32 Results of statistical tests for association of the BT and AT 0.1 mile long pavement segments in the RSL brackets, State of Louisiana

Pavement treatment types, the number of 0.1 mile long pavement segments, and the tau-c value		Pavement condition and distress types					
		IRI	Rut depth	Alligator cracking	Longitudinal cracking	Transverse cracking	Controlling RSL
A	Number of segments	219	224	202	71	134	439
	Tau-c value	0.0479*	0.1440	-0.0053*	0.0831*	0.3153	0.2295
B	Number of segments	1,416	1,242	1,199	595	984	2,279
	Tau-c value	0.0265*	0.0597	0.0262*	0.1605	0.1043	0.1026
C	Number of segments	1,089	574	1,605	772	819	2,131
	Tau-c value	0.2764	0.0433	0.1081	0.0703	0.2002	0.2669
D	Number of segments	206	43	177	61	44	316
	Tau-c value	0.0611*	0.2726	0.0351	0.0766*	0.2433*	0.1280
E	Number of segments	163	191	146	80	135	311
	Tau-c value	0.2012	0.1000	-0.3370	0.0216*	0.3854	0.1164
F	Number of segments	735	957	605	286	396	1,280
	Tau-c value	0.1215	0.2034	-0.0450*	0.1913	-0.0914	0.1384

Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.
* indicates insignificant results

Table 4.33 Model SE of the estimates summary in the State of Washington

<table>
<tr><th rowspan="2">Treatment type</th><th rowspan="2">Pavement condition and distress types and measurement units</th><th rowspan="2">Number of 0.1 mile long pavement segments modeled</th><th rowspan="2">Average BT condition or distress</th><th colspan="2">Model standard error of the estimates</th></tr>
<tr><th>Before treatment (BT)</th><th>After treatment (AT)</th></tr>
<tr><td rowspan="5">A</td><td>IRI (in/mi)</td><td>349</td><td>111</td><td>16</td><td>8</td></tr>
<tr><td>RD (in)</td><td>709</td><td>0.27</td><td>0.06</td><td>0.06</td></tr>
<tr><td>AC (ft)</td><td>1,746</td><td>64</td><td>44</td><td>30</td></tr>
<tr><td>LC (ft)</td><td>1,000</td><td>179</td><td>76</td><td>52</td></tr>
<tr><td>TC (ft)</td><td>1,538</td><td>11</td><td>3</td><td>2</td></tr>
<tr><td rowspan="5">B</td><td>IRI (in/mi)</td><td>10</td><td>119</td><td>11</td><td>14</td></tr>
<tr><td>RD (in)</td><td>122</td><td>0.27</td><td>0.06</td><td>0.06</td></tr>
<tr><td>AC (ft)</td><td>403</td><td>68</td><td>21</td><td>79</td></tr>
<tr><td>LC (ft)</td><td>310</td><td>160</td><td>18</td><td>39</td></tr>
<tr><td>TC (ft)</td><td>220</td><td>17</td><td>3</td><td>1</td></tr>
<tr><td rowspan="5">C</td><td>IRI (in/mi)</td><td>52</td><td>143</td><td>9</td><td>12</td></tr>
<tr><td>RD (in)</td><td>38</td><td>0.13</td><td>0.02</td><td>0.04</td></tr>
<tr><td>AC (ft)</td><td>156</td><td>6</td><td>9</td><td>2</td></tr>
<tr><td>LC (ft)</td><td>111</td><td>28</td><td>17</td><td>8</td></tr>
<tr><td>TC (ft)</td><td>194</td><td>1</td><td>0</td><td>0</td></tr>
<tr><td>D</td><td colspan="5"></td></tr>
<tr><td rowspan="5">E</td><td>IRI (in/mi)</td><td>123</td><td>100</td><td>14</td><td>7</td></tr>
<tr><td>RD (in)</td><td>701</td><td>0.34</td><td>0.09</td><td>0.06</td></tr>
<tr><td>AC (ft)</td><td>886</td><td>42</td><td>41</td><td>44</td></tr>
<tr><td>LC (ft)</td><td>357</td><td>69</td><td>44</td><td>30</td></tr>
<tr><td>TC (ft)</td><td>633</td><td>9</td><td>3</td><td>2</td></tr>
<tr><td>F</td><td colspan="5"></td></tr>
<tr><td colspan="6">Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.

IRI (in/mi); RD = rut depth (in); AC = alligator cracking (ft); LC = longitudinal cracking (ft); TC = transverse cracking (ft)</td></tr>
</table>

Table 4.34 A summary of treatment benefits for thin HMA overlay of asphalt surfaced pavement in the State of Washington

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		349			709			1,746			1,000			1,538		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	8	16	17	8	18	19	8	17	18	8	15	16	8	19	20
2	3 to 5	8	16	20	8	16	20	8	15	19	7	12	16	8	15	19
3	6 to 10	7	12	20	8	12	20	6	7	15	6	7	15	8	11	19
4	11 to 15	6	6	19	8	7	20	8	6	19	7	3	16	7	3	16
5	16 to 25	5	-1	19	7	0	20	5	-3	17	6	-3	17	6	-1	19
Overall		**7**	**9**	**19**	**7**	**6**	**20**	**6**	**5**	**18**	**7**	**4**	**17**	**6**	**2**	**19**

Table 4.35 A summary of treatment benefits for thick HMA overlay of asphalt surfaced pavement in the State of Washington

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		10			122			403			310			220		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	-	-	-	10	18	19	9	13	14	10	18	19	-	-	-
2	3 to 5	10	16	20	9	16	20	10	15	19	10	16	20	10	15	19
3	6 to 10	-	-	-	9	12	20	10	12	20	10	10	18	10	12	20
4	11 to 15	6	7	20	9	7	20	-	-	-	10	7	20	10	7	20
5	16 to 25	9	0	20	8	0	20	9	-3	17	8	-2	18	9	0	20
Overall		**8**	**4**	**20**	**9**	**9**	**20**	**9**	**4**	**16**	**8**	**-1**	**18**	**9**	**2**	**20**

Table 4.36 A summary of treatment benefits for single chip seal in the State of Washington

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		52			38			156			111			194		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	4	8	9	-	-	-	6	17	18	6	16	17	-	-	-
2	3 to 5	4	9	13	-	-	-	6	13	17	6	14	18	6	16	20
3	6 to 10	2	10	18	6	12	20	6	11	19	6	9	17	6	12	20
4	11 to 15	3	7	20	6	7	20	4	3	16	6	7	20	-	-	-
5	16 to 25	-4	-2	18	3	0	20	4	0	20	4	-1	19	4	0	20
Overall		**1**	**5**	**15**	**4**	**1**	**20**	**5**	**1**	**9**	**5**	**5**	**18**	**4**	**0**	**20**

Table 4.37 A summary of treatment benefits for thin mill & fill of asphalt surfaced pavement in the State of Washington

CS or BT RSL bracket number	RSL bracket range (year)	Pavement condition and distress types, the number of 0.1 mile long pavement segments, and the treatment life (TL), service life extension (SLE), and AT RSL for each BT RSL bracket (year)														
		IRI			Rut depth			Alligator cracking			Longitudinal cracking			Transverse cracking		
		123			701			886			357			633		
		TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL	TL	SLE	RSL
1	0 to 2	7	15	16	8	18	19	8	17	18	8	16	17	8	19	20
2	3 to 5	8	15	19	8	15	19	8	15	19	7	12	16	8	15	19
3	6 to 10	8	12	20	7	10	18	8	12	20	7	9	17	7	11	19
4	11 to 15	8	6	19	7	5	18	6	1	14	7	4	17	7	6	19
5	16 to 25	7	-1	19	7	-1	19	4	-3	17	3	-3	17	5	-1	19
Overall		**8**	**7**	**19**	**7**	**8**	**19**	**5**	**1**	**17**	**5**	**3**	**17**	**6**	**1**	**19**

Table 4.38 Results of statistical tests for association of the BT and AT 0.1 mile long pavement segments in the RSL brackets, State of Washington

Pavement treatment types, the number of 0.1 mile long pavement segments, and the tau-c value		Pavement condition and distress types					
		IRI	Rut depth	Alligator cracking	Longitudinal cracking	Transverse cracking	Controlling RSL
A	Number of segments	349	709	1,746	1,000	1,538	2,059
	Tau-c value	0.0800*	0.0031*	-0.0450	0.0526	0.0101*	0.0572
B	Number of segments	10	122	403	310	220	461
	Tau-c value	-	0.0570*	0.2070	-0.0360*	0.0321*	0.1492
C	Number of segments	52	38	156	111	194	203
	Tau-c value	0.4050	-	0.0738*	0.1456	-	0.2496
D	Number of segments						
	Tau-c value						
E	Number of segments	123	701	886	357	633	946
	Tau-c value	0.0746*	0.0357*	-0.0553	0.0249*	0.0111*	-0.0812
F	Number of segments						
	Tau-c value						

Treatment type: A = Thin HMA overlay of asphalt surfaced pavements; B = Thick HMA overlay of asphalt surfaced pavements; C = Single chip seal; D = Double chip seal; E = Thin mill and fill of asphalt surfaced pavements; F = Thick mill and fill of asphalt surfaced pavements.
* indicates insignificant results; - indicates no test was performed due to insufficient population distribution

pavement survey length, the database structure, the location reference system, the quality control procedures, and so forth.

2. The material design and evaluation programs including material selection, asphalt and concrete mix design, the types of laboratory and field testing programs to determine the material physical and engineering characteristics such as gradation, angularity, softness, strength, stiffness, moduli, and so forth.
3. The pavement structural design methodologies including the design of the thicknesses of all pavement layers (subbase, base, HMA and PCC surfaces), overlays, tie bar design, and so forth.
4. Selection of paving materials and their physical and engineering characteristics including strength, stiffness, moduli, angularity, softness, gradation, and so forth. The employed methodologies for the analyses of pavement condition and distress data such as modeling the pavement performance, predicting future pavement conditions and distresses, estimating treatment benefits and cost effectiveness, forensic investigations, determining the causes of pavement distress, and so forth.
5. The project scoping procedures including the selection of project boundaries based on the pavement condition and distress, treatment type based on the distress types and their causes, funding and programming the time of treatment, and so forth.
6. The employed procedures for the analysis and selection of alternative cost-effective pavement network treatment strategies to maximize the longevity of the pavement network.
7. The pavement construction, rehabilitation, preservation, and maintenance practices, which consist of numerous activities including the selection of materials, placement of pavement

layers, compaction of pavement layers, quality assurance/quality control (QA/QC), and the elimination of the causes of distresses.

8. The degree of communication and data sharing between the various disciplines within the organization.

The effects of the states-of-the-practices of the various districts or regions within each of the three SHAs (Colorado, Louisiana, and Washington) and between the SHAs are presented and discussed in the next two subsections. The presentations and discussions are based on the results of the analysis of the time-series pavement condition and distress data obtained from the three SHAs.

4.5.6 Comparison of Intrastate State-of-the-Practice

Each of the three SHAs is divided into several autonomous to semi-autonomous districts or regions. Each district is responsible for portions of the state pavement networks. Some of the likely similarities between the districts within each SHA include the environmental conditions, material sources, and pavement condition data collection (which is typically accomplished by the central office). The differences between the districts include the degree to which the pavement condition and distress data are analyzed and used in their decision making process, the pavement project selection or scoping process, the total road mileage and the percent of this mileage in each road class (Interstate, U.S. route, state road, etc.), and the traffic levels. These differences could result in differences in treatment selection, treatment benefits, and the relative health of the pavement network. It is assumed that for given pavement and treatment types, the differences between the states-of-the-practices of the various districts should be reflected in the selection of project boundaries, treatment type and time, and in the precipitating treatment benefits as presented and summarized in the T^2Ms. Hence, for each district within a SHA, the pavement treatment information and the time-series pavement condition and distress data were grouped and analyzed per treatment type and the results were summarized in T^2Ms. Analyses of the results for thin HMA overlay of asphalt surfaced pavement in the 6 districts in the State of Washington are detailed below. Please note that some pavement treatment types are not included in this analysis because sufficient data from more than one district were not available. Further, due to space constraints, the majority of the data are provided on the accompanying DVD in Appendix B.

Table 4.39 lists the T^2M for one asphalt surfaced pavement section that was subjected in 1999 to 1.4-inch HMA overlay. The 11.3 mile long pavement project is located along SR-9, direction 1, District 1, in the State of Washington. Please note that 10.3 miles of the 11.3 miles (91%) passed the two acceptance criteria (see section 4.4) and were accepted for analysis. Results of the analyses for all pavement sections subjected to thin HMA overlay in this district were used to populate the T^2M presented in Table 4.40. The T^2Ms for thin HMA overlay projects in the other districts are included in Appendix B. For each treatment type and pavement condition and distress type and for each district or region the T^2Ms for the various districts or regions within each of the three SHAs were used to study the similarities and differences in their PMS states-of-the-practices.

As stated earlier (see subsection 4.5.2), for each treatment type and for each 0.1 mile long pavement segment along each project, five BT and five AT RSL values (one value for each of five pavement condition and distress types; IRI, rut depth, and transverse, longitudinal, and alligator cracking) were calculated. All RSL calculations were based on the BT or AT time-series condition and distress data. The results for one pavement project are shown in the T^2M of

Table 4.39 T^2M for 11.3 mile long pavement project subjected to 1.4-inch HMA overlay in 1994, SR-9, direction 1, District 1, Washington

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	9	9		**0**	0	0	2	7	7	18	19
2	3 to 5	16	16		0	**0**	3	3	10	7	13	17
3	6 to 10	30	29		0	0	**10**	6	14	6	7	15
4	11 to 15	4	4		0	0	1	**1**	2	8	3	16
5	16 to 25	44	43		0	0	2	4	**38**	7	-1	19
Total/average		103/	100/		0/	0/	16/	16/	71/	/7	/5	/17

Table 4.40 T^{2}M for thin HMA overlay of asphalt surfaced pavement, District 1, Washington

Condition/distress type: condition/distress causing the minimum RSL												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	42	18		**2**	4	5	5	26	7	15	16
2	3 to 5	24	10		0	**0**	7	4	13	6	12	16
3	6 to 10	41	18		0	0	**10**	6	25	7	8	16
4	11 to 15	6	3		0	0	1	**1**	4	8	4	17
5	16 to 25	116	51		0	0	2	8	**106**	7	-1	20
Total/average		229/	100/		2/	4/	25/	24/	174/	/7	/5	/18

Table 4.39. The data in the BT section of the table lists the number of 0.1 mile long pavement segments and the percent of the project in each RSL bracket based on the lowest of the five BT RSL values. In the AT section of the table, only the numbers of 0.1 mile long pavement segments transitioned from a given BT RSL bracket to AT bracket are listed. Examination of the results listed in Table 4.39 indicates that the BT pavement condition and distress along the 103 accepted 0.1 mile long pavement segments of the 11.3 mile long HMA overlay project are not uniform. Indeed the RSL of 44 pavement segments or 43 percent of the project ranges from 16 to 25 years; whereas the RSL of 9 segments or 9 percent of the project ranges from 0.0 to 2 years. On the other hand, the majority (69%) of the pavement segments were transitioned to AT RSL bracket 5, with the remaining 31% split between RSL brackets 3 and 4. This implies that the thin HMA overlay was successful in reducing the pavement conditions and distresses and/or decreasing the pavement rate of deterioration. Hence, the treatment benefits are large, as listed on the right side of Table 4.39. Note that, for this project, while the average AT RSL is 17 years, the average SLE is only 5 years. This is due to the medium to high BT RSL values which decrease the SLE values. After analyzing each thin HMA overlay pavement project in District 1 (the T^2Ms are included on DVD in Appendix B), the data were compiled and are presented in Table 4.40. The arrangement of the data in Table 4.40 is the same as that in Table 4.39. Further, the distributions of the 0.1 mile long pavement segments in the BT and AT RSL brackets are more or less similar and so are the treatment benefits. This implies that the data for the single project along SR-9 listed in Table 4.39 are representative of the data of all thin asphalt overlay projects in District 1. Similar T^2Ms were produced for each district within the State of Washington and are included on the DVD of Appendix B. The remainder of this subsection will

be focused on comparing the states-of-the-practices between the various districts within the State of Washington.

Once again, the data listed in Table 4.40 for all pavement projects subjected to thin HMA overlays of asphalt surfaced pavement in District 1 of the State of Washington reflect the state-of-the-practice of that district. Similar T^2Ms were developed for the 5 other semi-autonomous districts. For the State of Washington, 71 pavement projects totaling 205.9 miles were subjected to thin HMA overlays. These include 12 projects (22.9 miles) in District 1, 22 projects (67.7 miles) in District 2, 20 projects (52.5 miles) in District 3, 8 projects (41.6 miles) in District 4, 6 projects (16.4 miles) in District 5, and 4 projects (4.8 miles) in District 6.

The data in Figure 4.20 show the cumulative BT distribution of the 0.1 mile long pavement segments selected for thin HMA overlays in each of the CSs in the various districts in the State of Washington. The data in the figure indicate that, except for Districts 1 and 6, the distributions in the BT pavement CSs are similar and close to the state averages. For example, the states-of-the-practices of the districts in project selection indicate that about 20 to 50 percent and overall 42 percent of the selected HMA overlay projects having BT RSL between 0.0 and 2 years. The distribution for the other BT condition states is even more similar, as shown in Figure 4.20. Note that the data collection process is managed at the central office and is therefore very similar between districts. However, the criteria used by the districts to select the treatment type likely differ and have resulted in relatively small differences in BT condition state distributions.

The pavement treatment effectiveness for each district and the average effectiveness in the State of Washington are shown in Figure 4.21. The data in the figure indicate that the TL due to thin HMA overlay in Washington is 7 years, while the districts vary from about 3 to 8 years. The SLE in Washington is 7 years, while the districts vary from about -2 to 10 years. Note that

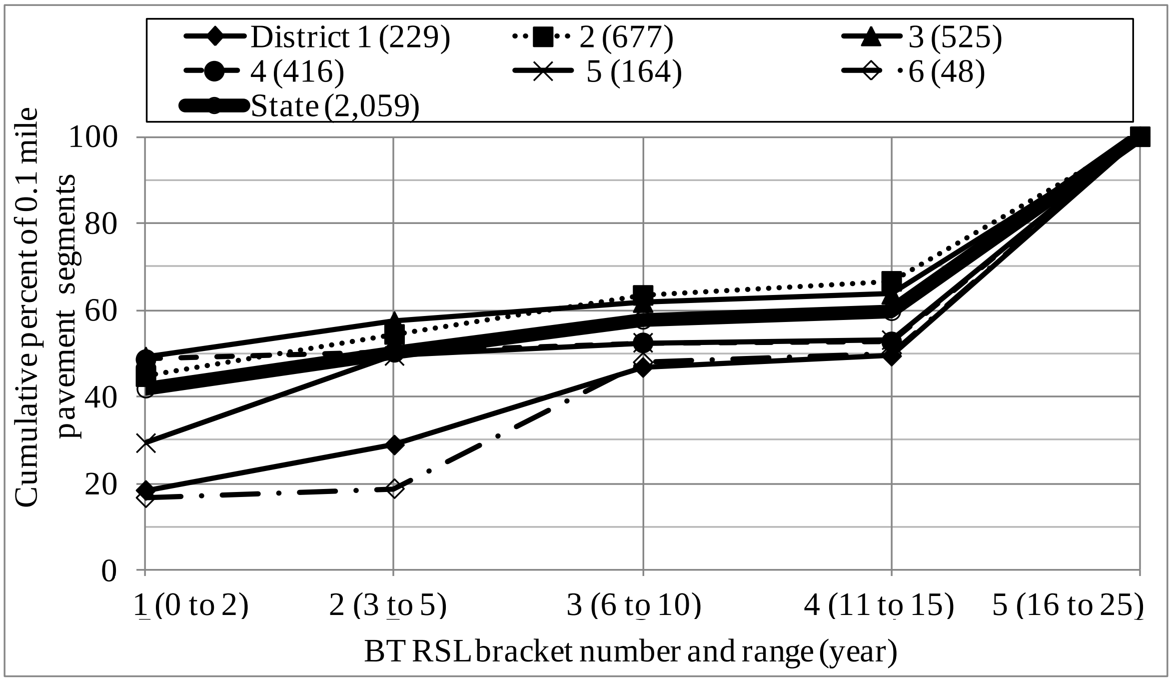


Figure 4.20 BT RSL distributions for thin HMA overlay of asphalt surfaced pavement projects, all districts, State of Washington

the negative SLE value indicates that the AT RSL is less than the BT RSL, and that only 4.8 miles were included for District 6 (where the SLE was negative). The AT RSL in Washington is 17 years, while the districts vary from about 11 to 20 years. The treatment benefits are more or less similar among Districts 1 through 4, while those of Districts 5 and 6 are significantly lower. The sample size in Districts 5 and 6 are less than the other districts and are therefore considered less representative of the district practice. That being said, the results shown in Figure 4.21 indicate that the thin HMA overlay of asphalt surfaced pavement are less effective in Districts 5 and 6 than those of the rest of the state. Although the environmental conditions and material sources are more or less similar throughout the state, some regions have more mountainous terrain, higher traffic counts, and/or higher percentages of urban or rural roadway. For example, the high traffic, urban concentration of the Seattle area is contained in District 2. Hence, it was expected, and is shown, that the pavement does not have the same longevity and the AT RSL is lower in District 2 relative to Districts 1, 3, and 4. These factors, along with other differences in the states-of-the-practices such as the construction contractors and level of quality control, between the districts has lead to these differences in pavement treatment benefits. Similar differences in the treatment benefits for the various districts are shown for thick HMA overlay and thin mill and fill treatments, as shown in Figures 4.22 and 4.23.

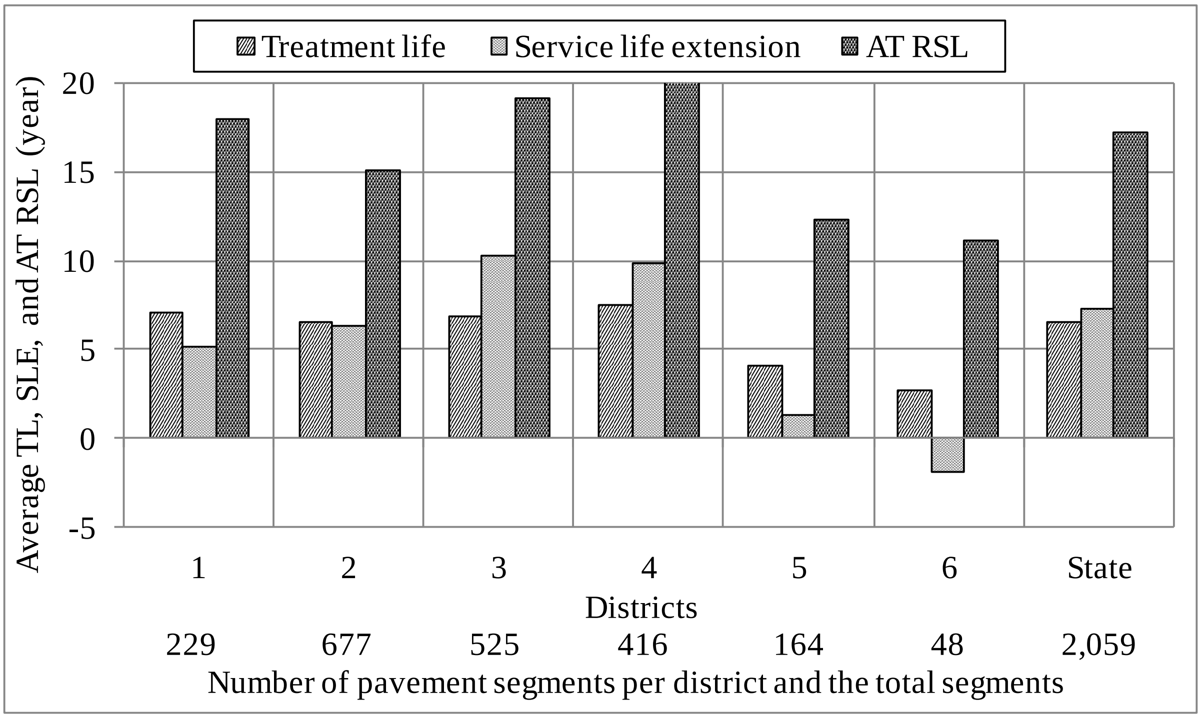


Figure 4.21 Treatment benefits of thin HMA overlay of asphalt surfaced pavement projects, all districts, State of Washington

T^2Ms for treated projects, for each district and for the State of Colorado were produced and are included on DVD in Appendix B. The resulting treatment benefits are discussed herein. The treatment benefits for thin HMA overlay of asphalt surfaced pavement and single chip seal in the State of Colorado are shown in Figures 4.24 and 4.25, respectively. The data in the figures indicate similar differences in the treatment benefits between the districts as those discussed for the State of Washington. In Colorado, the majority of the mountainous terrain is located in

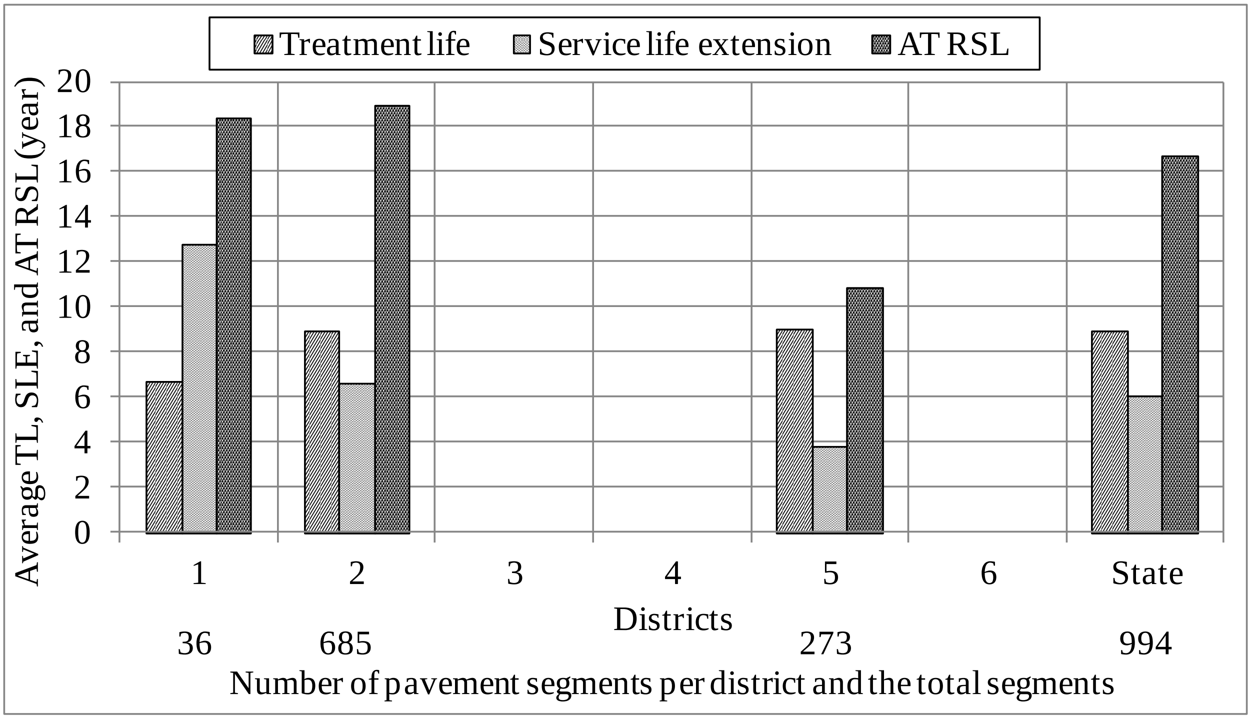


Figure 4.22 Treatment benefits of thick HMA overlay of asphalt surfaced pavement projects, all districts, State of Washington

Figure 4.23 Treatment benefits of thin mill and fill projects, all districts, State of Washington

Districts 3 and 5 with smaller portions of Districts 1, 2, and 4 containing mountainous terrain. On the other hand, the high traffic, urban pavement of the Denver area is all contained within District 6, which is only about 10% the size of the other districts. Hence, it was expected for District 6 to have lower longevity and treatment benefits relative to the other districts. Unfortunately, insignificant data are available for thin HMA overlay treatment in District 6. However, as expected, the treatment benefits for chip seal are all lower in District 6 than the other districts.

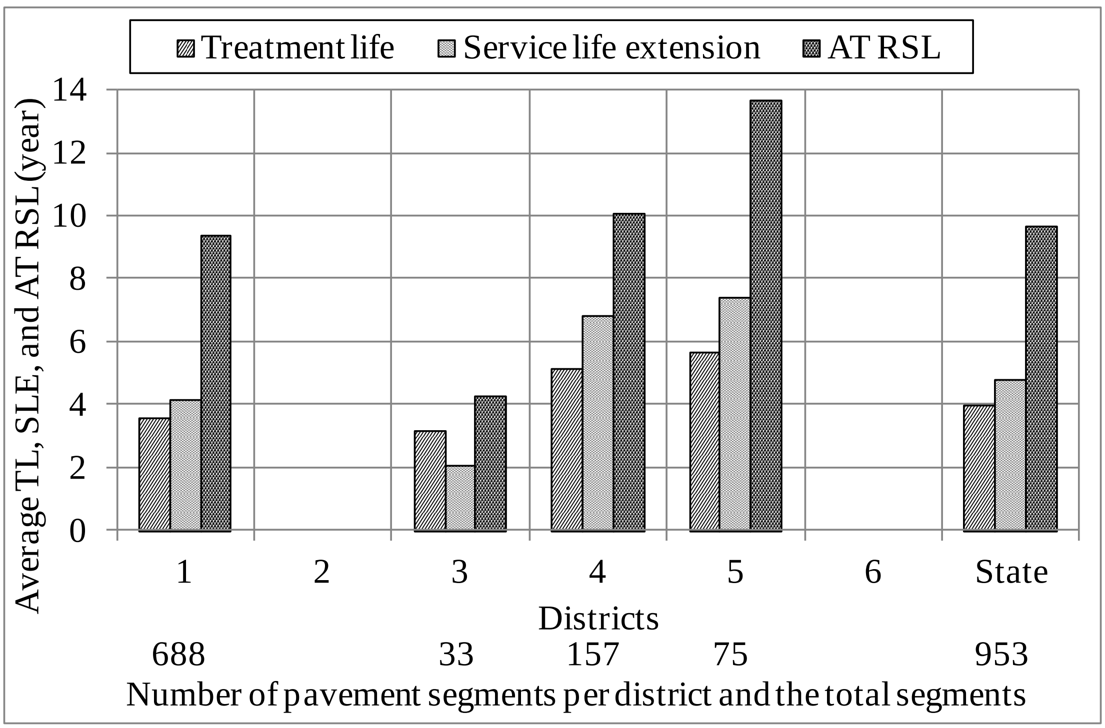


Figure 4.24 Treatment benefits for thin HMA overlay of asphalt surfaced pavement projects, all districts, State of Colorado

Likewise, T^2Ms for treated projects, for each district and for the State of Louisiana were produced and are included on DVD in Appendix B. The resulting treatment benefits are discussed herein. The treatment benefits for thin and thick HMA overlays of asphalt surfaced pavements, single and double chip seal, and thin and thick mill and fill of asphalt surfaced

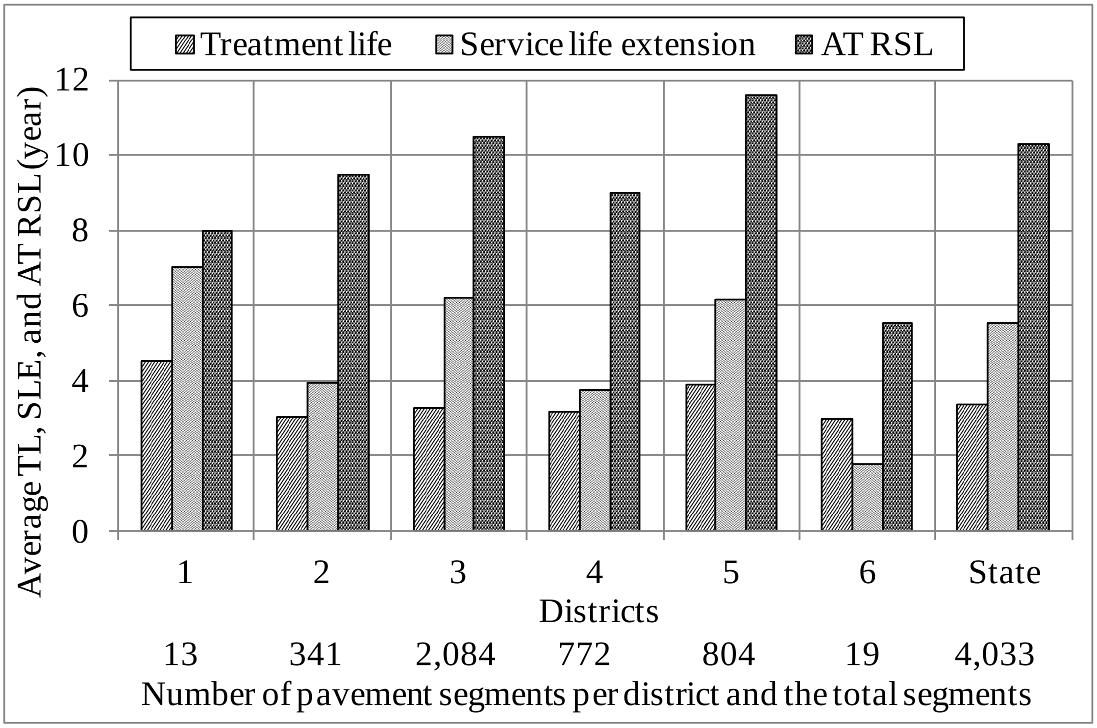


Figure 4.25 Treatment benefits for single chip seal projects, all districts, State of Colorado

pavements in the State of Louisiana are shown in Figures 4.26 through 4.31, respectively. The data in the figures indicate similar differences in the treatment benefits between the districts as those discussed for the States of Washington and Colorado. However, in Louisiana, the majority of the terrain is relatively flat and close to sea level. Likewise, a large portion of the land is near or part of the Mississippi River Delta. Hence, a large portion of the pavement structures in Louisiana are on bridges or fill areas surrounded by low lying lands. Therefore, the differences in the treatment benefits between the districts are mainly attributed to the criteria used by the Louisiana districts to select the treatment type. These include improving ride quality, skid resistance, and/or structural capacity, eliminating rutting, and/or retarding distress propagation. In addition, the various districts evaluate the pavement conditions using visual survey, the PMS condition and distress data, composite pavement index, destructive and nondestructive testing, condition indices, and/or the RSL (Khattak 2011). These factors plus the differences in construction practices and quality control, have lead to the differences in the treatment benefits among the various districts.

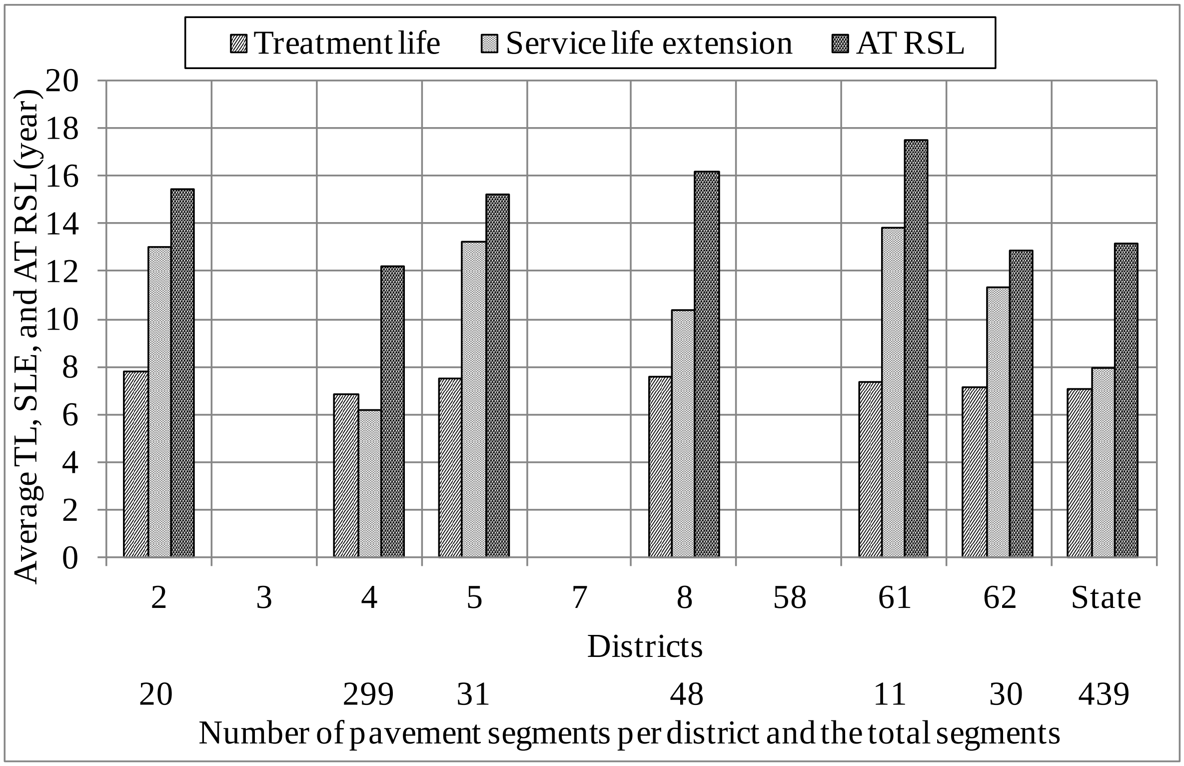


Figure 4.26 Treatment benefits for thin HMA overlay of asphalt surfaced pavement projects, all districts, State of Louisiana

4.5.7 Comparison of Interstate State-of-the-Practice

The differences between the three SHAs (Colorado, Louisiana, and Washington) include terrains, environmental conditions, material sources, and pavement condition and distress data collection practices. Further, the SHAs differ in the degree to which the pavement condition and distress data are analyzed and used in their decision making process, the pavement project selection or scoping process, the total road mileage and the percent of this mileage in each road class (Interstate, U.S. route, state road, etc.), and the traffic levels. These differences could result in differences in treatment selection, treatment benefits, and the relative health of the pavement network. It is assumed that for a given pavement and treatment type, the differences between the

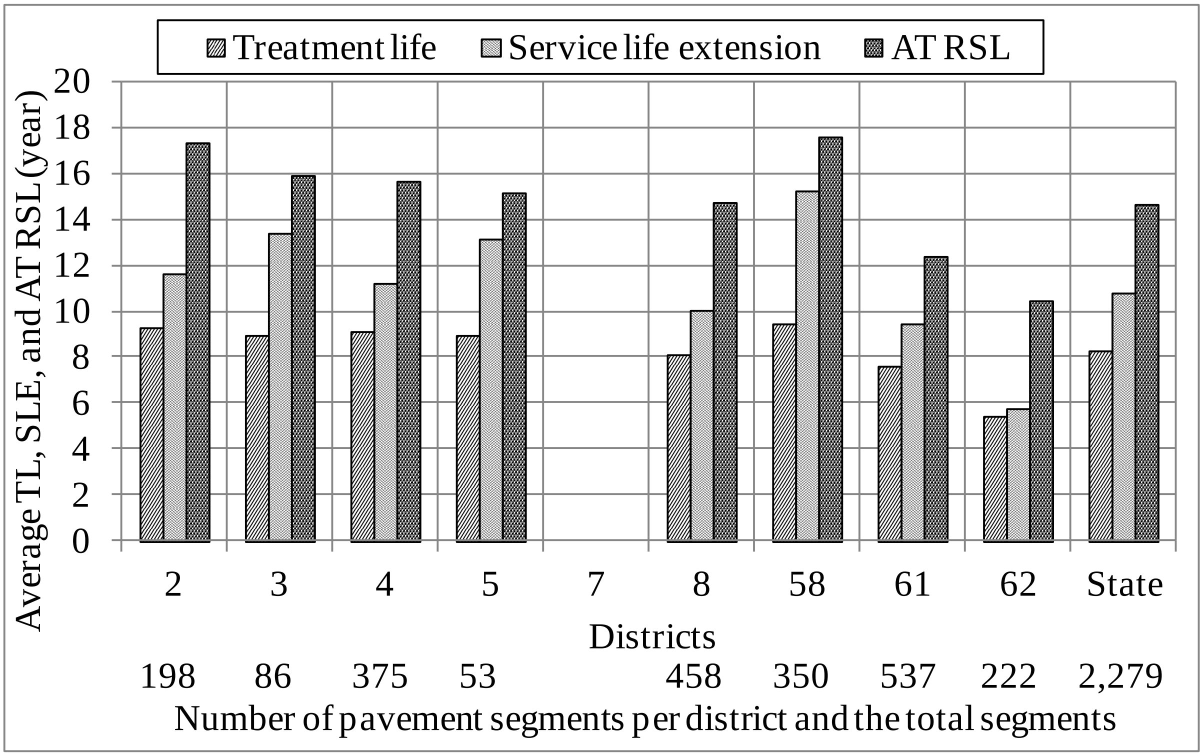


Figure 4.27 Treatment benefits for thick HMA overlay of asphalt surfaced pavement projects, all districts, State of Louisiana

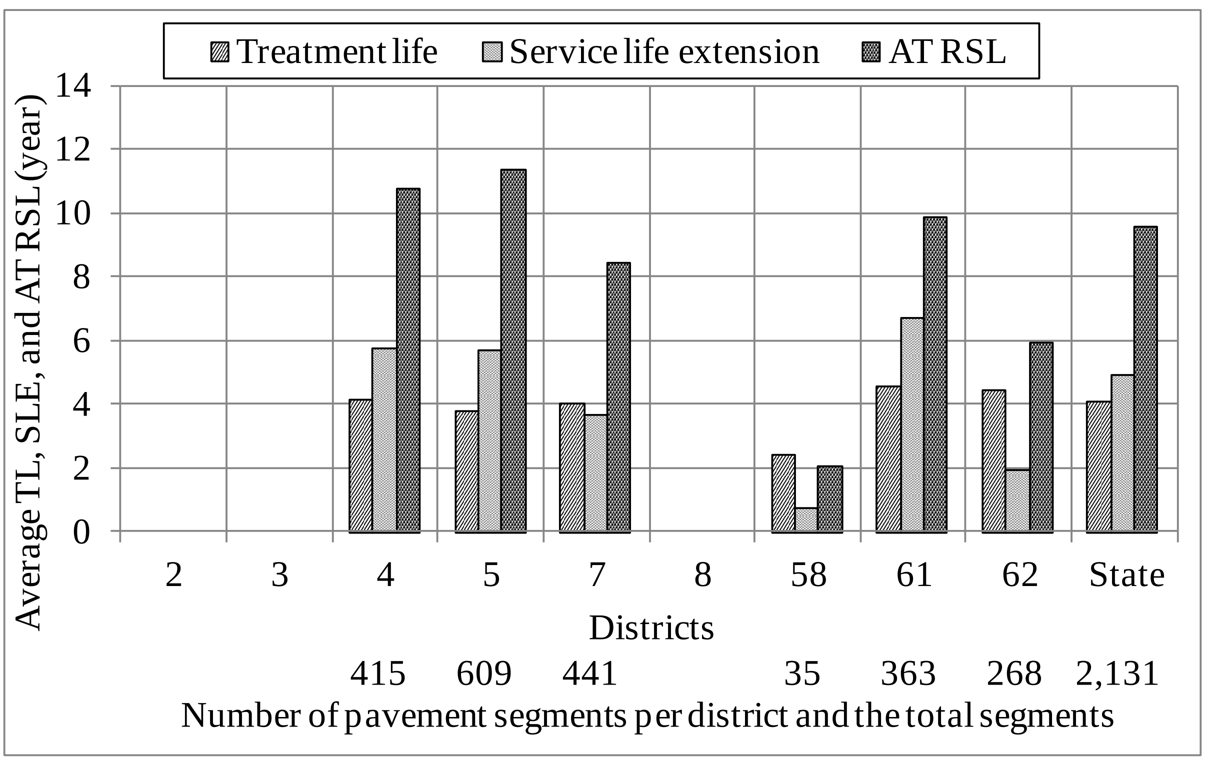


Figure 4.28 Treatment benefits for single chip seal projects, all districts, State of Louisiana

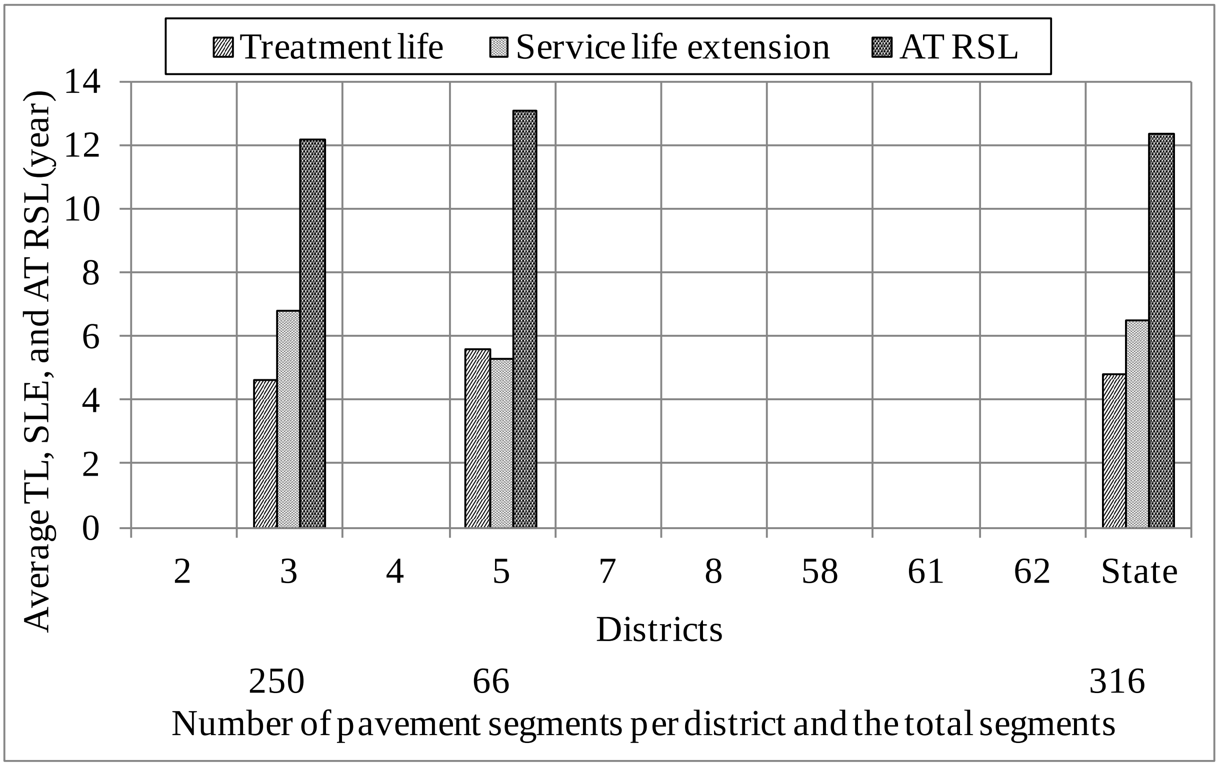


Figure 4.29 Treatment benefits for double chip seal projects, all districts, State of Louisiana

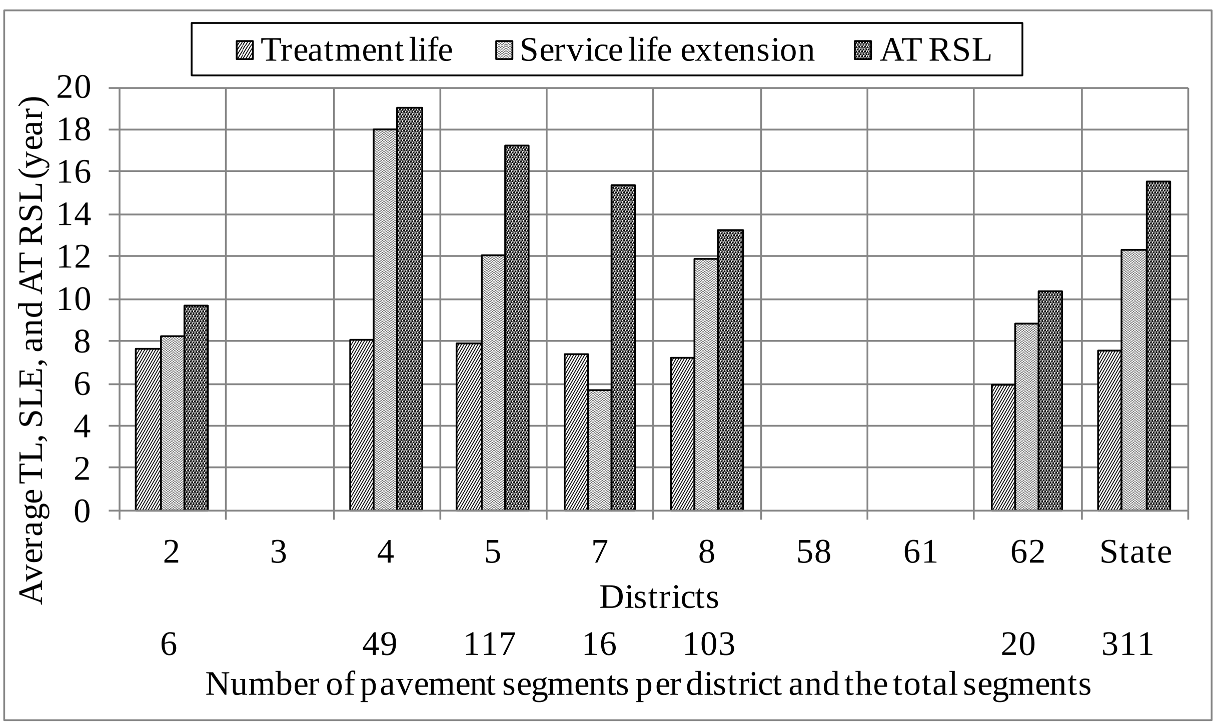


Figure 4.30 Treatment benefits for thin mill and fill projects, all districts, State of Louisiana

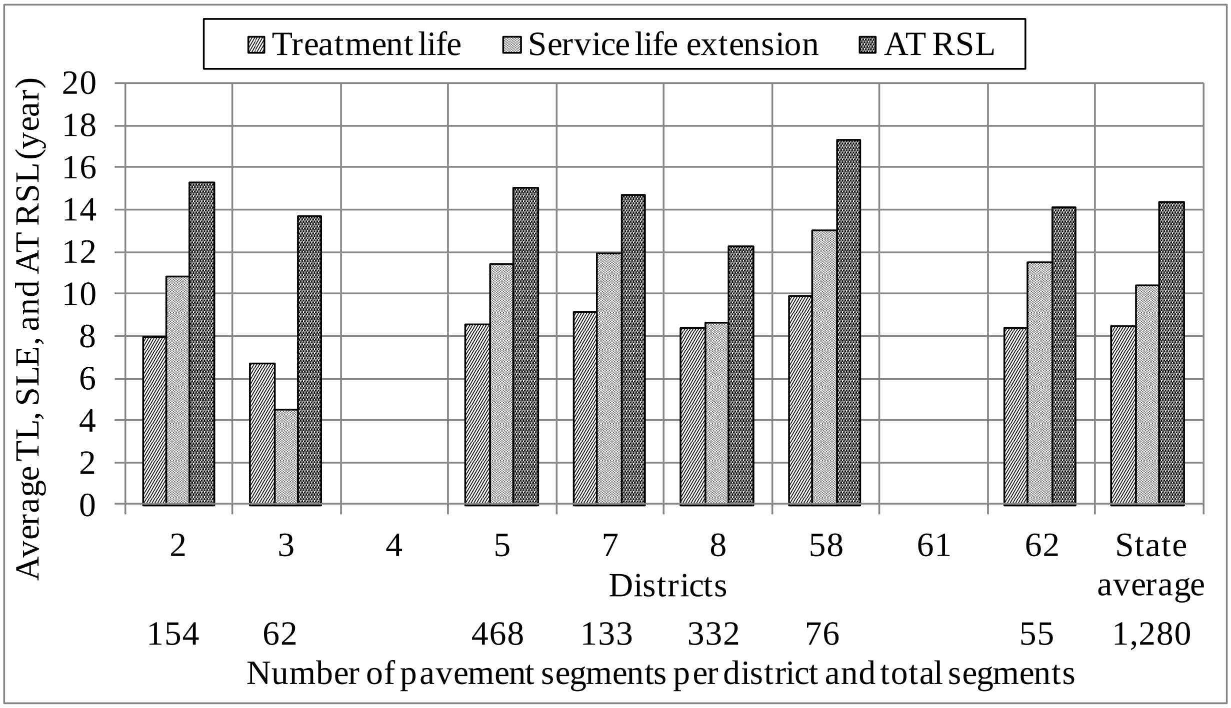


Figure 4.31 Treatment benefits for thick mill and fill projects, all districts, State of Louisiana

states-of-the-practices of the three states should be reflected in the results presented in the T^2Ms. Hence, the pavement treatment information and the time-series pavement condition and distress data were grouped and analyzed per treatment type in each state and the T^2Ms were populated. Due to space constraints, the majority of the data are provided on the accompanying DVD in Appendix B. Analyses of the results for thin HMA overlay of asphalt surfaced pavement in the three states are detailed below. Please note that some pavement treatment types are not included in this analysis because sufficient data from more than one state were not available.

Table 4.41 lists the T^2M for one asphalt surfaced pavement section that was subjected in 2001 to 2-inch HMA overlay. The 7.4 mile long pavement project is located along LA-9, control section 043-06-1, direction 1, District 4, in the State of Louisiana. Please note that 7.2 miles of the 7.4 miles (97%) passed the two acceptance criteria and were accepted for analysis. Results of the analyses for all pavement sections subjected to thin HMA overlay in this district were used to populate the T^2M in Table 4.42. Finally, results of the analyses for all pavement sections subjected to thin HMA overlay in this state were used to populate the T^2M in Table 4.43. The T^2Ms for thin HMA overlay projects in the other states are included on the DVD in Appendix B. For each treatment type and pavement condition and distress type and for each state the T^2Ms for each of the three SHAs were used to study the similarities and differences in their states-of-the-practices. The 146 thin HMA overlay of asphalt surfaced pavement projects totaling 330.7 miles accepted for analyses consist of 57 projects (87.7 miles) in Colorado, 18 projects (37 miles) in Louisiana, and 71 projects (205.9 miles) in Washington.

As stated earlier (see subsection 4.5.2), for each treatment type and for each 0.1 mile long pavement segment along the project, five RSL values were calculated based on the BT time-series condition and distress data. One RSL value was calculated for each of five pavement condition and distress types (IRI, rut depth, and transverse, longitudinal, and alligator cracking). The same procedure was used to calculate five RSL values using the AT time-series data. The results for one pavement project are shown in Table 4.41. The data in the BT section of the table lists the number of 0.1 mile long pavement segments and the percent of the project in each RSL bracket based on the lowest of the five RSL values. In the AT section of the table, only the numbers of 0.1 mile long pavement segments transitioned from a given BT RSL bracket to AT bracket are listed. Examination of the results listed in Table 4.41 indicates that the BT pavement condition and distress along the 72 accepted pavement segments of the 7.4 mile long HMA overlay project are more or less uniform. Indeed the RSL of 64 pavement segments or 89 percent of the project ranges from 0.0 to 2 years. On the other hand, the majority (62%) of the pavement

Table 4.41 T^{2}M for 7.4 mile long project subjected to 2-inch HMA overlay in 2001, LA-9, control section 043-06-1, direction 1, District 4, Louisiana

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	64	89		**1**	24	33	4	2	6	6	7
2	3 to 5	3	4		0	**1**	1	0	1	7	7	11
3	6 to 10	3	4		0	0	**2**	0	1	6	4	12
4	11 to 15	1	1		0	1	0	**0**	0	2	-9	4
5	16 to 25	1	1		0	0	0	0	**1**	10	0	21
Total/average		72/	100/		1/	26/	36/	4/	5/	/6	/6	/8

Table 4.42 T^2M for thin HMA overlay of asphalt surfaced pavement, District 4, Louisiana

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	190	64		**1**	57	83	17	32	7	8	9
2	3 to 5	18	6		0	**3**	3	1	11	7	11	15
3	6 to 10	19	6		0	1	**8**	3	7	6	5	13
4	11 to 15	9	3		0	1	0	**2**	6	7	4	17
5	16 to 25	63	21		0	0	5	3	**55**	8	-1	19
Total/average		299/	100/		1/	62/	99/	26/	111/	/7	/6	/12

Table 4.43 T^{2}M for thin HMA overlay of asphalt surfaced pavement, State of Louisiana

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each CS or BT RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	306	70		**2**	67	105	33	99	7	11	12
2	3 to 5	22	5		0	**3**	5	1	13	7	11	15
3	6 to 10	22	5		0	1	**8**	3	10	6	6	14
4	11 to 15	12	3		0	3	0	**2**	7	7	2	15
5	16 to 25	77	18		0	0	10	7	**60**	8	-2	18
Total/average		439/	100/		2/	74/	128/	46/	189/	/7	/8	/13

segments were transitioned to AT RSL brackets 2 and 3, with the remaining 38% split between RSL brackets 1, 4, and 5. This implies that the thin HMA overlay was relatively successful in reducing the pavement conditions and distresses and/or decreasing the pavement rate of deterioration. Hence, the treatment benefits are generally between about 5 and 10 years, as listed on the right hand side of Table 4.41.

After analyzing all thin HMA overlay pavement projects in District 4, the data were compiled and are presented in Table 4.42. The BT data in Table 4.42 are similar to the data in Table 4.41, with some higher percentage of pavement segments in BT RSL bracket 5. However, the higher AT RSL brackets are more highly populated and hence, the treatment benefits are higher. This implies that the data for the single project along SR-9 is representative of District 1 in project boundary selection (BT RSL) but the treatment was less effective. Further, after analyzing all thin HMA overlay pavement projects in the State of Louisiana, the data were compiled and are presented in Table 4.43. The data in Table 4.42 are similar to that in Table 4.43 and indicate that District 4 is representative of the entire state. The remainder of this subsection will be focused on comparing the states-of-the-practices between the three states.

The state-of-the-practice in the selection of pavement project boundaries has a significant impact on the effectiveness of the pavement treatments. Treatments applied to pavement sections in worse condition typically require more pre-treatment repairs (milling, patching, sealing, etc.) and therefore require higher costs to achieve similar service lives as pavement sections in better conditions. The data in Table 4.44 show the cumulative distribution of the BT condition states of the pavement sections subjected to thin HMA overlay of asphalt surfaced pavement in the States of Colorado, Louisiana, and Washington. The data in the table indicate that the distributions of the BT condition states between Colorado and Louisiana are similar but are dissimilar from that of Washington. In Colorado and Louisiana, a large portion (about 62 to 66%) of the selected projects is in BT condition state 1 and only about 11 to 20% are from BT condition state 5. While in Washington about 42% of the selected projects are in BT condition state 1 and about 40% in BT condition state 5. The distributions of the other BT condition states are more similar. This implies that thin HMA overlay of asphalt surfaced pavement is applied to pavements with lower RSL values in the States of Colorado and Louisiana (worse conditions) than in the State of Washington. The BT RSL distributions for all treatment types for all states are included in Table 4.44.

The data in Figure 4.32 show the treatment benefits (TL, SLE, and AT RSL) for thin HMA overlay in the States of Colorado, Louisiana, and Washington. The data in the figure indicate that the TL due to thin HMA overlay in Colorado is about 4 years, while in Louisiana and Washington it is about 7 years. The SLE in Colorado is about 5 years, while in Louisiana and Washington the SLE is about 8 and 7 years, respectively. The AT RSL in Colorado, Louisiana, and Washington are about 9, 14, and 17 years, respectively. Each of the TL, SLE, and AT RSL are the lowest in Colorado and are more or less similar between Louisiana and Washington. Although Washington has the highest longevity (AT RSL of 17 years), the SLE is less than that of Louisiana because Washington applies thin HMA overlay of asphalt surfaced pavement to pavements with higher BT RSL.

As it was expected, the treatment benefits in the State of Washington are higher due likely to the application of thin HMA overlay treatment to pavement sections in better conditions. Although the BT distributions of the 0.1 mile long pavement segments in Colorado and Louisiana are similar, the treatment benefits are higher in Louisiana. This is likely due to the differences in the environments between the two states. Colorado is mountainous and undergoes

Table 4.44 BT CS distributions for various treatments in three SHAs

State	CS or RSL bracket number	RSL bracket range (year)	Treatment type and the percent of 0.1 mile long pavement segments in each RSL bracket						
			HMA overlay (< 2.5”)	HMA overlay (≥ 2.5”)	Single chip seal	Double chip seal	Mill and fill (< 2.5”)	Mill and fill (≥ 2.5”)	Overall
Colorado	1	0 to 2	62	-	58	-	43	-	58
	2	3 to 5	14	-	18	-	14	-	18
	3	6 to 10	8	-	9	-	8	-	9
	4	11 to 15	4	-	3	-	2	-	4
	5	16 to 25	12	-	11	-	31	-	12
Louisiana	1	0 to 2	70	75	71	66	81	74	73
	2	3 to 5	5	7	7	6	3	6	6
	3	6 to 10	5	5	5	5	3	6	5
	4	11 to 15	3	6	4	3	7	7	5
	5	16 to 25	18	8	14	21	5	8	11
Washington	1	0 to 2	42	36	18	-	44	-	40
	2	3 to 5	8	10	10	-	10	-	9
	3	6 to 10	7	3	10	-	9	-	7
	4	11 to 15	2	1	3	-	4	-	3
	5	16 to 25	40	49	58	-	33	-	41

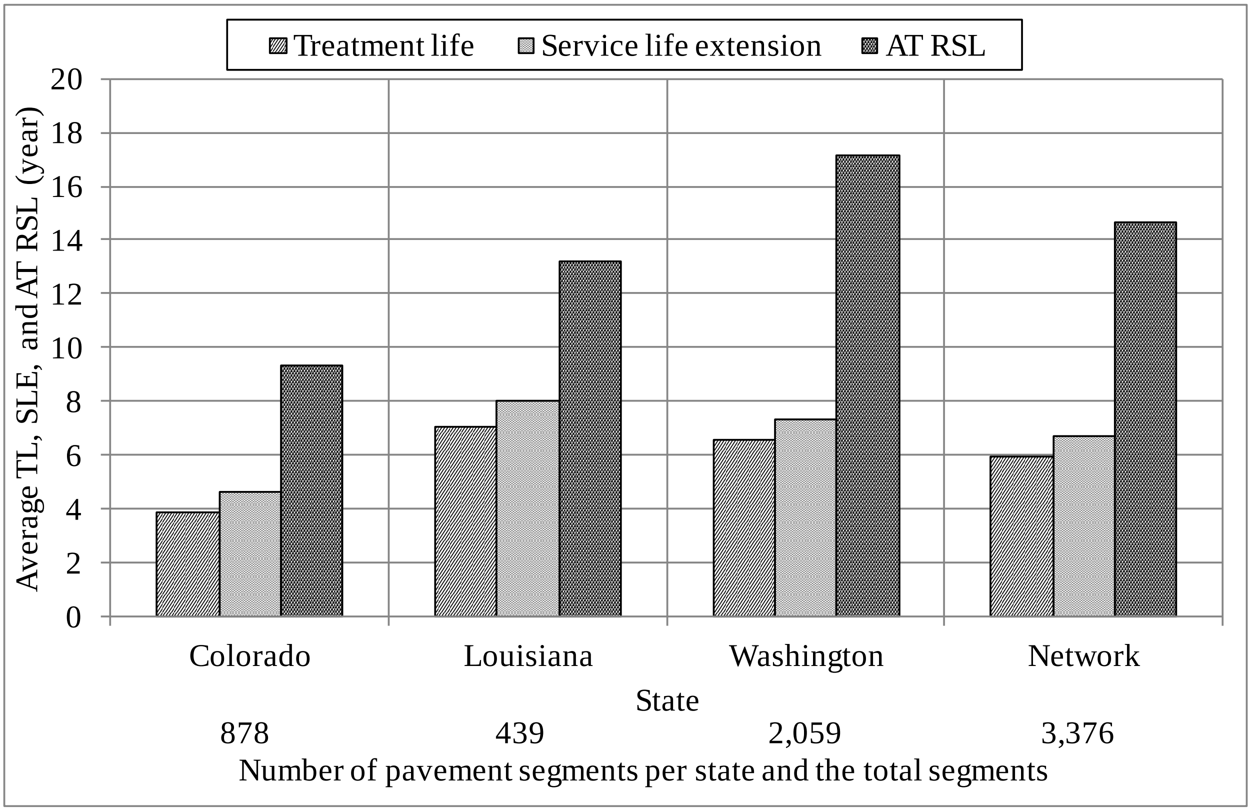


Figure 4.32 Thin HMA overlay of asphalt surfaced pavement benefits in three states

many freeze-thaw cycles annually, while Louisiana is relatively flat with no freeze-thaw cycles. Nevertheless, the similarities and dissimilarities between the states-of-the-practices, which have yielded the results in Figure 4.32, are likely due to many factors including:

- The climate - Colorado undergoes a wide range of climatic conditions including freeze-thaw cycles, Louisiana is not subjected to freeze-thaw cycles but has a lot of moisture, and Washington is somewhere in between with some freeze-thaw cycles and high precipitation.
- The terrain – Colorado and Washington have mountainous terrain while Louisiana is more flat with many low, wet areas along the Mississippi River Delta.
- The annual budget – Colorado spends about \$1.25 billion annually and controls 22,912 lane-miles (about \$54,557/lane-mile), Louisiana spends about \$1.92 billion annually and controls

38,458 lane-miles (about ($49,925/lane-mile), and Washington spends about $0.95 billion annually and controls 18,392 lane-miles ($51,653/lane-mile) (Hartegen et al. 2009).

- The PMS policy – The policies regarding the collection and use of the PMS data very between the states. Washington collects data annually while Colorado and Louisiana collect data more or less every other year.
- The use of PMS data in the selection of project boundaries and pavement treatment type are required (legislated) in Washington; it is optional in the States of Colorado and Louisiana.

Due to space constraints, the remaining analysis results are provided on the accompanying DVD in Appendix B, and are summarized in Figures 4.33 through 4.35. The differences in the treatment benefits between the states, shown in the figures, can be attributed to the same factors detailed above which constitute the states-of-the-practices. Hence, similar relationships were found between the states and the treatment benefits.

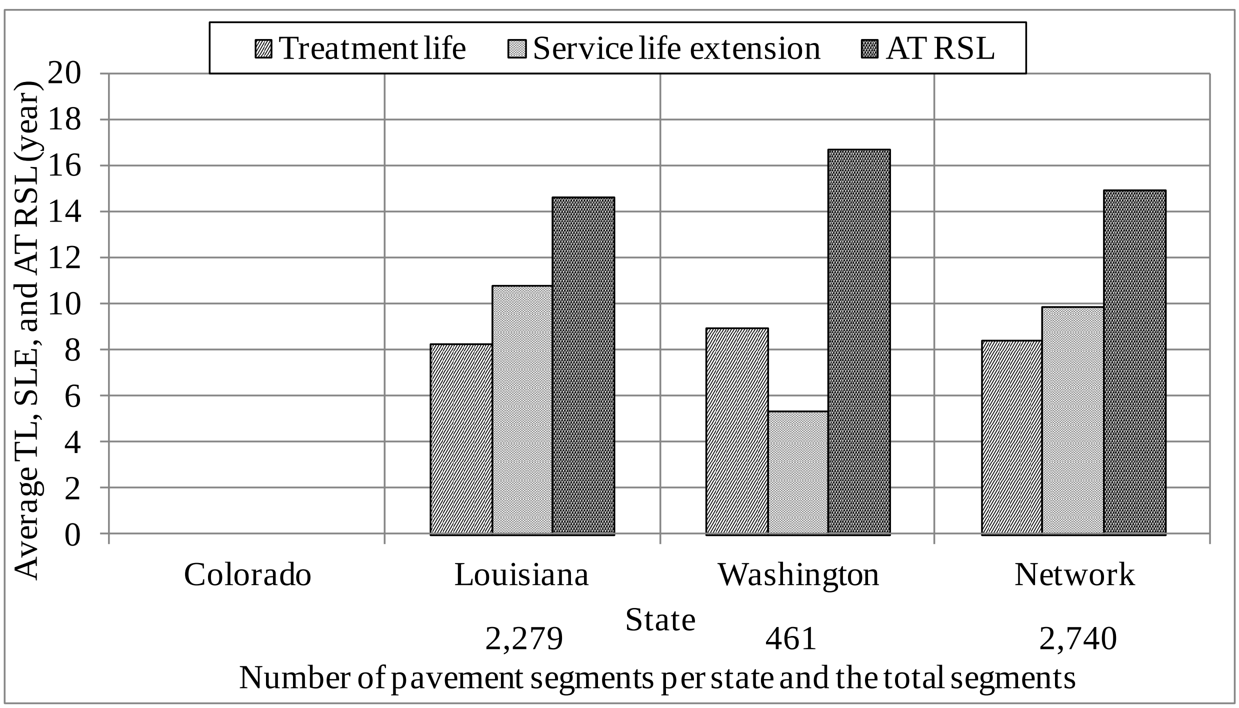


Figure 4.33 Thick HMA overlay of asphalt surfaced pavement benefits in three states

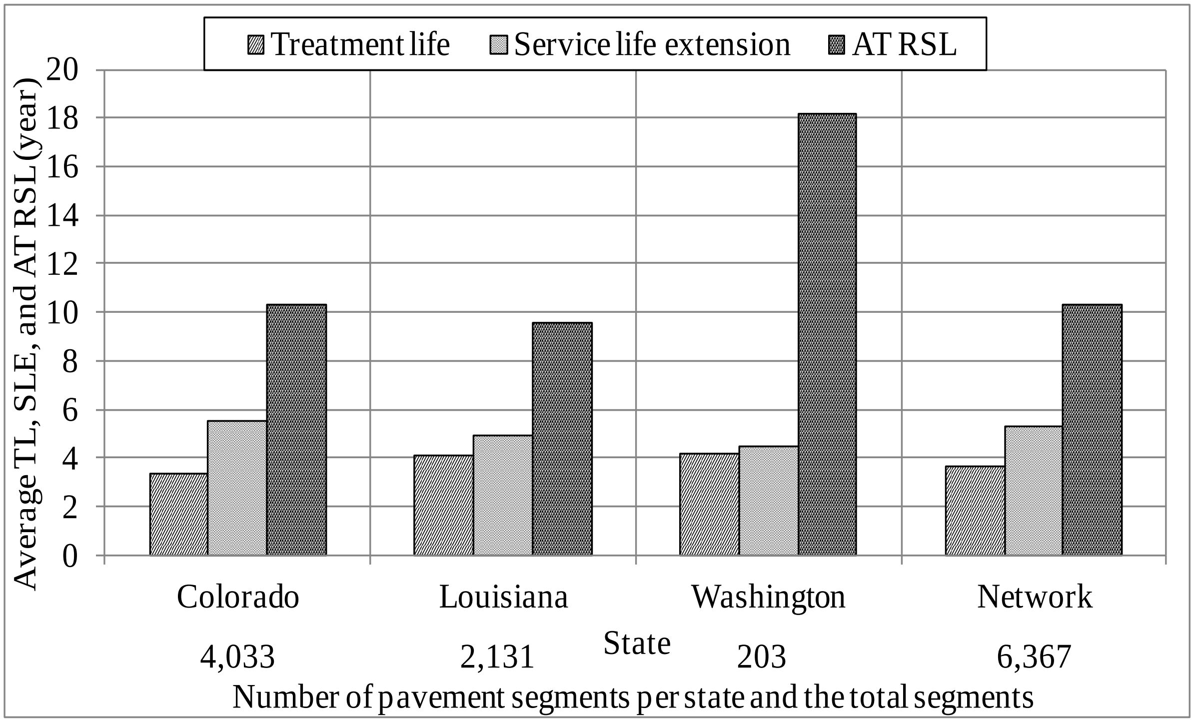


Figure 4.34 Single chip seal benefits in three states

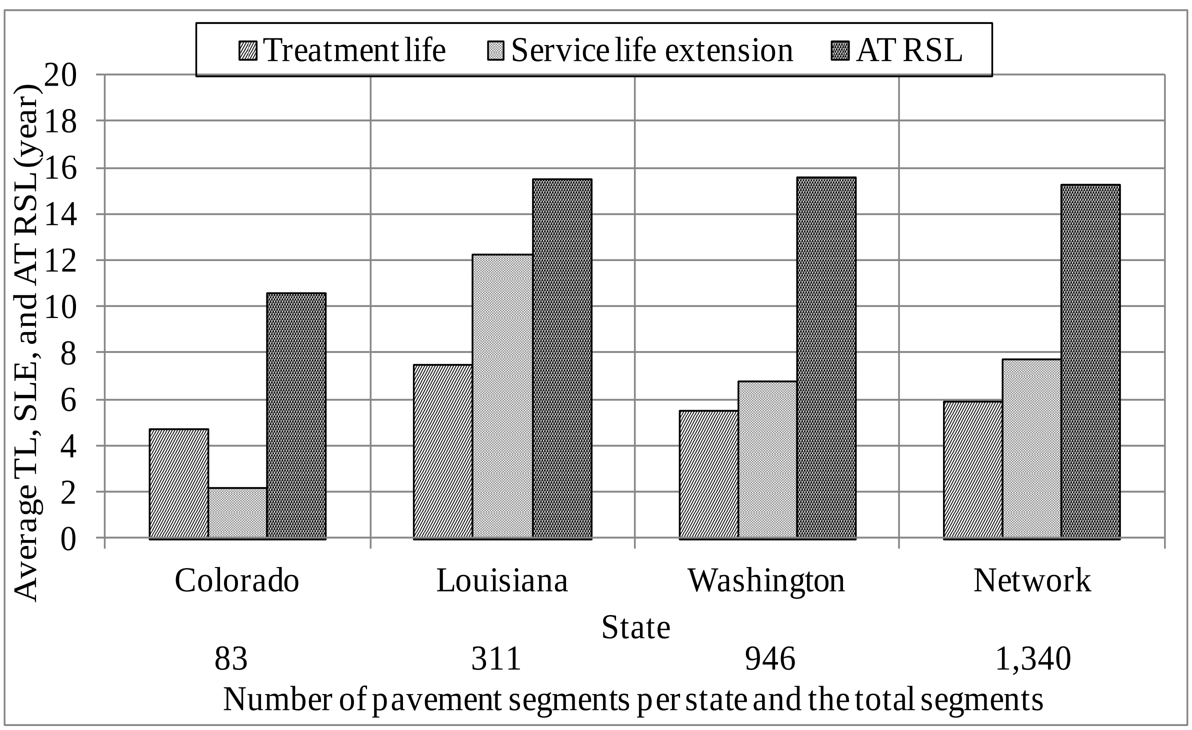


Figure 4.35 Thin mill and fill of asphalt surfaced pavement benefits in three states

Finally, the data in the T^2Ms could be used to investigate the treatment type selection by studying the BT controlling condition and distress types. The BT controlling condition and distress types, for all treatment types, are shown in Figures 4.36 through 4.38 for the States of Colorado, Louisiana, and Washington, respectively. The data in the figures indicate that:

- Alligator cracking is the most common controlling pavement distress for most treatments in the States of Louisiana and Washington, while no general pattern for the controlling condition and distress was found in the State of Colorado.
- The data for each state indicate that, as it was expected, chip seal treatment is rarely applied to pavements controlled by rut depth. Chip seal treatment does not provide much benefit to pavements having rutting problems.
- Mill and fill treatment of asphalt surfaced pavements is most commonly applied to pavement controlled by rutting in all three states. This finding was expected because mill and fill is a very effective treatment to remove the rut channels and restore the pavement structure.

The above observations imply that these pavement treatment types were properly selected. Nevertheless, alternative T^2Ms were made based on the actual condition and distress values without the need to predict the BT and AT RSL values. These T^2Ms are presented and discussed in the next subsection.

4.5.8 Alternative T^2Ms

Some Highway Authorities (HAs) have the necessary data to model pavement conditions and distresses before and after treatment and to populate T^2Ms based on BT and AT RSL values. However, for some reasons (such as lack of the required time-series data, the data are collected at large time interval such as every five years, technical capability, and so forth) some HAs cannot

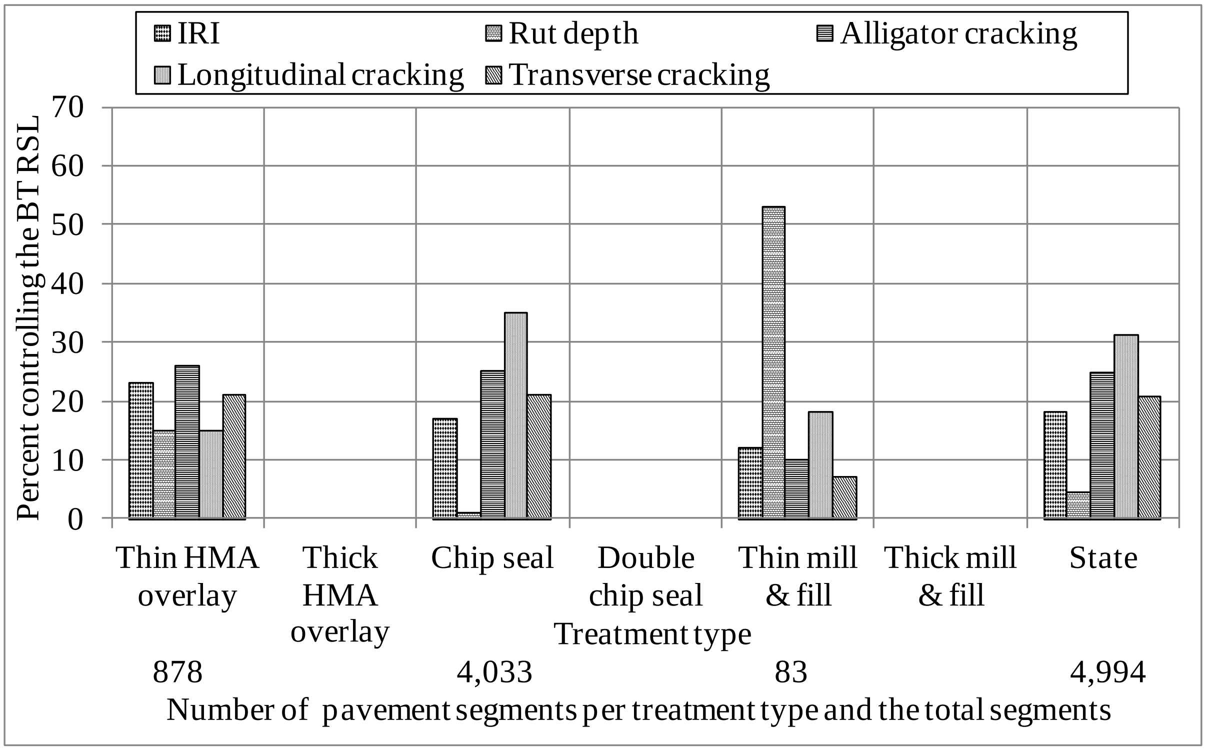


Figure 4.36 Pavement treatment types and the percent of pavement condition and distress types causing the minimum RSL value in the State of Colorado

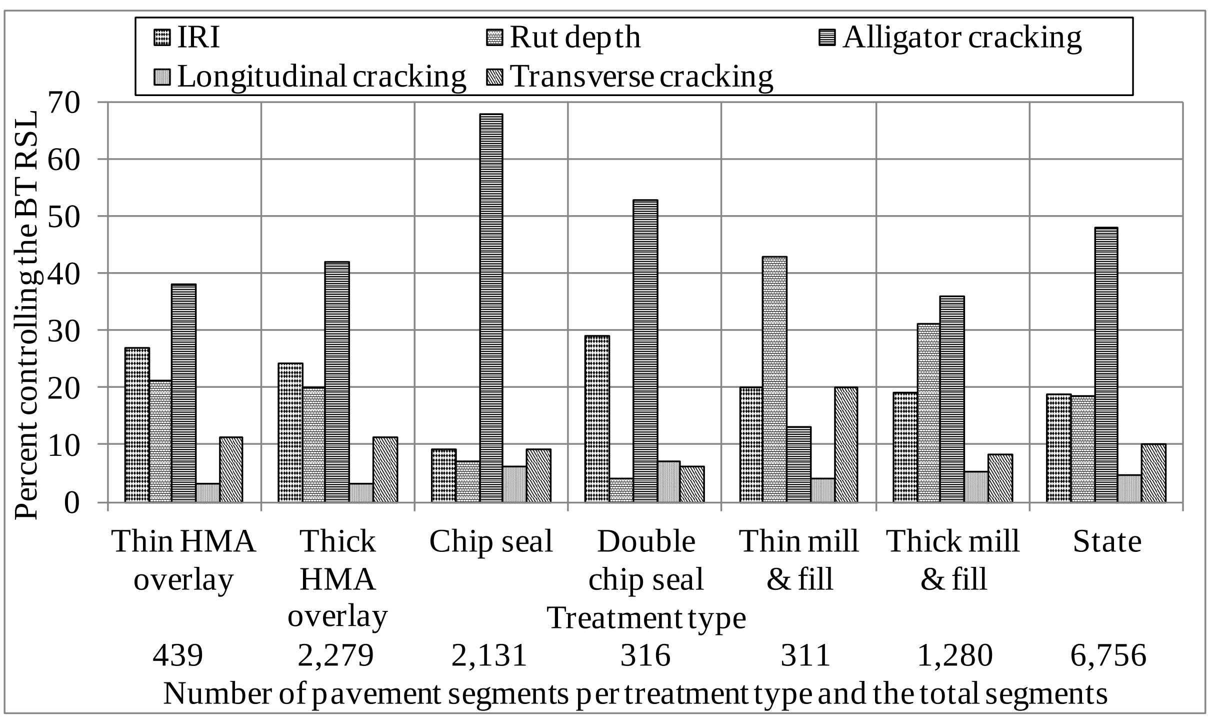


Figure 4.37 Pavement treatment types and the percent of pavement condition and distress types causing the minimum RSL value in the State of Louisiana

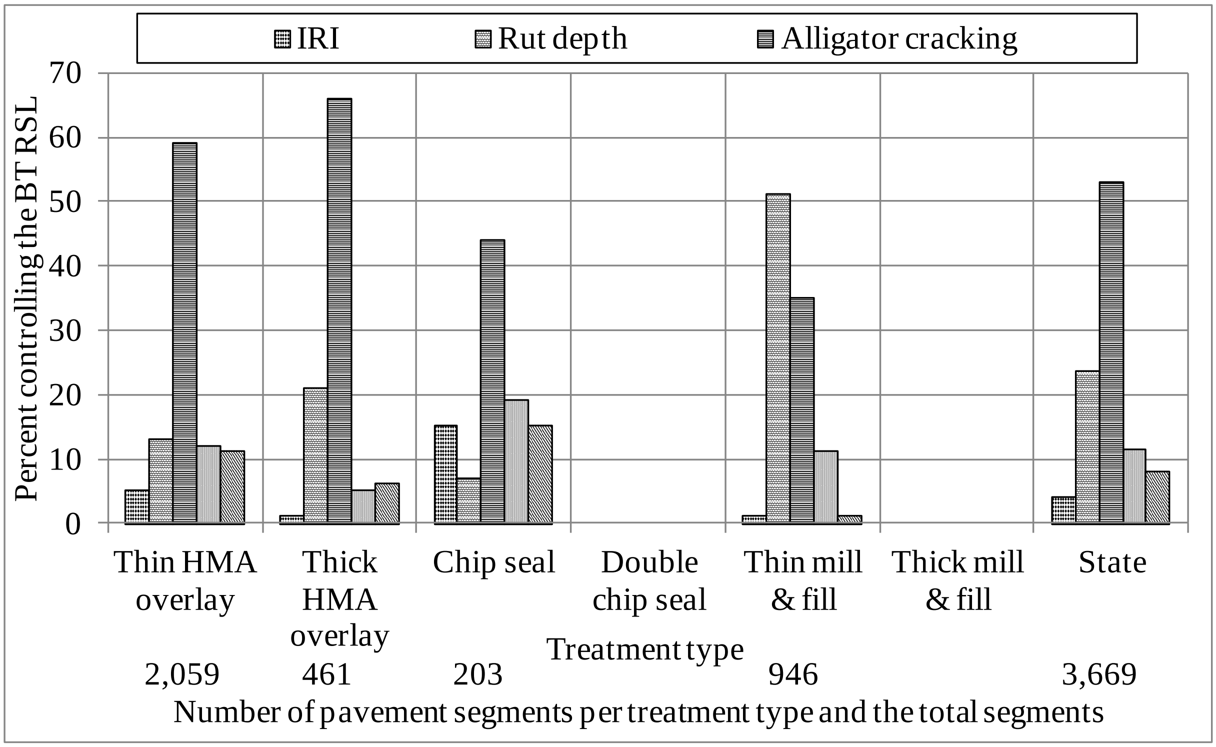


Figure 4.38 Pavement treatment types and the percent of pavement condition and distress types causing the minimum RSL value in the State of Washington

model the pavement condition or distress data over time. Hence, some of these HAs may choose to simply examine the last collected pavement condition and distress data before treatment and the pavement condition and distress data collected at various times after treatment without modeling the time-series data. For example, the pavement condition and distress data could be plotted as the BT pavement conditions or distresses versus the conditions or distresses at a given time in the future after treatment. Figures 4.39 through 4.43 show the pavement conditions and distresses immediately BT plotted against the pavement condition and distress data collected 6 years after the application in 2001 of thin HMA overlay of an asphalt surfaced pavement section along LA 9, in Louisiana. The diagonal 45-degree lines in the figures are the lines of equality indicating identical pavement conditions or distress after 6 years. The data in the figures clearly show that the measured IRI and rut depth 6 years after the treatment are lower than the measured

BT conditions and distresses. However no clear trend was found for alligator, longitudinal, or transverse cracking. This information could be useful if similar figures were created for each data collection cycle AT. Using this procedure, the AT year at which the pavement condition or distress data are close to the line of equality is the TL. Likewise, the AT year when the data reach the pre-specified threshold value is the AT RSL. In other words, the data could be tracked in time-series to determine the treatment benefits. The advantages and shortcomings of analyzing the pavement condition data without modeling the time-series pavement conditions or distresses are listed in Table 4.45.

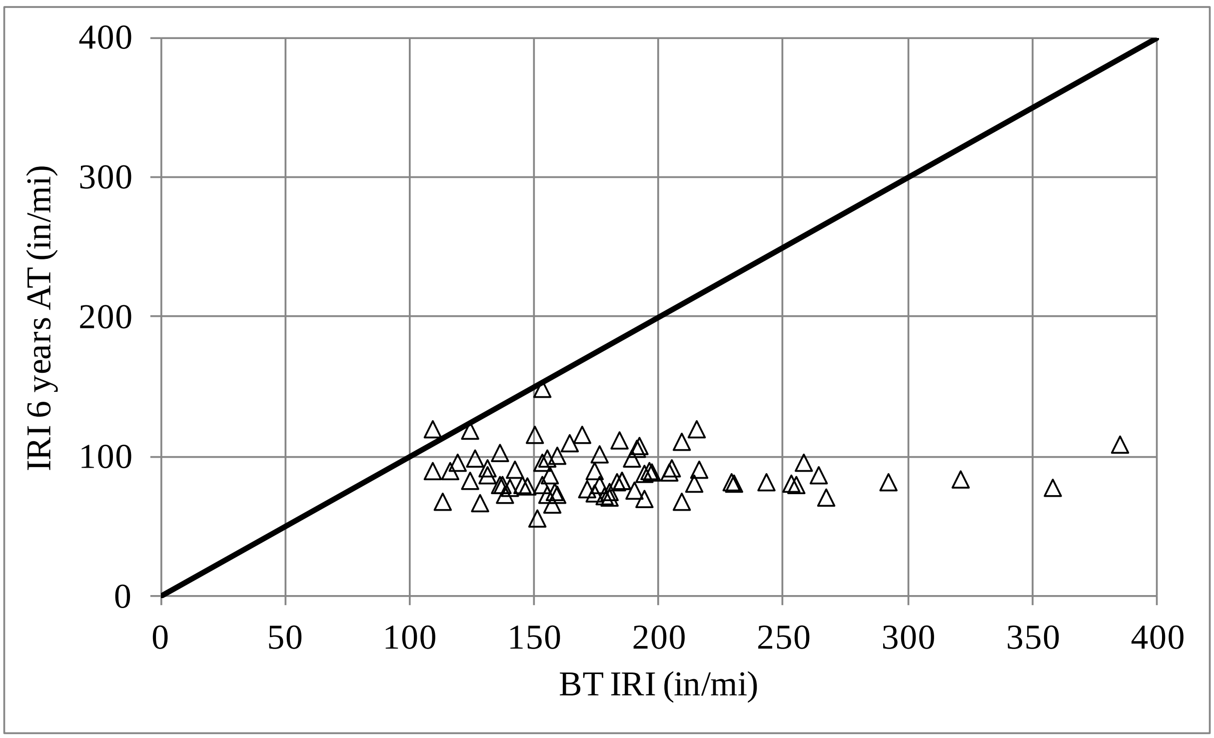


Figure 4.39 BT IRI vs. IRI 6 years AT, LA 9, control section 043-06-1, BMP 7.1 to 14.5, Louisiana

To combat some of the disadvantages of not modeling the time-series data, the results could be stored in modified T^2Ms based on condition or distress alone. The main disadvantage of requiring multiple analyses remains, as one modified T^2M is required for each AT year when the

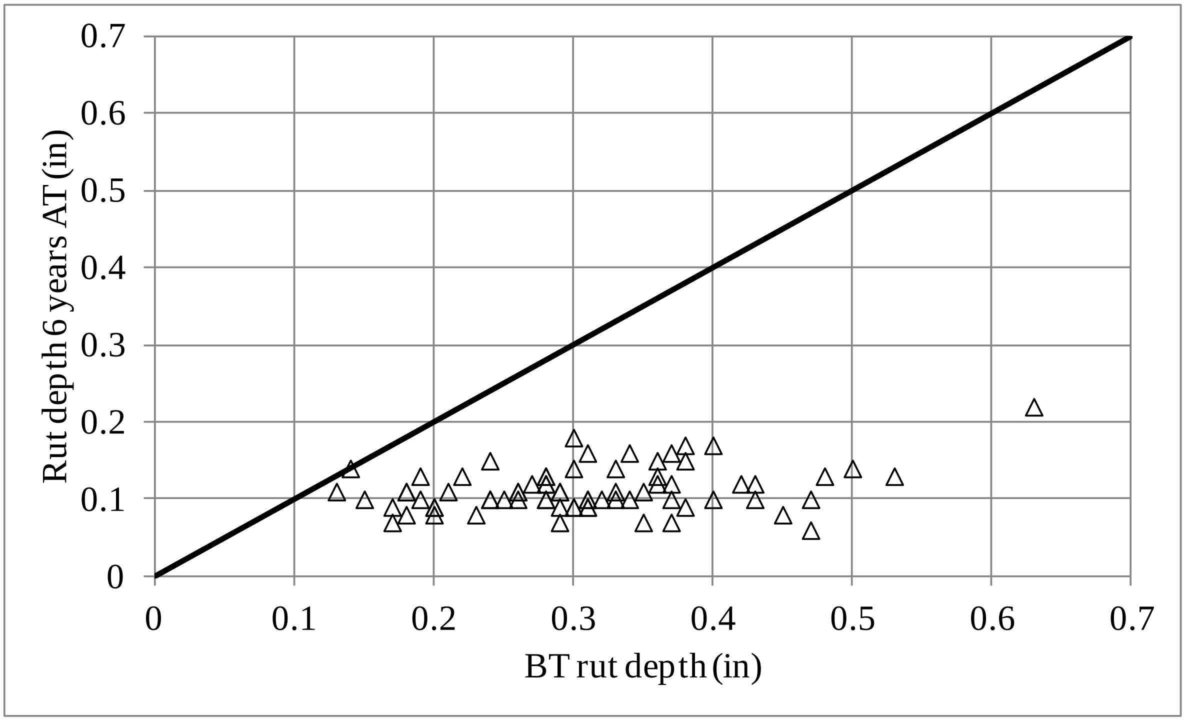


Figure 4.40 BT rut depth vs. rut depth 6 years AT, LA 9, control section 043-06-1, BMP 7.1 to 14.5, Louisiana

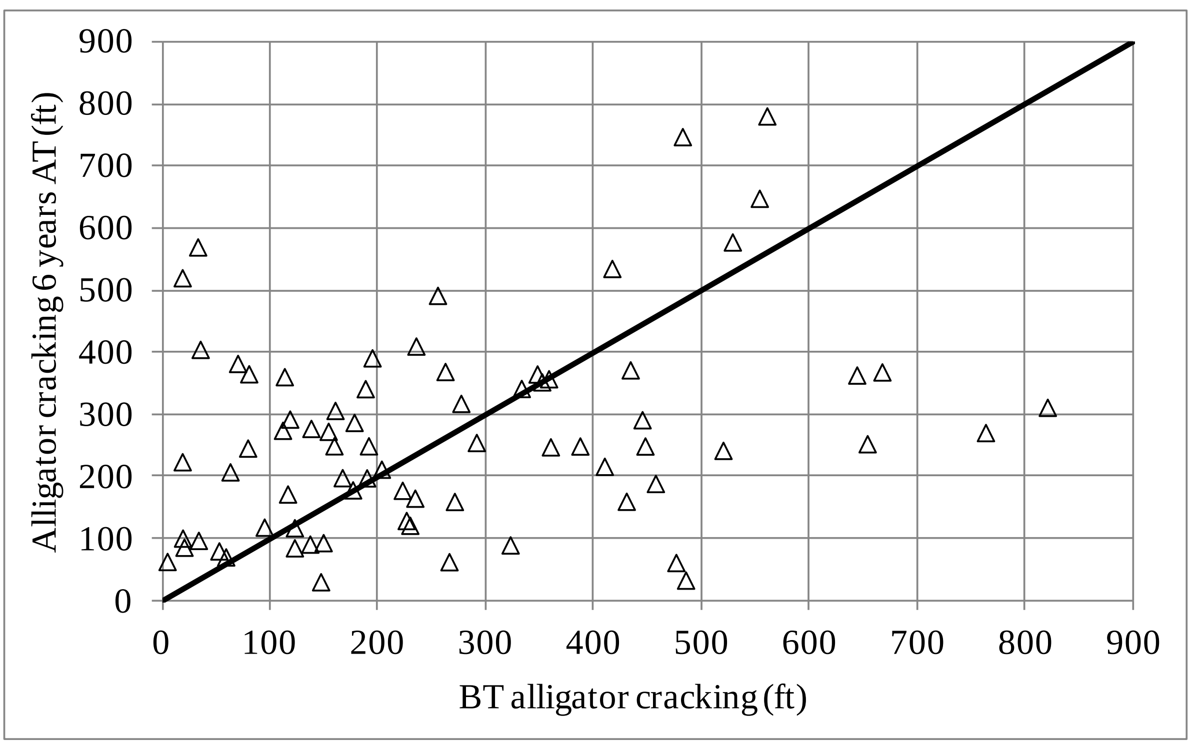


Figure 4.41 BT alligator cracking vs. alligator cracking 6 years AT, LA 9, control section 043-06-1, BMP 7.1 to 14.5, Louisiana

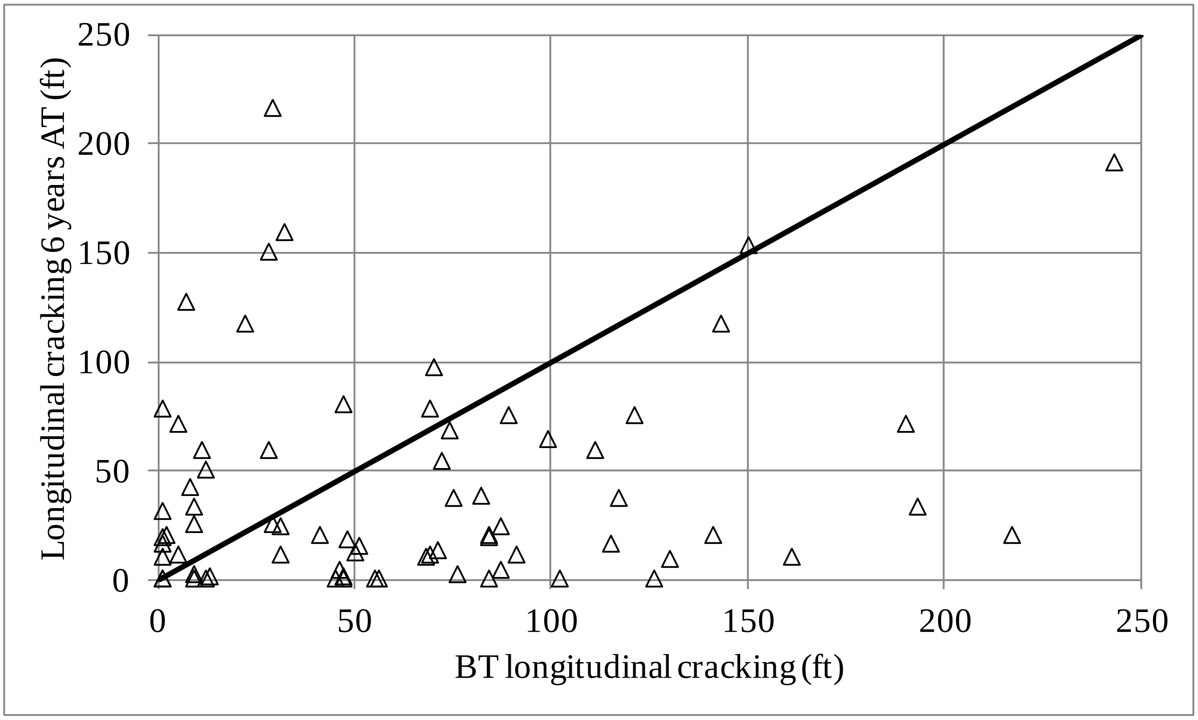


Figure 4.42 BT longitudinal cracking vs. longitudinal cracking 6 years AT, LA 9, control section 043-06-1, BMP 7.1 to 14.5, Louisiana

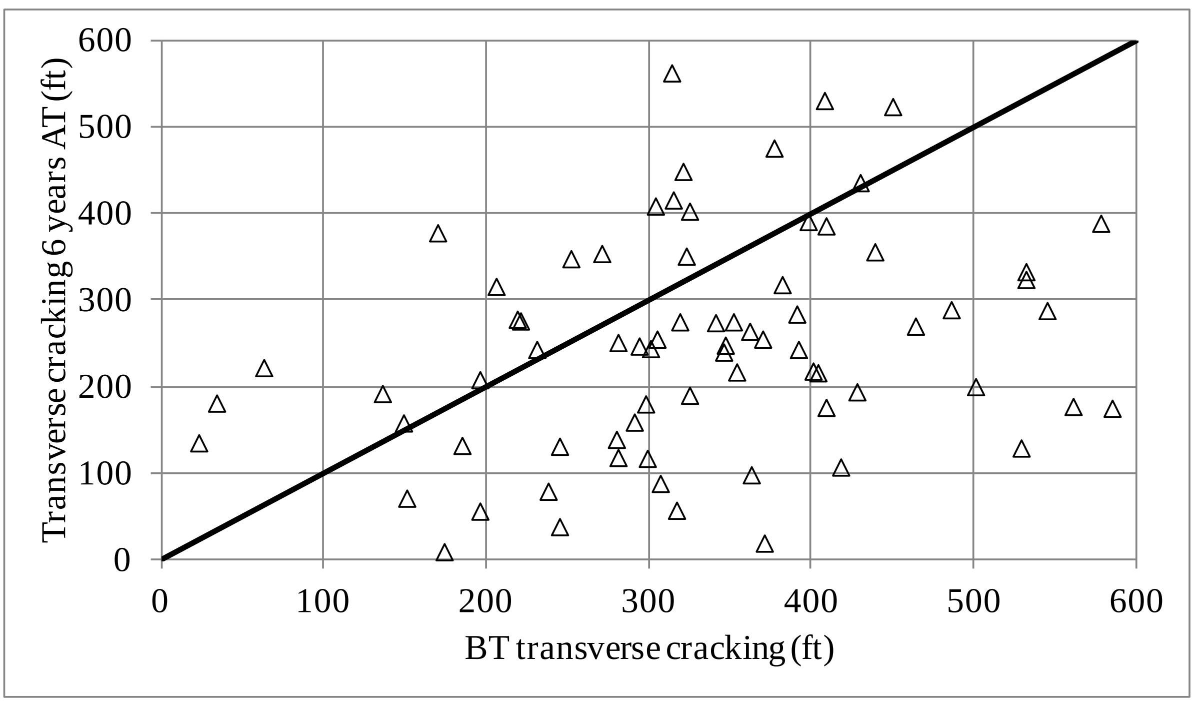


Figure 4.43 BT transverse cracking vs. transverse cracking 6 years AT, LA 9, control section 043-06-1, BMP 7.1 to 14.5, Louisiana

Table 4.45 Advantages and shortcomings of analyzing pavement condition data without modeling pavement conditions (Alternative T^2M)

Advantages	Shortcomings
No variability and/or uncertainty are associated with pavement condition modeling and prediction	Separate analyses are required for each year when the pavement condition and distress data are collected
	Cannot predict future pavement conditions, distresses, or treatment benefits
	The AT service life or the treatment life cannot be obtained until they are realized
	The units used in the analysis (such as crack length, crack area, rut depth, and IRI) are different

data are collected. Before listing the data in T^2M, the pavement condition and distress data were divided into brackets that are compatible with the condition states (RSL brackets discussed earlier). The condition and distress brackets were determined by assuming a pavement segment has a service life of 20 years and that it will deteriorate following the mathematical functions used in this dissertation (exponential function for IRI and cracking and power function for rut depth). Figure 4.44 illustrates the idealized pavement condition and distress models and the ranges of pavement conditions and distresses corresponding to the condition states. The pavement condition and distress brackets shown in Figure 4.44 are listed in Table 4.46.

The IRI data from Figure 4.44 were sorted by the condition brackets listed in Table 4.46 and used to populate the modified T^2M listed in Table 4.47. The modified T^2M lists the number of pavement segments with certain range of BT conditions and certain ranges of conditions 6 years AT. For comparison, the condition state based T^2M for the same project is listed in Table 4.48. The condition bracket distributions listed in Table 4.47 are similar to those listed in Table 4.48, but the treatment benefits cannot be determined from the data in Table 4.47. Similar

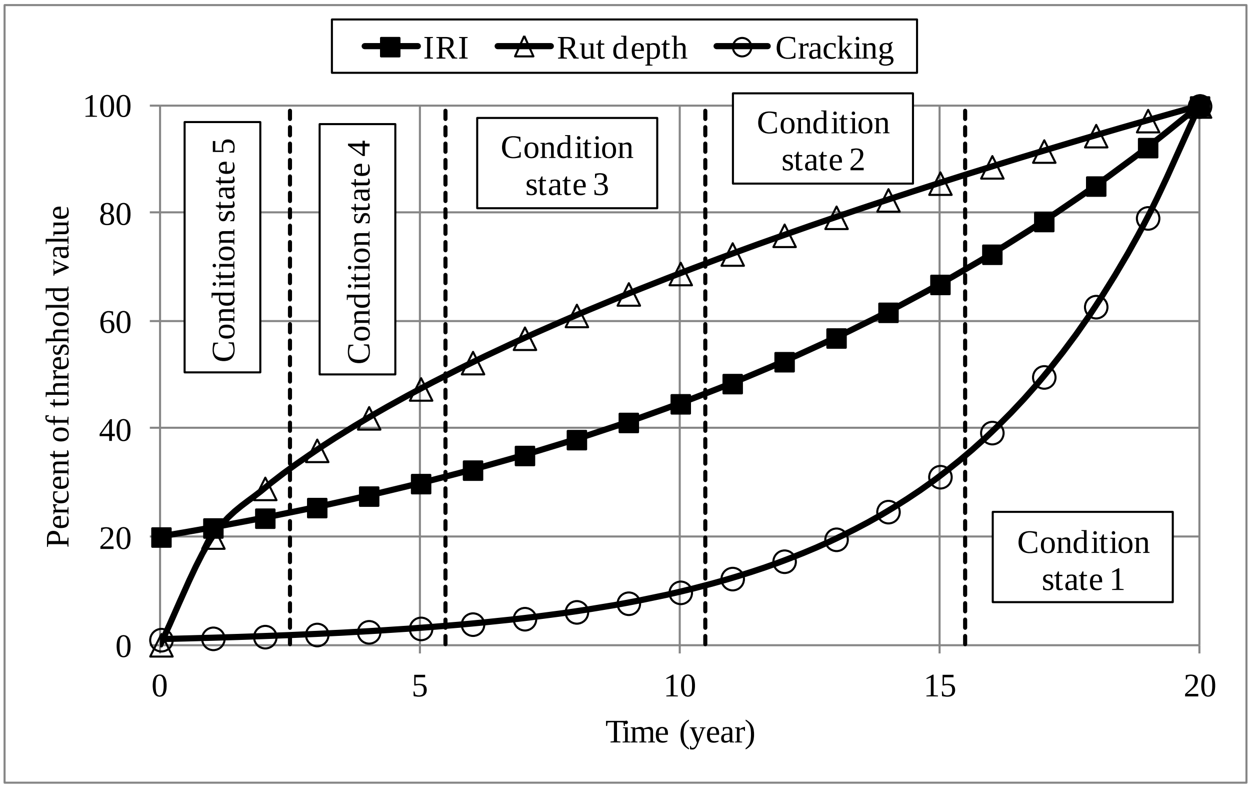


Figure 4.44 Ranges of pavement conditions or distresses corresponding to the condition states

Table 4.46 Pavement condition and distress brackets corresponding to the condition states

CS or RSL bracket and condition or distress bracket number and RSL range in years		Pavement condition or distress type and range				
		IRI (in/mi)	Rut depth (in)	Alligator crack (ft)	Longitudinal crack (ft)	Transverse crack (ft)
1	0 to 2	≥ 170	≥ 0.47	≥ 66	≥ 364	≥ 364
2	3 to 5	134 to 169	0.43 to 0.46	33 to 65	136 to 363	136 to 363
3	6 to 10	89 to 133	0.34 to 0.42	10 to 32	26 to 135	26 to 135
4	11 to 15	60 to 88	0.24 to 0.33	4 to 9	5 to 25	5 to 25
5	16 to 20	≤ 59	0 to 0.23	0 to 3	0 to 4	0 to 4

modified T^2Ms could be populated for each AT year, creating a series of modified T^2Ms which could then be used in place of the single T^2M (based on BT and AT RSL values) to determine some of the treatment benefits. The modified T^2Ms need to be populated each time the AT pavement condition or distress data area collected until the specified (threshold) conditions are reached at certain times in the future. The AT years when the threshold values are reached would correspond to the AT RSL listed in the pavement condition state based T^2M. For example, the pavement sections treated from IRI condition state 1 (or condition bracket 1) would reach the threshold value (200 in/mi) in 20 years (RSL is 20 years in the condition state based T^2M); whereas the series of modified T^2Ms could be searched until the IRI threshold value was reached at the twentieth iteration (AT year 20). The main difference between the RSL based T^2Ms and distress values based T^2Ms is that the former account for the pavement rate of deterioration whereas the latter does not. The modified T^2Ms present a snap-shot of one time when the pavement conditions and distresses are measured. Nevertheless, similar method could be used to determine the TL, however the SLE cannot be determined because the BT RSL is unknown. The population of a series of modified T^2Ms is laborious and time consuming. Further, it may not be possible to populate modified T^2Ms covering the entire pavement service life as pavement treatments are often applied prior to reaching the threshold values. This would certainly be less effective than the procedures listed in the previous subsections. Hence, the remainder of the dissertation is based on modeling the pavement conditions and distresses and the population of condition state based T^2Ms.

Table 4.47 Modified T^2M based on IRI for one pavement project, 6 years AT, along LA 9, control section 043-06-1, Louisiana

<table>
<tr><th colspan="13">Condition/distress type: IRI</th></tr>
<tr><td colspan="4" rowspan="2">Before treatment (BT) data</td><td colspan="8">After treatment (AT) data</td></tr>
<tr><td colspan="5">Condition bracket number and range in in/mi, and the number of the 0.1 mile pavement segments transitioned from each BT condition bracket to the indicated condition brackets</td><td colspan="3">Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)</td></tr>
<tr><td rowspan="2">Condition bracket number</td><td rowspan="2">Condition bracket range (in/mi)</td><td colspan="2">0.1 mile pavement segments</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td rowspan="2">TL</td><td rowspan="2">SLE</td><td rowspan="2">RSL</td></tr>
<tr><td>Number</td><td>Percent</td><td>≥ 170</td><td>134 to 169</td><td>89 to 133</td><td>60 to 88</td><td>0 to 59</td></tr>
<tr><td>1</td><td>≥ 170</td><td>29</td><td>56</td><td>0</td><td>0</td><td>0</td><td>2</td><td>27</td><td>NA</td><td>NA</td><td>NA</td></tr>
<tr><td>2</td><td>134 to 169</td><td>15</td><td>29</td><td>0</td><td>0</td><td>0</td><td>0</td><td>15</td><td>NA</td><td>NA</td><td>NA</td></tr>
<tr><td>3</td><td>89 to 133</td><td>8</td><td>15</td><td>0</td><td>0</td><td>0</td><td>1</td><td>7</td><td>NA</td><td>NA</td><td>NA</td></tr>
<tr><td>4</td><td>60 to 88</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>NA</td><td>NA</td><td>NA</td></tr>
<tr><td>5</td><td>0 to 59</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>0</td><td>NA</td><td>NA</td><td>NA</td></tr>
<tr><td colspan="2">Total/average</td><td>52/</td><td>100/</td><td>0/</td><td>0/</td><td>0/</td><td>3/</td><td>49/</td><td>/NA</td><td>/NA</td><td>/NA</td></tr>
</table>

Table 4.48 T^2M based on IRI for one pavement project, 6 years AT, along LA 9, control section 043-06-1, Louisiana

Condition/distress type: IRI												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the average SE per CS or RSL bracket, and the number of the 0.1 mile long pavement segments transitioned from each BT CS or RSL bracket to the indicated CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Average standard error (SE) (in/mi)	1	2	3	4	5	TL	SLE	RSL
		Number	Percent		0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
					Average SE of each CS or RSL bracket (in/mi)							
								4	4			
1	0 to 2	28	54	19	**0**	0	0	1	27	10	19	20
2	3 to 5	5	10	16	0	**0**	0	0	5	10	16	20
3	6 to 10	7	13	18	0	0	**0**	0	7	10	12	20
4	11 to 15	5	10	8	0	0	0	**1**	4	10	6	19
5	16 to 25	7	13	10	0	0	0	1	**6**	10	-1	19
Total/average		52/	100/		0/	0/	0/	3/	49/	/10	/14	/20

4.5.9 Treatment Timing Cost Effectiveness

The state-of-the-practice in the selection of pavement project boundaries has a significant impact on the effectiveness of the pavement treatments, as shown in the previous few subsections. Treatments applied to pavement sections in worse conditions typically require more pre-treatment repairs (milling, patching, sealing, etc.) and therefore higher costs to achieve similar service lives as pavement sections in better conditions, as portrayed theoretically in Figure 4.45. For example, pavement segments in BT RSL bracket 5 may not require any pre-treatment repair prior to an HMA overlay and thus zero additional cost, while a pavement segment in BT RSL bracket 1 may require extensive patching and crack sealing and thus extensive additional cost. Unfortunately the detailed cost data for each 0.1 mile long pavement segment do not exist in the databases of the three SHAs, as discussed in section 3.5 of Chapter 3. In fact, this level of detail in the cost data is likely not collected by any SHAs. Therefore, the pre-treatment repair cost was assumed to be uniform for all treated pavement segments regardless of their before treatment conditions as shown in Figure 4.45.

Nevertheless, insufficient cost data were available from the States of Colorado and Washington to conduct cost-effectiveness analyses of the treatments. However, the average total treatment costs for thin and thick HMA overlays of asphalt surfaced pavements in each district in Louisiana were available and used to study the variation in cost relative to the BT RSL brackets. However, the cost data per 0.1 mile long pavement segment were not available, it was assumed that the cost is uniformly distributed along each project. The number of 0.1 mile long pavement segments in BT RSL brackets 1 through 5 was plotted versus the average total treatment cost per 0.1 mile long segment for each district in the State of Louisiana. The results for thin and thick HMA overlays of asphalt surfaced pavements are shown in Figures 4.46 and 4.47, respectively.

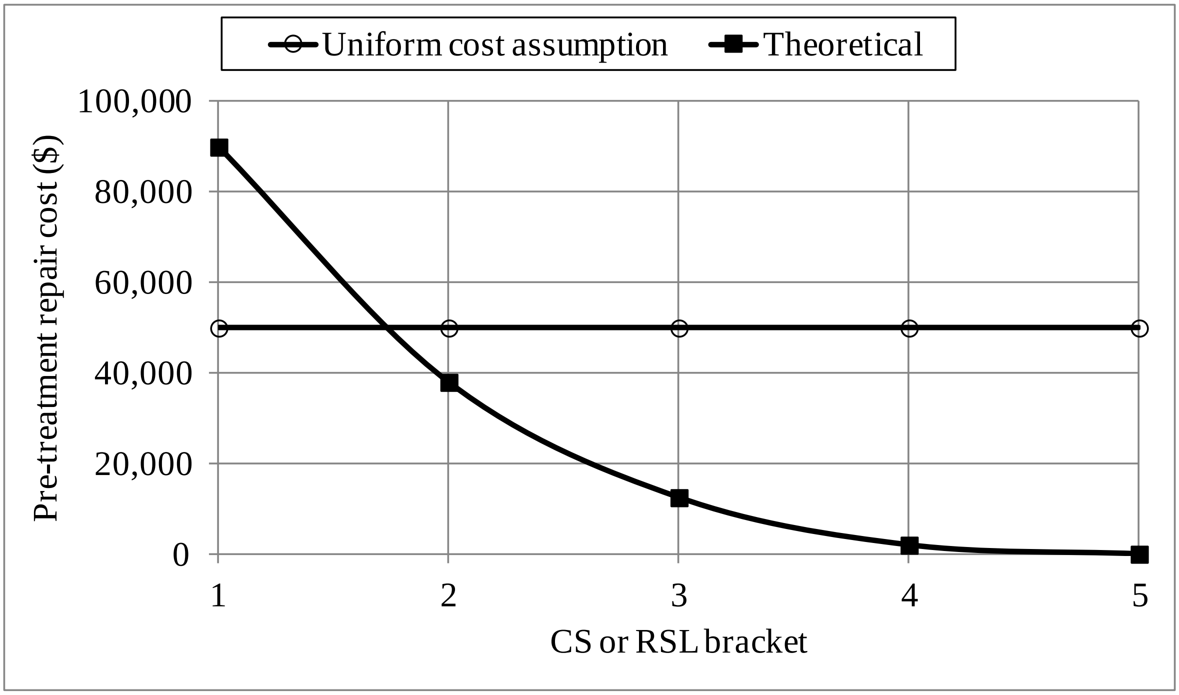


Figure 4.45 Uniform and theoretical pre-treatment costs versus pavement condition state

It was expected that the districts which have higher number of pavement segments in BT RSL bracket 1 (worse condition) would have higher cost, and lower cost for high number in BT RSL bracket 5 (good condition). However, the data in the figures show that the cost per 0.1 mile long pavement segment is independent of the number in BT RSL brackets 1 through 5. This finding is not surprising given the coarseness of the available cost data and the uniform pre-treatment cost assumption. Nevertheless, the data in Figure 4.46 to some extent show that as the number of pavement segments in BT RSL bracket 1 (solid diamonds) increases the total, average cost of treatment increases, with the exception of the one outlier near 190 pavement segments. This implies that treating pavement in worse condition (lower RSL) costs more money.

Knowing the limitations of the cost data, the total treatment costs and the treatment benefits were analyzed to study the cost-effectiveness relative to the BT condition states of the various pavement treatments in Louisiana using Equation 4.1.

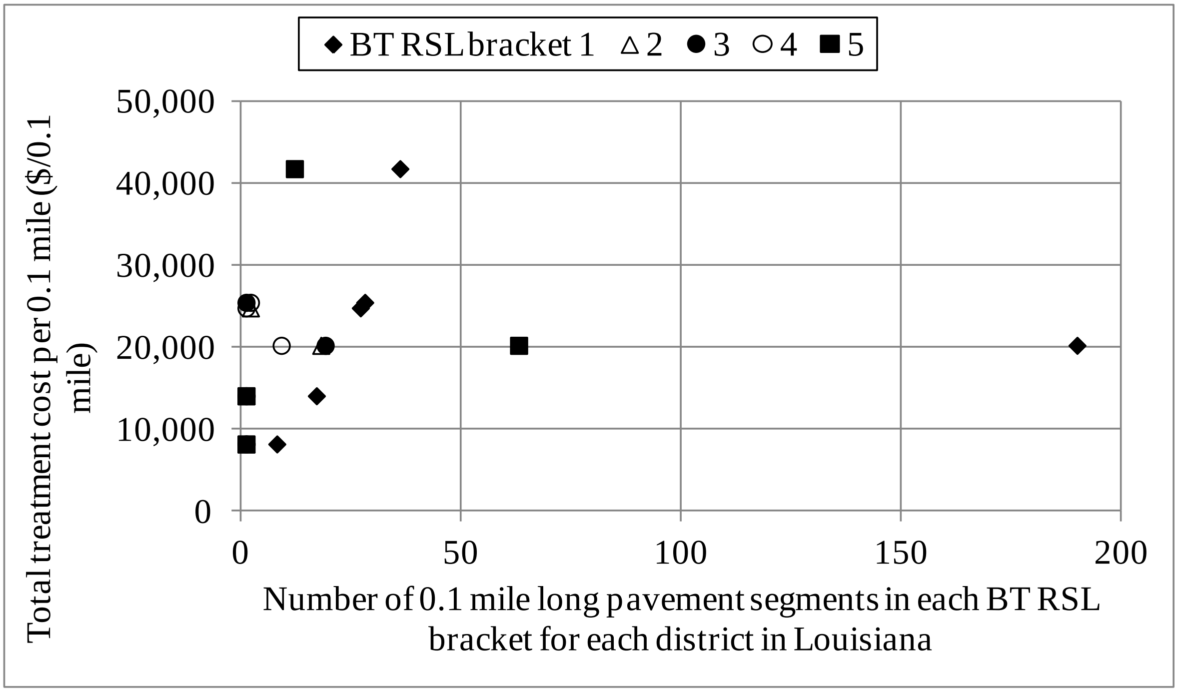


Figure 4.46 Number of 0.1 mile long pavement segments in BT RSL brackets 1 through 5 vs. the total treatment cost per 0.1 mile for each district in Louisiana, thin HMA overlay of asphalt surfaced pavements

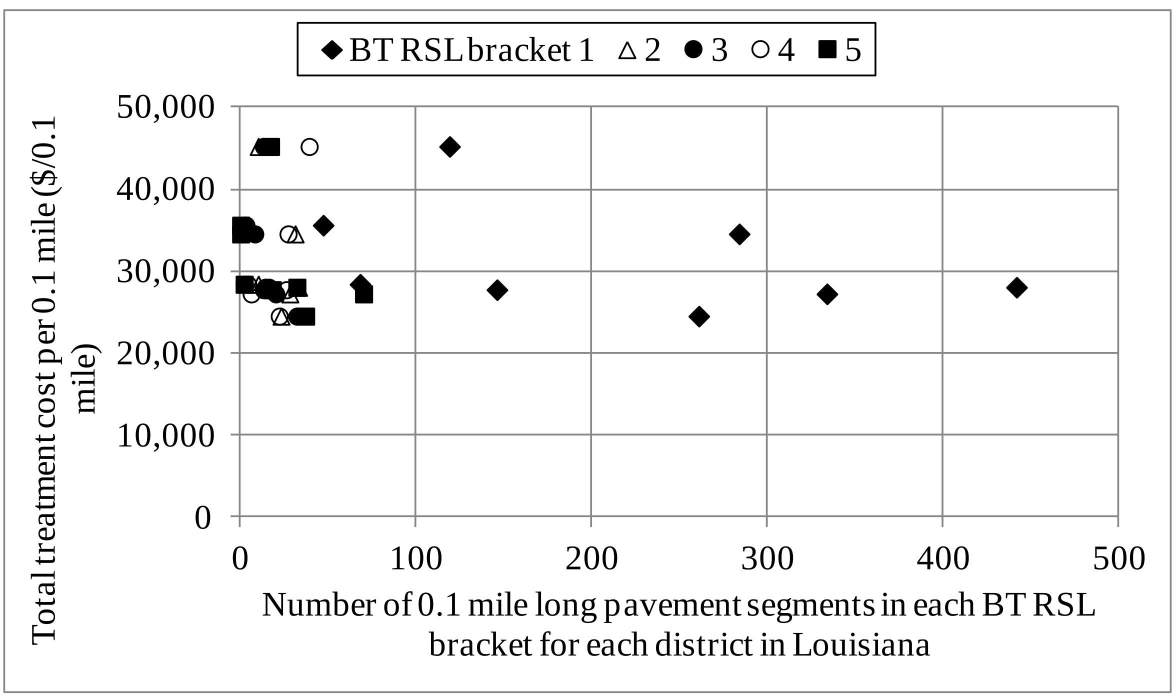


Figure 4.47 Number of 0.1 mile long pavement segments in BT RSL brackets 1 through 5 vs. the total treatment cost per 0.1 mile for each district in Louisiana, thick HMA overlay of asphalt surfaced pavements

$$\text{Objective Function} = \text{Minimum}\left(\frac{\text{Costs}}{\text{Benefits}}\right) \qquad \text{Equation 4.1}$$

Figures 4.48 through 4.50 show the BT condition states versus the ratio of the total treatment cost per 0.1 mile to the AT benefits TL, SLE, and RSL, respectively for each treatment type. The data in the figures indicate that:

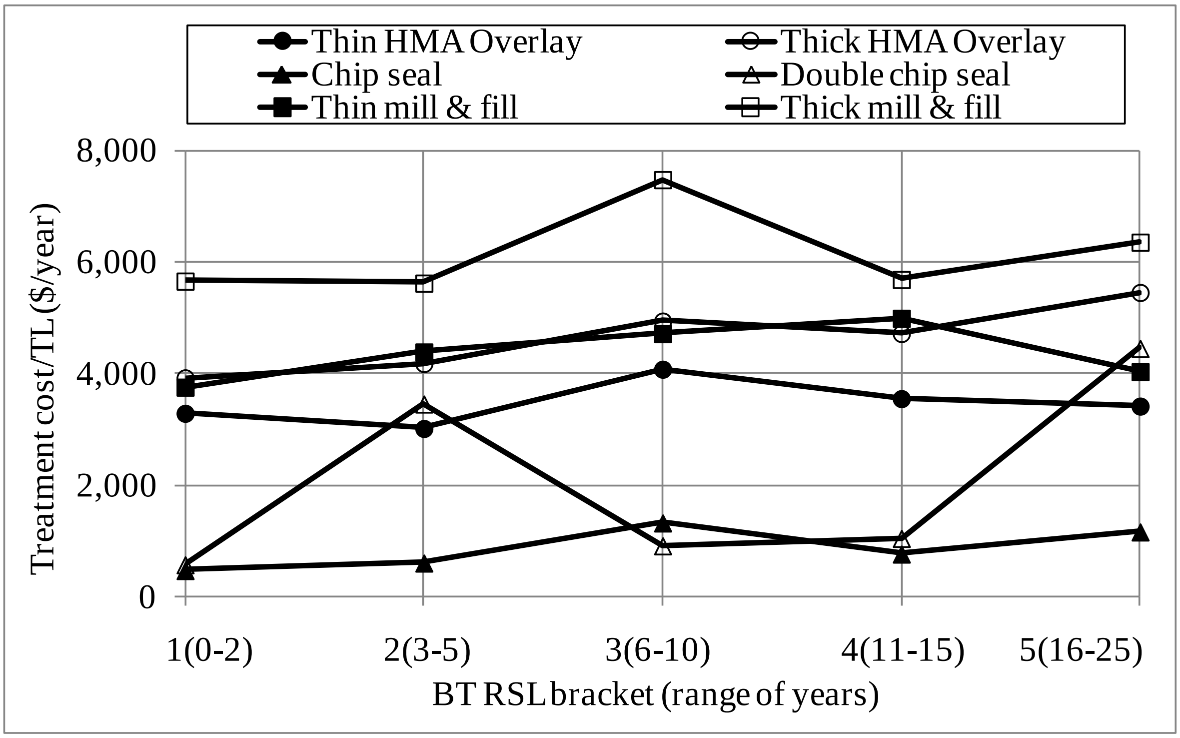


Figure 4.48 The BT RSL bracket versus the treatment cost per year of TL for each treatment in Louisiana

- The cost per year of TL is independent of the BT RSL as shown in Figure 4.48. This was expected because the TLs of 0.1 mile long pavement segments in the lower BT RSL brackets (worse conditions) are generally longer and the costs are generally higher. Whereas, the TLs for pavement segments in better BT conditions are shorter and the costs are lower. Therefore, the TL is not an adequate denominator to be used in the analysis, and the SLE and the AT RSL were considered.

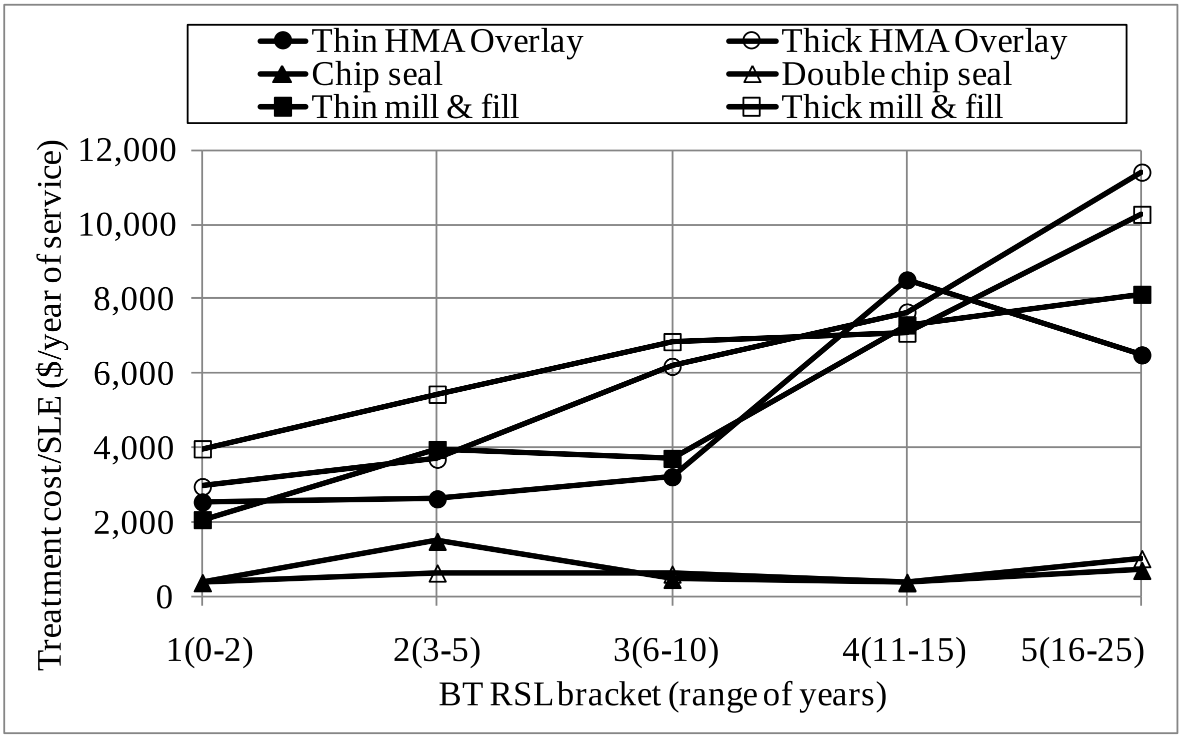


Figure 4.49 The BT RSL bracket versus the treatment cost per year of SLE for each treatment in Louisiana

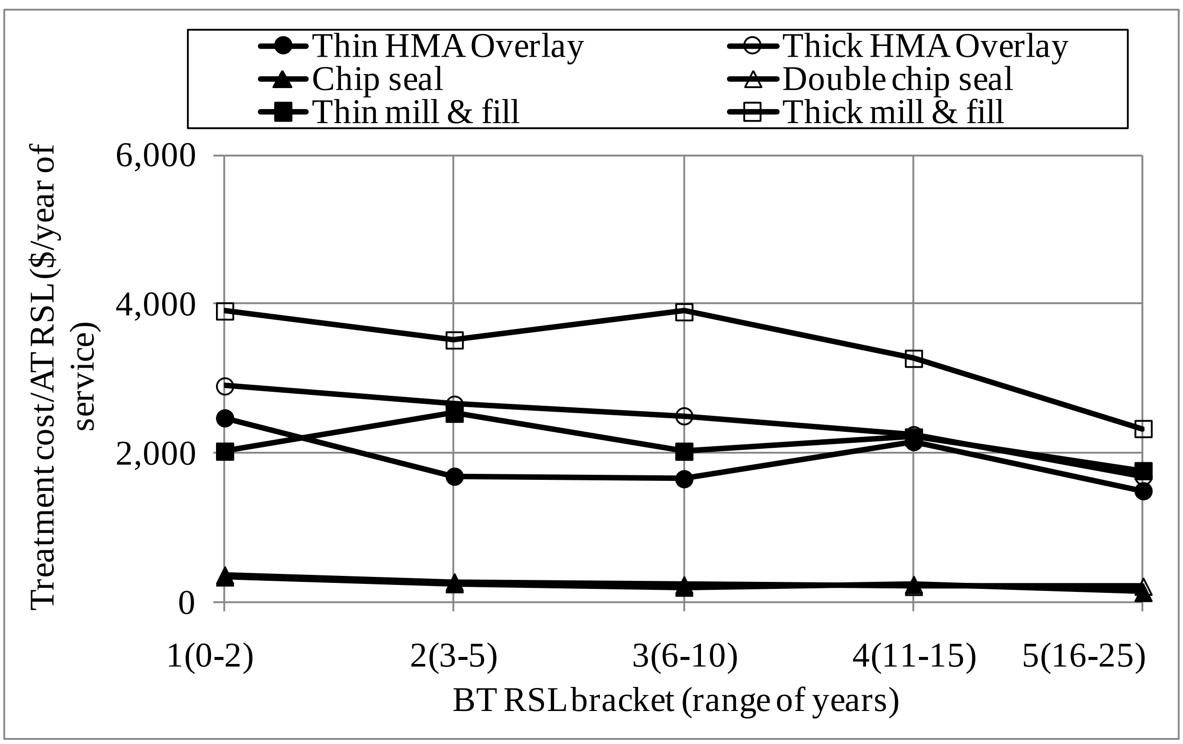


Figure 4.50 The BT RSL bracket versus the treatment cost per year of AT RSL for each treatment in Louisiana

- The cost per year of SLE increases as the BT RSL increases for thin and thick HMA overlays of asphalt surfaced pavements and for thin and thick mill and fill treatments as shown in Figure 4.49. This implies that the cost to treat pavement segments to add additional years of service is the lowest when the pavement is in worst conditions. Further, the cost per year of SLE is independent of the BT RSL bracket for single and double chip seal. The last observation was expected because single and double chip seal treatments do not provide the pavement with additional structural capacity nor do they solve roughness, rutting, and/or cracking problems. Chip seal treatments increase the pavement surface friction and/or slow down the rate of pavement oxidation and aging. Hence, the cost effectiveness of chip seal treatments is independent of the BT RSL bracket.
- For thin and thick HMA overlays of asphalt surfaced pavements and for thin and thick mill and fill treatments, the cost per year of AT RSL slightly decreases as the BT RSL increases as shown in Figure 4.50. This implies that the cost of the treatments to maximize the after treatment longevity of the pavement sections decreases if the selected pavement segments have better condition states before treatment. This was expected because pavement segments having high RSL brackets before treatment will generally have high RSL brackets after treatment. For single and double chip seal treatments, on the other hand, the cost per AT RSL is independent of the BT RSL bracket. This was expected for similar reason as described in the previous bullet.

The above results should be reviewed with caution because the cost data include all extraneous costs such as sign, guard rail, and shoulder improvements, mobilization charges, the cost of incentive payments, and so forth. The cost data were obtained from the project contract files and the cost per 0.1 mile long pavement segment within each pavement project was assumed uniformly distributed regardless of the BT condition state. The above analysis is simply made as an example of how the results of this study could be further investigated if the detailed cost data were made available. Since the pavement conditions along almost each pavement project vary from excellent to poor, and since the costs of treating the various pavement conditions are drastically different, the analyses would be more accurate if the cost data for each 0.1 mile long pavement segment are made available.

4.5.10 Methodology to Improve the Population of T^2Ms

The data presented in the T^2Ms of the previous subsections were calculated and estimated based on the analyses performed on the pavement condition and distress data measured by the various SHAs. Through the process of analyzing the pavement condition and distress data and the supporting data, such as the treatment and maintenance files, several things became apparent which hindered the pavement condition modeling and reduced the accuracy of the results listed in the T^2Ms. These issues and the suggested methodologies to improve the PMS operations and hence the population of T^2Ms are discussed below.

1. The completeness and consistency of the pavement treatment and maintenance files could be improved substantially. Every time a pavement treatment is applied, from routine maintenance such as crack sealing to major rehabilitation such as PCC slab replacement, the treatment types, locations, detailed costs, etc. should be stored in the database. These information are likely available in various forms (computer files, hard copy in the project files, micro-films, and so forth) in different specialty offices within the SHAs. The ability to integrate the data in the PMS database is minimal to non-existent. The main reason is that most data are stored using different reference location systems such as survey station, lots

and sub-lots, mile points, etc. These reference systems are not compatible and are extremely difficult to integrate into one common system. These scattered information are critical for:

a. Identifying treated pavement sections for analysis.

As described in section 3.3 of Chapter 3, the first step in the analysis of the pavement condition and distress data is the identification of treated pavement sections from the treatment files. This information must be as complete and detailed as possible to increase the data pool and to improve the efficiency of the analyses. Pavement sections which were treated without record cannot be included in the analyses. Likewise, treated pavement sections included in the analyses which were subjected to secondary treatments, not included in the database, typically were not included in the analysis because of the second acceptance criterion.

The responsibility for performing routine maintenance is often delegated to the districts and regions. This practice is likely needed and efficient; however the data regarding the applications of these treatments often remain in the districts and are not integrated into the database. For example, few pavement sections were found to show decreasing rut depths for few segments almost every year for some periods. The anomaly was brought to the attention of the appropriate SHAs who shared that this was likely the result of sporadic filling of the rut channels to maintain safety by the district offices. Although the proper action was taken to maintain safety, the information were not included in the PMS database.

b. Properly identifying the pavement treatment types and locations.

During the data mining parts of this study, it was found that the data stored in the pavement treatment files were often very limited in detail, such as the treatment type, the exact overlay thicknesses, and occasionally, the project locations. For example, some HMA overlay projects are listed as having range of thickness from 1 to 3 inches. The details regarding which pavement sections or segments received 1-inch and which received 3-inches were found on papers in the stored project files not in the PMS database. Further, the location reference system to the location where the overlay thickness was changed is project specific and cannot be deciphered. Another example is when the treatment category is simply listed as pavement rehabilitation. Pavement rehabilitation could include replacement of PCC slabs, significant crack and joint patching, HMA overlay, mill and fill, etc. Again, the details are listed on papers located in the project files and not included in the PMS database.

c. Accurately assigning pavement surface age for modeling.

The pavement surface age is an integral part of the pavement modeling process. In particular, the BT surface age was difficult to determine due to the incompleteness of the treatment files. Recall that, due to the lack of sufficient information, the first available pavement condition and distress data collection cycle was assumed to be completed when the pavement surface age was one year. This assumption could have some impact on the pavement condition or distress model and the calculated treatment benefits. The most significant impact would be for those SHAs which calculate the total benefits (TB), which includes the do-nothing area. Shortened BT surface age decreases the do-nothing area and consequently inflates TB. The impact of the surface age on the RSL, and therefore the results of this study, is much less than that of the TB. Nevertheless, the use of the proper surface age would likely improve the pavement modeling and the calculation of treatment benefits.

d. Performing cost-benefit analyses.

The cost data collected by most SHAs are not detailed enough to properly conduct cost-benefit analyses with the same detail as that included in the T^2Ms and discussed in subsection 4.5.9. The treatment cost, when available, only includes the total contract amount and no details regarding the breakdown on the materials, labor, additional treatments, incentive/disincentives, mobilization, safety improvement, guardrails, shoulder improvement, or any other information. Detailed data are required in the analysis of the cost-effectiveness of the treatments. The cost of performing treatment depends on the required amount of pre-treatment repair and should be detailed for each 0.1 mile long pavement segment to properly determine the treatment cost-effectiveness.

2. On the other hand, the accuracy of the available data could be greatly improved, specifically the location referencing system. Each of the three SHAs stores the pavement condition and distress data for each 0.1 mile long pavement segment using linear referencing system. Analyses of the accuracy of the linear referencing system were conducted by comparing the BMPs to the recorded Global Positioning System (GPS) data recorded in Colorado, Louisiana, and Michigan. The analyses were conducted utilizing the Pythagorean Theorem to calculate the distance between two GPS locations, as shown in Equations 4.9 through 4.11. The Pythagorean Theorem was used to calculate the hypotenuse of the right triangle formed between two points, assuming the road had no horizontal or vertical curvature. This assumption could result in large errors over long distances as the pavement surface curves upward, downward, and side to side. However, since the analyses were conducted for each

0.1 mile long pavement segment of relatively level and straight Interstate, U.S. and state roads, the associated error was minimized. Equations 4.9 through 4.11 were used to determine: a) the length between the BMPs of adjacent pavement segments and b) the distance between BMPs of the “same” pavement segment in time-series.

$$PSL = \sqrt{(\Delta\, Latitude)^2 + (\Delta\, Longitude)^2} \qquad \text{Equation 4.9}$$

$$\Delta\, Latitude = (Latitude_A - Latitude_B) * 69.047 \qquad \text{Equation 4.10}$$

$$\Delta\, Longitude = (Longitude_A - Longitude_B) * 69.047 * COS\,(Latitude_A) \qquad \text{Equation 4.11}$$

Where, PSL is the pavement segment length in miles;

$Latitude_A$ and $Latitude_B$ are the measured latitude for two BMPs, A and B;

$Longitude_A$ and $Longitude_B$ are the measured longitude for two BMPs, A and B;

69.047 is the conversion factor from degrees latitude to miles;

69.047 * COS (Latitude) is the conversion factor from degrees longitude to miles to account for the Earth curvature and decreasing distances between two latitudes as the location moves north toward the equator

a. The accuracy of the pavement segment length (PSL) was checked by comparing the difference in the reported, linear distance between BMPs, of adjacent pavement segments, to the PSL calculated using the measured GPS data. This was conducted for many segments in each of the three states listed in Table 4.49. The data in the table indicate that the PSL is accurate within ± 10 feet more than 95% of the time and ± 529 feet for only 0.1 percent of the time. Hence, the data in the table indicate that the use of the linear referencing system to measure distance is quite accurate. However, the linear references are not physical locations marked on the pavement. Hence, the pavement survey equipment cannot match locations identically from one condition survey cycle to the next. This results in large inconsistencies in time-series, as discussed below.

Table 4.49 Accuracy of the pavement segment length measured by the linear referencing system

Range of difference between the BMPs and the PSL (feet)	SHAs, the number of pavement segments included in the analyses, and the percentage within each range		
	CDOT	LADOTD	MDOT
	65,526	55,139	62,464
± ≤ 10	96.3	98.4	95.7
± 11 to 50	2.6	0.9	4.2
± 51 to 100	0.3	0.2	0.1
± 101 to 250	0.3	0.3	0
± 251 to 528	0.4	0.2	0
± > 529	0.1	0	0
The CDOT data were recorded in 2007, LADOTD in 2005, and MDOT in 2008 & 2009 (Sensor data)			

b. The accuracy of the linear reference system was checked in time-series by comparing the difference in the measured GPS locations of the linear referenced BMPs between two pavement condition collection cycles. Accurate linear location reference to a given point along the pavement should result in similar GPS coordinates from one data collection cycle to the next. For example, the BMP labeled 1.0 should be near the exact same physical location from one year to the next. To check this, the difference between the GPS coordinates measured for each given BMP between data collection cycles were determined for each state, as listed in Table 4.50. The data in the table indicate that the BMPs are rarely assigned to the same pavement location. In fact the BMPs are more than 50% (± 250 feet) in error 51.4%, 12.9%, and 3% of the time, in CDOT, LADOTD, and MDOT, respectively. The error could be corrected if the BMPs were determined from the measured GPS coordinates, which would greatly improve the consistency in the time-series data.

Table 4.50 Accuracy of the assignment of BMPs by the linear referencing system

Range of distance between the assigned BMPs (feet)	SHAs, the number of pavement segments included in the analyses, and the percentage within each range		
	CDOT	LADOTD	MDOT
	32,905	37,843	63,393
± ≤ 10	9.5	15.0	12.3
± 11 to 50	27.0	21.8	47.4
± 51 to 100	6.8	26.7	22.7
± 101 to 250	5.3	23.6	14.6
± 251 to 528	32.3	8.4	2.5
± > 529	19.1	4.5	0.5
The CDOT data were recorded in 2007 & 2008, LADOTD in 2005 & 2007, and MDOT in 2006 through 2009 (Sensor data)			

The accuracy of the pavement condition and distress models and the T^2Ms could be greatly improved if the pavement treatment files were complete and consistent and if the data were referenced using the more accurate GPS coordinates. The data required to improve the completeness of the pavement treatment files are mostly collected by the SHAs, and their integration into the PMS database would greatly improve the accuracy and efficiency of the data analyses and the population of T^2Ms. Likewise, the GPS coordinates are collected and could be used to assign the linear BMPs much more accurately and improve the time-series pavement condition data for analyses. Finally, if adopted, the GPS could be used by all employees of the agency causing unified reference location system. The requirement is to equip the appropriate agency vehicles by a GPS unit.

3. The pavement projects included in each coarse treatment type could be further subdivided. For example, thin and thick HMA overlay could be further divided into less than 1-inch, 1 to 1.75-inch, 1.75 to 2.5-inch, 2.4 to 4-inch, and greater than 4-inch overlay thickness. Also, the projects could be subdivided by road class, such as Interstate, U.S. route, and state route. Further, the projects could be subdivided by the traffic load and frequency, such as less than

10,000 equivalent single axle load (ESAL) per year, 10,000 to 100,000 ESAL, 100,000 to 500,000 ESAL, and more than 500,000 ESAL per year. In this study, such subdivisions would yield insufficient data for the analyses. The subdivisions would create many small groups of pavement segments which would not be sufficient to properly populate T^2Ms.

Finally, the presented methods and procedures are preliminary in nature due to the availability of the necessary data to properly define the pavement surface age, identify the project boundaries, perform cost/benefit analyses, and so forth. Having more and finer detailed data and more pavement sections for analysis could refine the methodologies. Hence, the presented data analyses and recommendations could serve as a road map to guide SHAs to collect and store more detailed data and to perform the analyses and reap the benefits of the outlined methodologies.

4.6 New Algorithms for the Assessment of Pavement Conditions of Multi-Lane Facilities

In the United States and other developed countries, the majority of the Interstate mileage consists of two lanes (driving and passing lanes) with few mileage having more than two lanes in one direction. According to the 1993 AASHTO Design Guide, the driving lane carries about 85 percent of the traffic, whereas the passing lane about 15 percent (AASHTO 1993). In the past, SHAs have selected pavement treatments for certain pavement sections of multi-lane roads based on the driving lane conditions and distresses and have applied the treatment to all lanes. For economic and other reasons, they began to re-think this process and to apply different treatments to the driving and passing lanes at different times. This new way of thinking and the selection of cost-effective pavement treatment type, time, and project boundaries for each lane individually require knowledge of the pavement conditions and distresses along each lane. Currently, most SHAs collect pavement condition and distress data along the driving lane only; extending the data collection to the other lanes increases their data collection costs significantly. Hence, there is a need to model or estimate the time-series pavement condition and distress data along the passing lane using the measured data along the driving lane. Based on this need it was thought that pavement performance is mainly a function of three factors; traffic load, pavement materials, and the environment. Neglecting the interaction between the three factors, it can be assumed that the effects of the latter two factors on adjacent lanes of multi-lane facilities are similar. The differences in their performances are mainly related to the traffic load distribution between the lanes. Based on this assumption, the following hypothesis was developed in this study:

> "For facilities with multiple lanes in one direction, the measured pavement conditions and distresses and the traffic lane distribution factor (LDF) can be used to estimate the pavement conditions and distresses of the other lanes".

To verify the hypothesis, a new methodology was developed based on the measured pavement condition and distress data along all lanes of the Minnesota Road Research (MnROAD) test facility. According to MnROAD data, the driving lane carries about 80% of the traffic and the passing lane about 20% (MnROAD 2008). Thus, in the methodology development, LDF factors of 0.8 and 0.2 were used in the analysis of the driving and passing lanes, respectively. If the hypothesis is successfully verified, the estimated time-series data along the passing lanes could be analyzed to calculate the pavement deterioration rates, the RSL values, treatment benefits (such as TL, SLE, or AT RSL) and to select cost-effective pavement treatment type, time, and project boundaries as discussed in section 4.7 below.

To this end, for each data collection cycle, the pavement surface condition and distress data of both the driving and passing lanes of the MnROAD test facility were obtained. The data include cracking, IRI, and rut depth. The latter two sets of time-series pavement condition and distress data were used to develop and verify mathematical models for the estimation of the pavement conditions and distresses along the passing lanes using the driving lane data and the LDFs.

Table 4.51 provides a list of the sixteen standard HMA pavement cells along the mainline section of MnROAD. The data in the table indicate that the sixteen cells were designed differently (different asphalt, base, and subbase thicknesses) and all are supported on clay roadbed soil. The pavement along each cell was subjected to in-service traffic by closing the regular west bound lanes of I-94 and directing the traffic onto the test cells, which run parallel to I-94. The driving and passing lanes have been monitored frequently since 1994.

Table 4.51 Standard HMA cells of the mainline MnROAD

Mainline cell	Surface layer		Base thickness (in)	Subbase thickness (in)	Roadbed soil type
	Thickness (in)	Binder grade (PG)			
1	6	58-28	33	0	Clay
2	6.1	58-28	4	28	Clay
3	6.3	58-28	4	33	Clay
4	9.1	58-28	0	0	Clay
14	10.9	58-28	0	0	Clay
15	11.1	62-22	0	0	Clay
16	8	62-22	28	0	Clay
17	7.9	62-22	28	0	Clay
18	7.9	62-22	12	9	Clay
19	7.8	62-22	28	0	Clay
20	7.8	58-28	28	0	Clay
21	7.9	58-28	23	0	Clay
22	7.9	58-28	18	0	Clay
23	9.2	58-28	4	3	Clay
50	6	58-28	0	0	Clay
51	6	58-28	0	0	Clay

The time-series pavement condition and distress data collected along the 14 mainline standard HMA pavement cells (cells 1 through 4 and 14 through 23) and the two mainline HMA overlay cells (cells 50 and 51) and the LDF data were used in the analyses and verification.

Although occasionally, the IRI data of the pavement of each cell were measured and recorded one to five times during the same day and the ranges of one day measurements for one cell vary from few inch per mile to as much as 121 to about 207 inch/mile. Although the ranges in the measurements on certain days were significant, not one single data point was labeled as an outlier or excluded from the analysis in this study. Nevertheless, the methodologies for estimating the time-series IRI and rut depths data of the passing lanes using the driving lane data were developed based on the data from cells 3, 4, and 14 through 23, while the data from cells 1, 2, 50, and 51 were used for verification. Unfortunately, very limited cracking data were found in the database to be included in the analysis as detailed in Subsection 4.6.3. Nevertheless, the developed methodologies and the results for IRI and rut depths are presented and discussed in the next two subsections.

4.6.1 IRI Model

The methodology for estimating the IRI of the passing lane using the measured time-series IRI data of the driving lane was developed using the following steps:

1. Each of the time-series IRI data measured along the driving and passing lanes of each of the mainline HMA cells of the MnROAD test facility were modeled using the exponential function stated in Equation 4.12. Figure 4.51 depicts the measured IRI data along the driving and passing lanes of cell 3. The solid curves in the figure are the best-fit curves that were obtained using Equation 4.12. The resulting equations and the regression parameters α and β for each lane are included in the figure.

$$\mathrm{IRI} = \alpha \exp(\beta t) \qquad \text{Equation 4.12}$$

where, IRI is the International Roughness Index;

α and β are regression parameters;

t is the elapsed time (year)

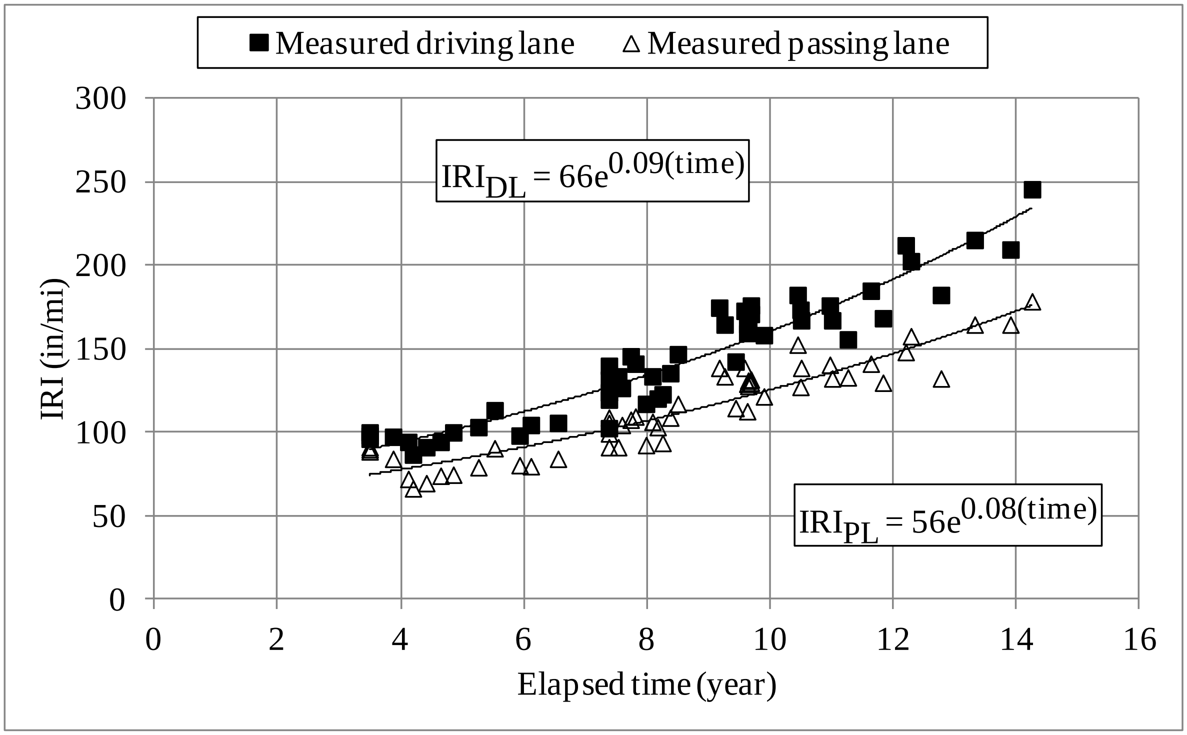


Figure 4.51 Best-fit curves for the measured IRI of the driving and passing lanes, cell 3

2. Step 1 was repeated for each of the twelve cells (cells 3 and 4 and 14 through 23) used in developing the methodology. The average values of each of the regression parameters α and β and for each of the driving and passing lanes of the twelve cells were calculated. These average values of each of the driving and passing lanes were then expressed as functions of the LDF (0.8 for the driving lane and 0.2 for the passing lane) using Equations 4.13 and 4.14, respectively. The parameters of the two equations were then calculated (a = 7.68, b = 34, c = 0.035, and d = 0.14) and used throughout the analyses.

$$\alpha_i = a(LDF_i) + b \qquad \text{Equation 4.13}$$

$$\beta_i = c(LDF_i) + d \qquad \text{Equation 4.14}$$

Where, i is either the driving or the passing lane;

α_i is the average regression parameter of either the driving or the passing lane;

β_i is the average regression parameter of either the driving or the passing lane;

LDF_i is the lane distribution factor for either the driving or the passing lane;

a, b, c, and d are the parameters of Equations 4.13 and 4.14

3. Equations 4.12, 4.13, and 4.14 were then combined to generate Equation 4.15, which is a general equation that can be used to estimate the time-series IRI data for any lane.

$$IRI = \left[\alpha_{DL}\left(\frac{a(LDF_i)+b}{a(LDF_{DL})+b}\right)\right] * \exp\left[\beta_{DL}\left(\frac{c(LDF_i)+d}{c(LDF_{DL})+d}\right)t\right] \qquad \text{Equation 4.15}$$

Where, IRI is the International Roughness Index;

α_{DL} and β_{DL} are the regression parameters of the driving lane;

LDF_{DL} is the lane distribution factor for the driving lane;

LDF_i, a, b, c, and d are as before

For each pavement project or in this case cell, the inputs to Equation 4.15 are the regression parameters α and β of the driving lane (α_{DL} and β_{DL}), the LDF of the driving lane (LDF_{DL}), and the LDF of any other lane including the passing lane (LDF_i). The equation modifies the values of the regression parameters α and β of the driving lane for any lane with known LDF. The possible LDF values to be used as inputs to Equation 4.15 include:

- LDF_i of the driving lane (0.8 for the MnROAD). In this scenario, the values of the regression parameter α and β of Equation 4.15 will be the same as those in Equation 4.12

(the best fit equation). Hence, the estimated IRI values would be the same as those estimated using Equation 4.12.

- LDF_i of the passing lane (0.2 for the MnROAD), this would yield estimates of the time-series IRI data of the passing lane as shown in Figure 4.52.

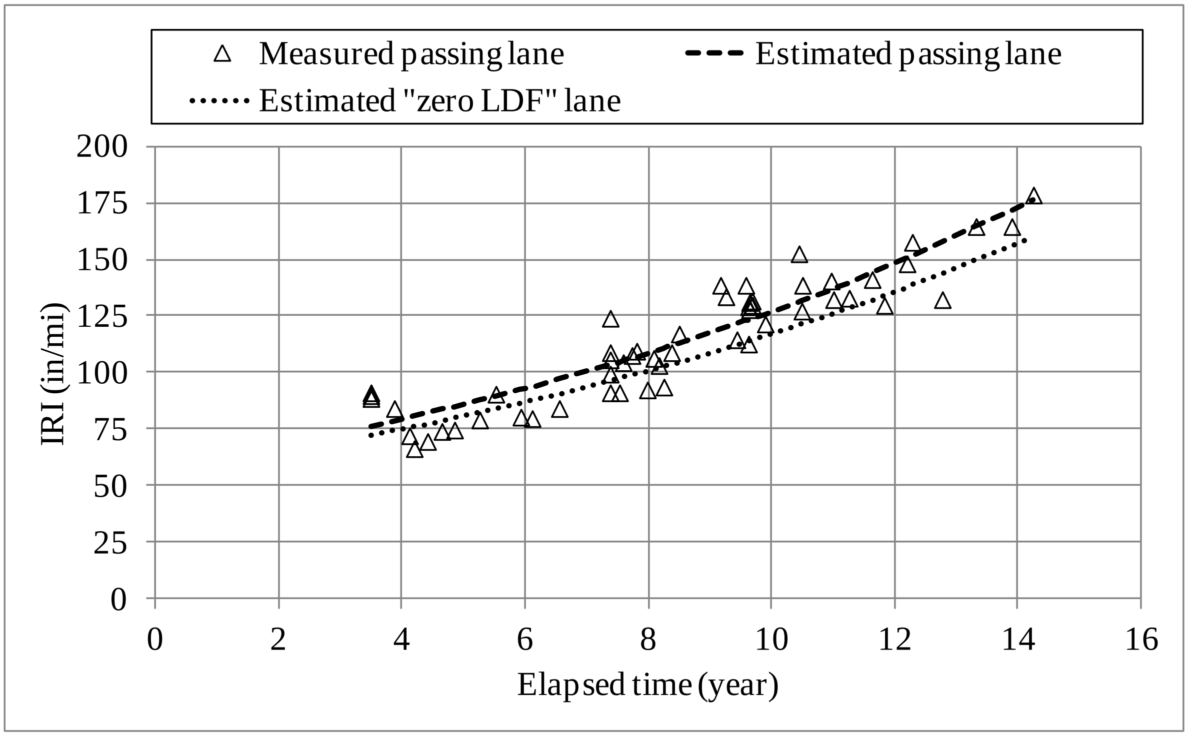


Figure 4.52 Measured IRI data along the passing lane and the estimated time-series IRI curves for passing and zero traffic lanes, cell 3

- LDF_i of the "zero-traffic" lane of 0.0, this would yield estimates of the time-series IRI of a lane with no traffic. Stated differently, Equation 4.15 would estimate the effects of the environment on IRI as shown in Figure 4.52. The open triangles in the figure indicate the measured passing lane time-series IRI data, while the dashed curve is the estimated time-series IRI data of the passing lane using Equation 4.15. Finally, the dotted curve in the figure is the estimated IRI curve for zero traffic. Once again, the data in the figure indicate that the environment has a significant impact on the IRI over time. The estimated curve for zero traffic lane in the figure indicate that the IRI values increase from about 55 to about 160 in/mi in 14 years. Whereas the measured IRI data of the passing lane (with 20 percent of the traffic) indicate that the IRI values increase from about 55 to less than 175 in/mile in 14 years. Indeed, this observation could also be deduced from Figure 4.51. The data in the figure indicate that the measured IRI values of the driving lane increase from about 65 to about 225 in/mile for an increase of 160 in/mile in 14 years. For the same time period, the measured IRI of the passing lane increase from about 55 to about 175 in/mi for an increase of 120 in/mile. According to the MnROAD data, the traffic along the driving lane is 4 times higher than the passing lane; the increase in the IRI in the driving lane of 160 in/mi is only about 35 percent higher than the increase in IRI along the passing lane of 120 in/mi. Once again, the implication of the measured data is that most of the increases in roughness or IRI values are not caused by the traffic. They are caused by the environment, which has significant effects on the measured IRI data.

4. The accuracy of the estimated IRI data was examined by plotting the estimated IRI values against the measured IRI values for the passing lane of all the cells used in developing Equation 4.15, as shown in Figure 4.53. The diagonal line in the figure is the line of equality indicating a perfect estimation of the measured data. The data in the figure indicate that the estimated IRI values are very similar to the measured data and the standard error of the estimates was only 16.8 in/mi, which is an insignificant fraction of the average measured data. This result was expected because the parameters of Equations 4.12 through 4.15 were obtained using the measured IRI values of the driving and passing lanes of those cells. The true test of the accuracy of the estimated values is presented in item 5 below. Nevertheless,

other statistical tests could be performed, such as the t test, to determine the goodness of fit between the measured and the estimated data.

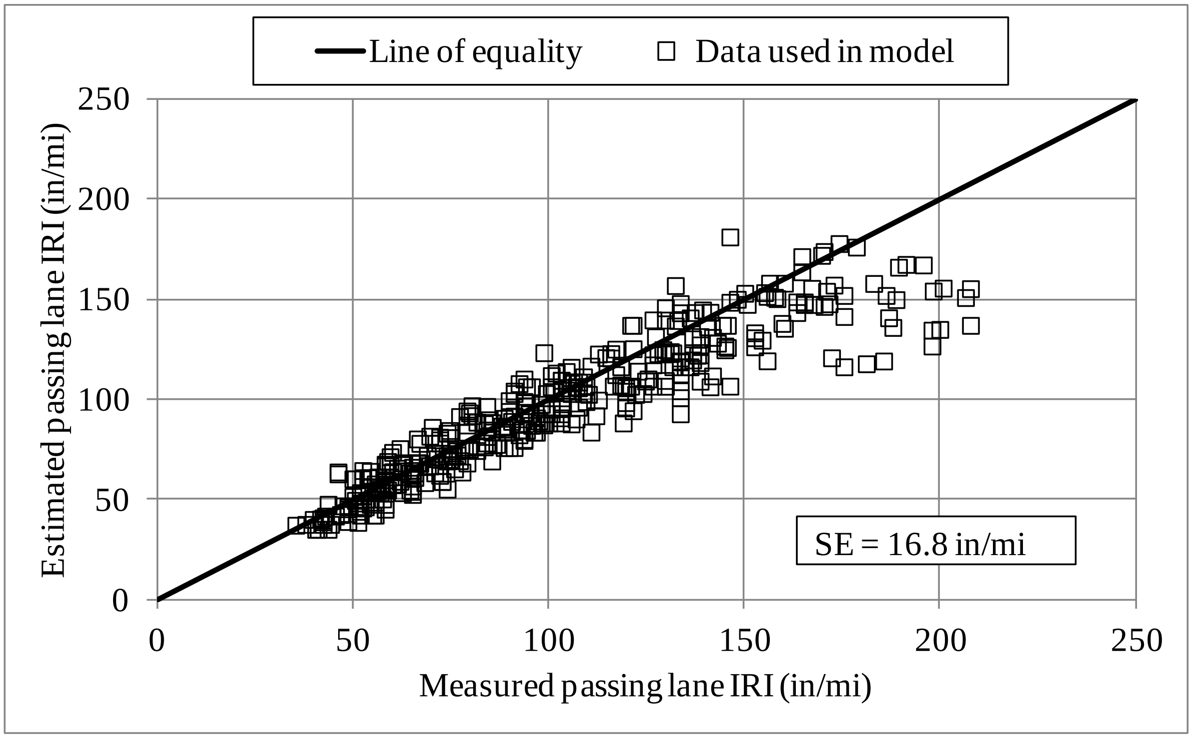


Figure 4.53 Measured vs. estimated IRI for cells used in developing the model

5. The applicability and accuracy of Equation 4.15 to the IRI data of the HMA pavements of cells 1 and 2 (the data were not used in developing the methodology) and to the HMA overlaid pavements along cells 50 and 51 (the data also were not used in developing the methodology) was also examined. The results are presented and discussed below.

   a. Equation 4.15 and the IRI data of the driving lane of cells 1 and 2 were used to estimate the IRI values of each of the corresponding passing lanes of the two cells. Further, the time-series IRI values for a zero traffic lane (LDF = zero) were also estimated using Equation 4.15 with LDF value of zero. As stated earlier, the latter estimates of IRI values represent the effects of the environment on the IRI. Nevertheless, the HMA pavement along the driving lane only of cell 1 was subjected to 1.5-inch inlay in 2006 while the passing lane was not. Figure 4.54 depicts the measured driving lane IRI data before and after the inlay and the best fit curves for each. The open triangles in the figure represent the measured time-series IRI data of the passing lane, while the dashed curve depicts the estimated IRI values. These estimated values were calculated using the regression parameters α and β of the driving lane data before treatment and LDF value of 0.2. The data in the figure indicate that the estimated IRI values of the passing lane are similar to the measured data with the standard error of the estimates of only 5 in/mi. Finally, the dotted curve in Figure 4.54 depicts the estimated time-series IRI value for a zero traffic lane (LDF = 0.0). Once again, the increase in the IRI due to zero traffic is significant and is mainly related to the environment.

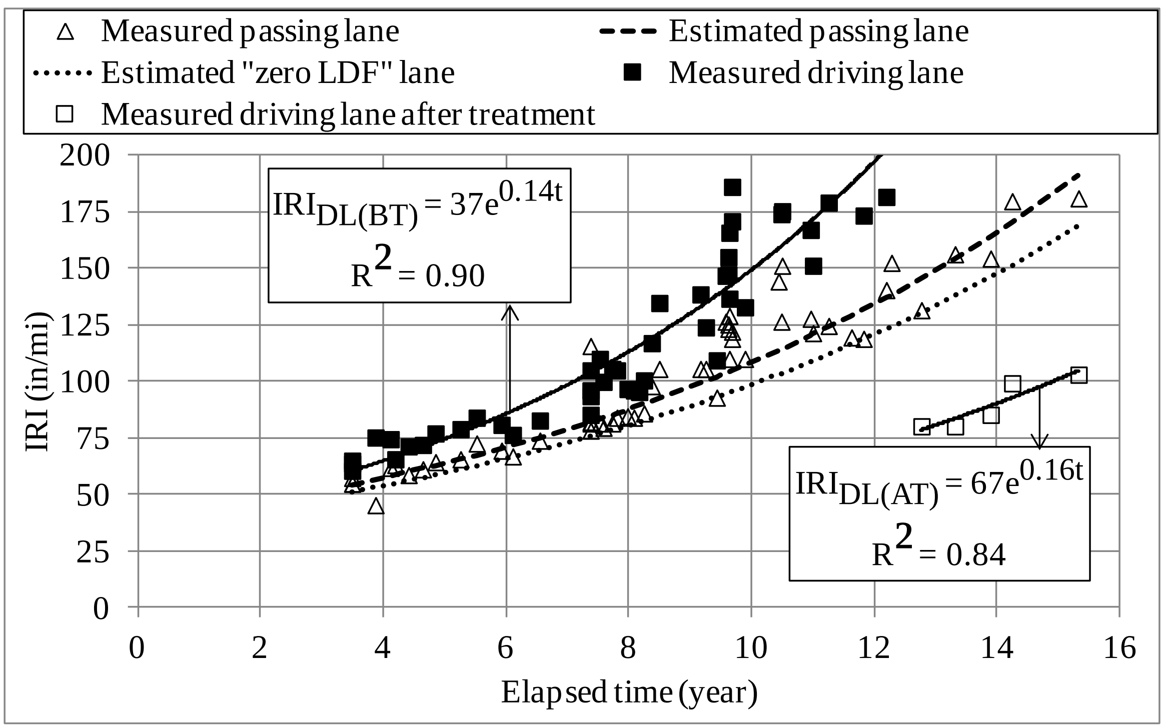


Figure 4.54 Measured driving lane IRI before and after 1.5-inch inlay and their best fit curves, measured and estimated IRI for passing lane, and estimated IRI for a zero traffic lanes, cell 1

Further, the open triangles in Figure 4.55 show the measured time-series IRI data of the passing lane of cell 2. Whereas the dashed curve indicates the estimated IRI value using Equation 4.15, the parameters α and β of the driving lane and LDF value of 0.8. Once again the estimated values are similar to the measured ones with a standard error of the estimates of only 10.2 in/mi. The dotted curve in the figure depicts the increase in IRI due to environmental factors (zero traffic lane).

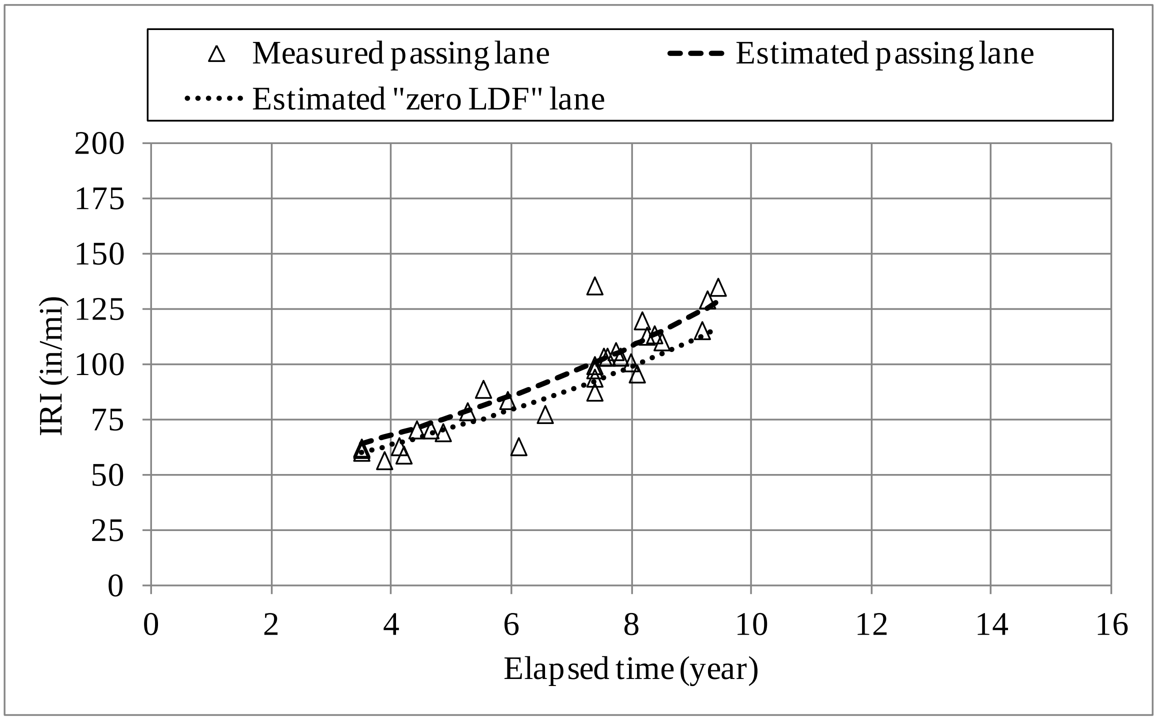


Figure 4.55 Measured and estimated IRI for passing lane and estimated IRI for a zero traffic lanes, cell 2

b. The pavements along the driving and passing lanes of cells 50 and 51 were subjected to HMA overlay in 1997, three years after construction. The IRI data of both lanes were measured since 1997 and were not used in developing the methodology. Equation 4.12 was used to model the measured time-series IRI data of the driving lane and to determine the regression parameters α and β. Equation 4.15, the driving lane parameters α and β, and LDF values of 0.2 and 0.0 were used to estimate the time-series IRI data of the passing lane and of a zero traffic lane, respectively. The estimated IRI values for the passing lane and for a zero traffic lane (LDF_i = zero) for cells 50 and 51 are shown in

Figures 4.56 and 4.57, respectively. The open triangles in both figures indicate the measured time-series IRI data of the passing lane, while the dashed and dotted curves are the estimated time-series IRI curves of the passing and of a zero traffic lane, respectively. The data in the figures indicate that the estimation is similar to the measured data with the standard error of the estimates of 16.9 and 13.9 in/mi for cells 50 and 51, respectively.

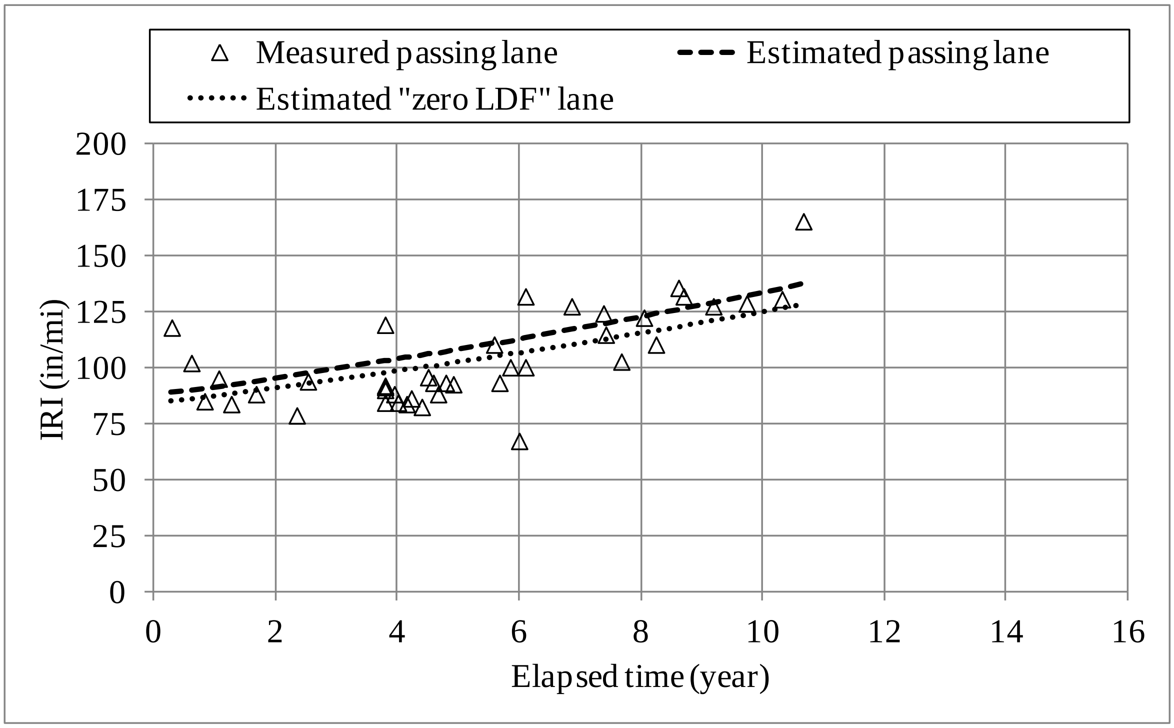


Figure 4.56 Measured and estimated time-series IRI curves for passing lane and estimated IRI of a zero traffic lane, cell 50

Finally, the measured and estimated IRI data for all cells, those used in developing Equation 4.15 and the verification cells are plotted in Figure 4.58. The diagonal line in the figure is the line of equality indicating a perfect estimation of the measured data. The data in the figure indicate that the estimated IRI values are very similar to the measured ones with an overall standard error of the estimates of only 15.9 in/mi.

Several important points should be noted regarding the development of the above presented methodology. These are:

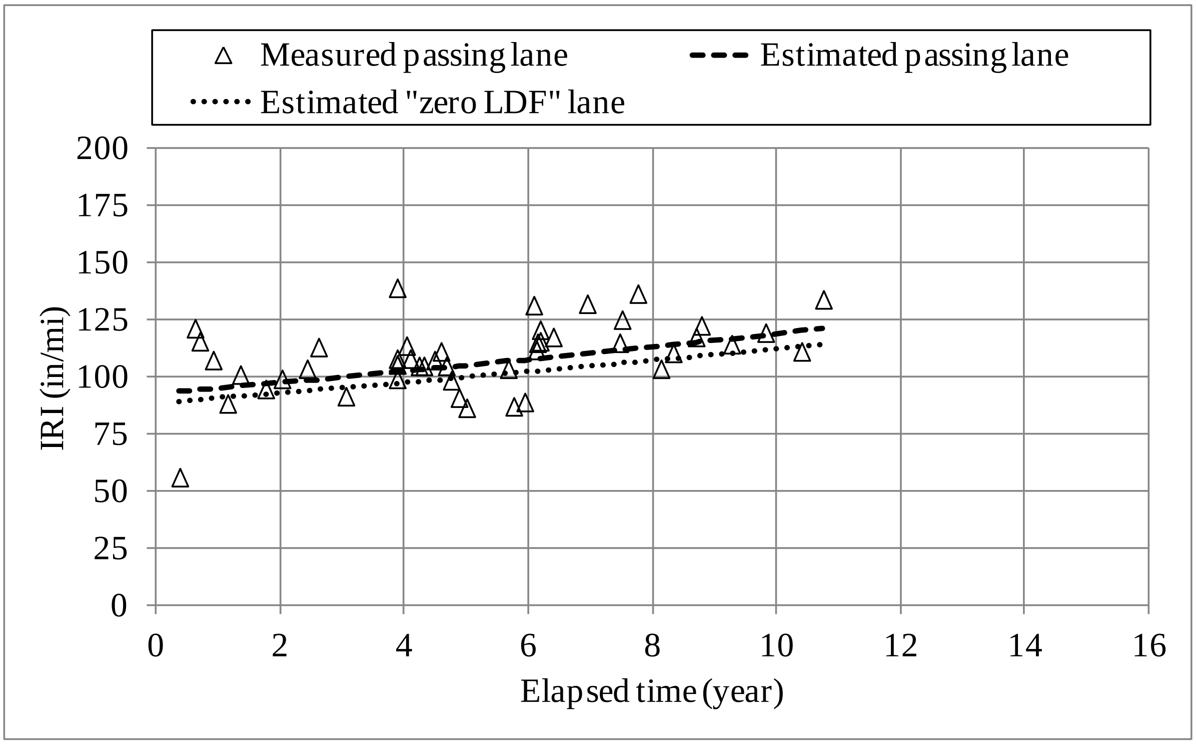


Figure 4.57 Measured and estimated time-series IRI curves for passing lane and estimated IRI of a zero traffic lane, cell 51

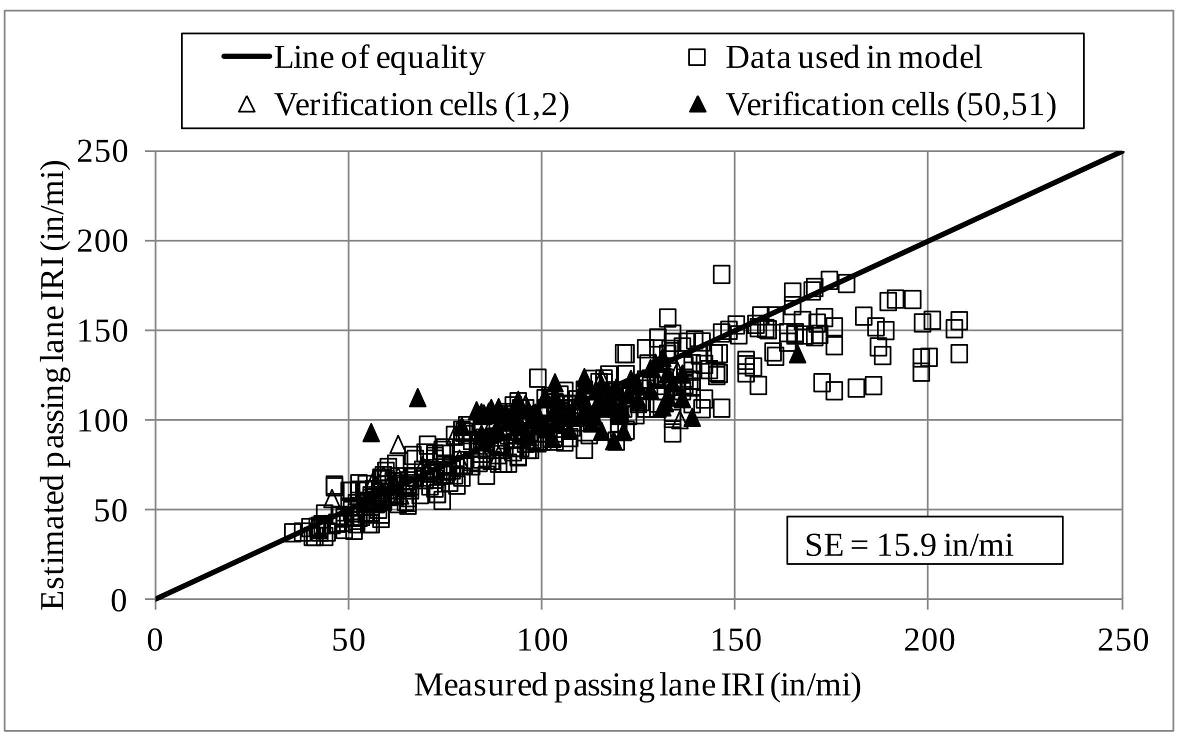


Figure 4.58 Measured vs. estimated IRI for all cells

1. Although the MnROAD test facility consists of HMA and rigid pavement cells, the data from only the HMA cells were used in the analysis. The rigid pavements did not exhibit much roughness (60 to 100 in/mi) and the differences between the measured IRI of the driving and passing lanes were insignificant as shown in Figures 4.59 through 4.61 for cells 5, 10, and 13. The closed squares and open triangles in the figures indicate the measured driving and passing lane IRI data, respectively. The almost equal IRI data of the two lanes could be due mainly to environmental and material factors and caused by curling and warping and traffic load repetitions have had negligible effects on the IRI of the rigid pavements of the MnROAD test facility. Hence, no estimation models were developed for the rigid pavement cells.

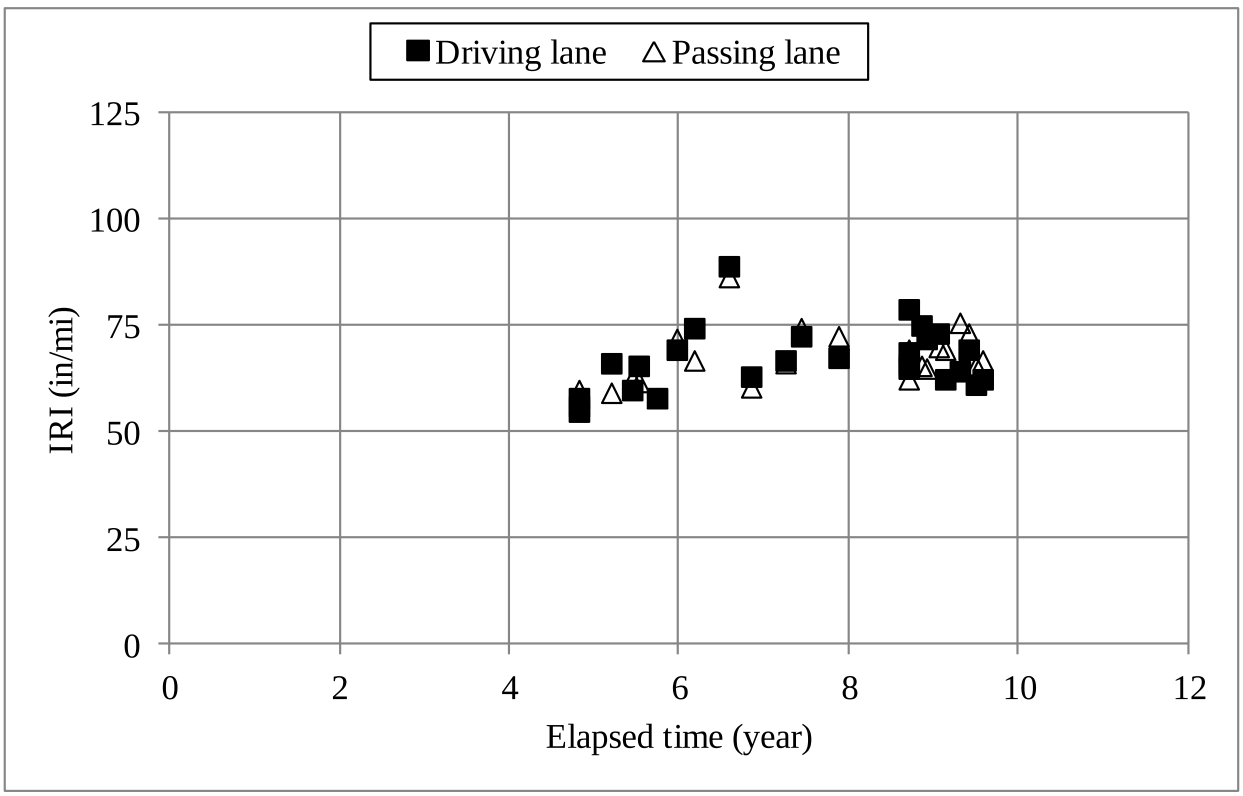


Figure 4.59 Measured time series IRI data along the driving and passing lanes, cell 5

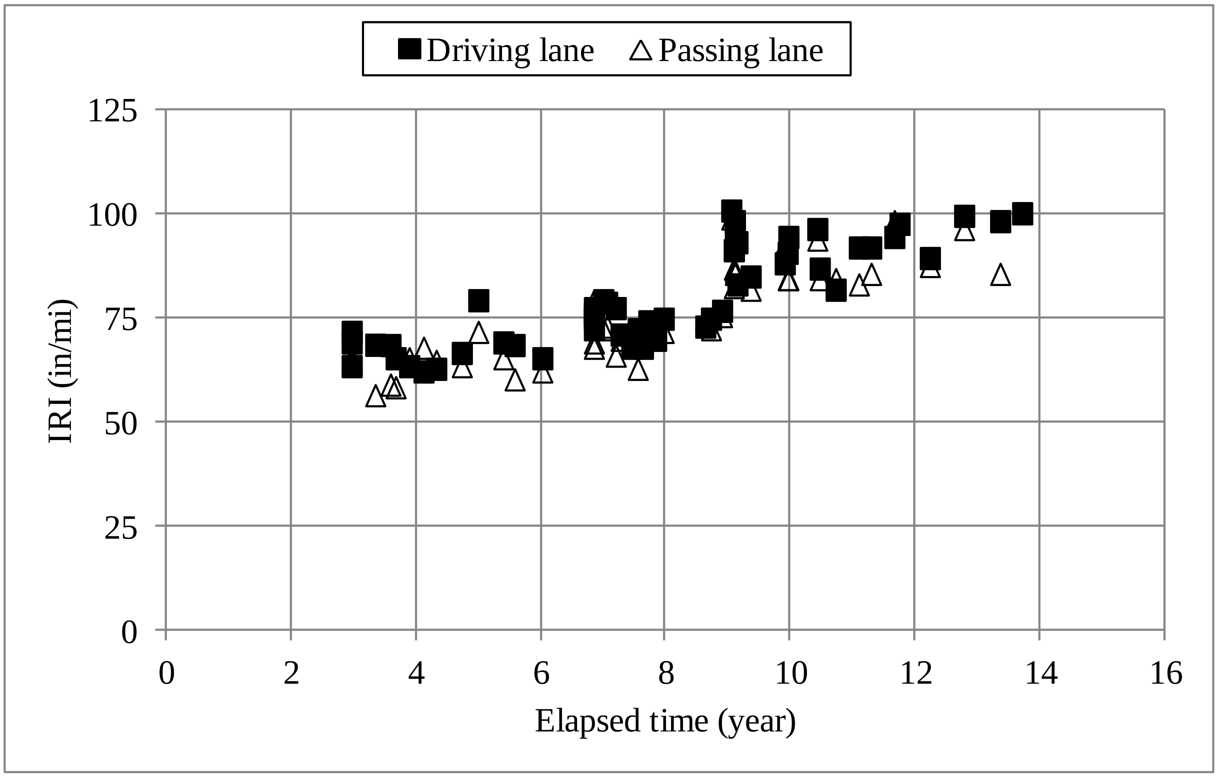


Figure 4.60 Measured time series IRI data along the driving and passing lanes, cell 10

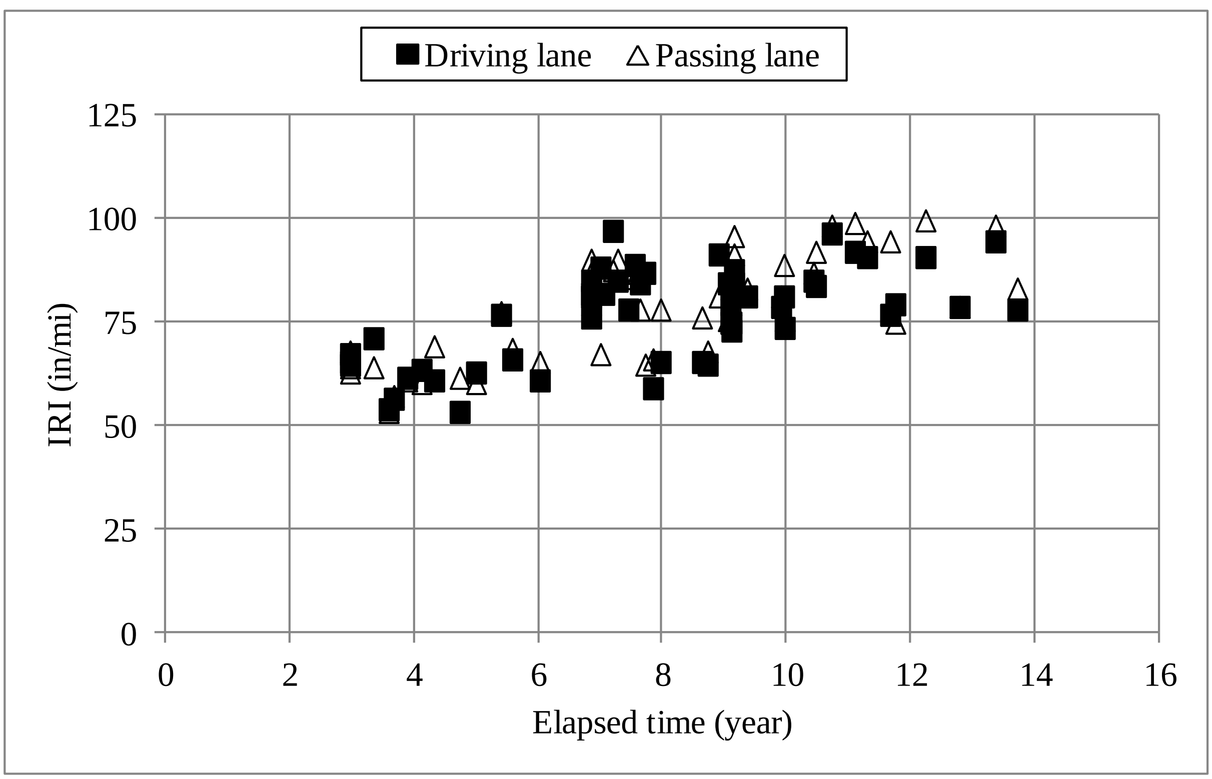


Figure 4.61 Measured time series IRI data along the driving and passing lanes, cell 13

2. The equipment used for measuring IRI along the MnROAD test facility were changed in 1997 (about 3 years after construction). In general, the IRI data collected using the old equipment are notably different from the IRI values measured by the new equipment, as shown in Figure 4.62 for cell 3. The closed squares and open triangles indicate the measured driving and passing lane time-series IRI data using the new and old equipment, respectively. Hence, the IRI data measured with the first set of equipment were not used in the analysis.

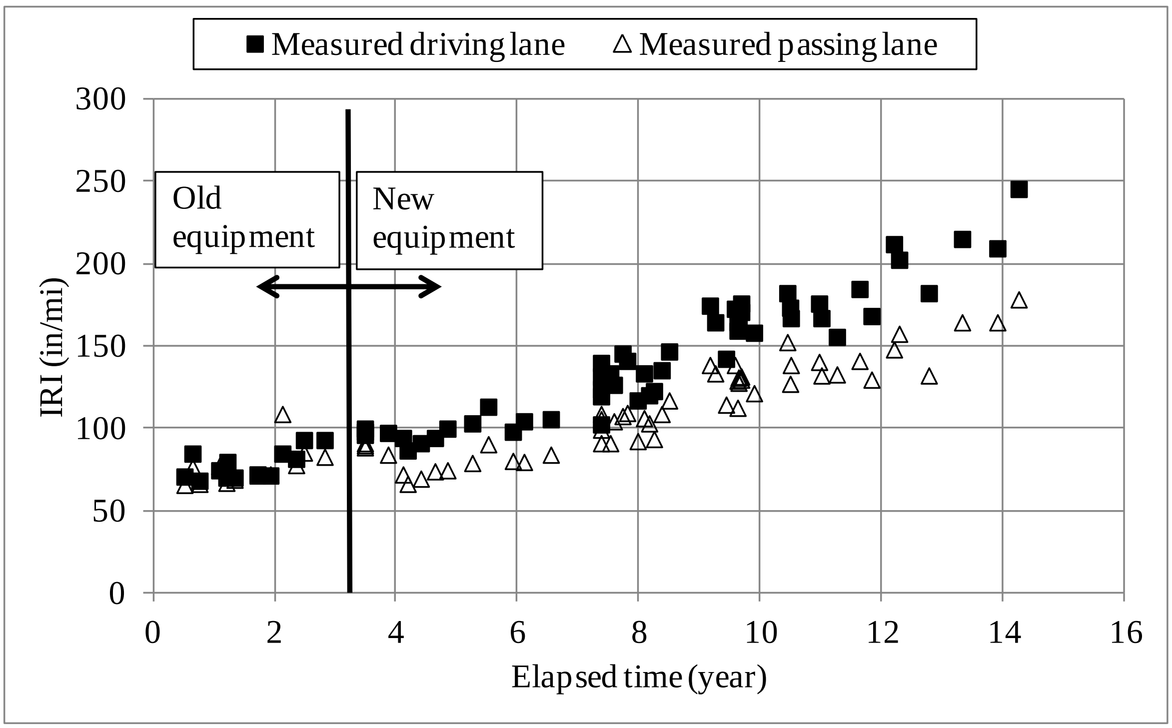


Figure 4.62 Measured IRI from cell 3 with old and new equipment

Nevertheless, SHAs could use the above method to estimate the time-series passing lane IRI data if the regression parameters (α and β) of Equation 4.15 of the measured IRI data of the driving lanes are known. However, the exact relationships between the regression parameters (α and β) and the LDF could vary from one pavement network to another. In this study, said relationship is assumed to be linear (only two data points were available; LDF = 0.8 and 0.2). For more than two lane roads, more than two values of the LDF should be available. Hence, the SHA could use the LDF data to verify or to calibrate the parameters a, b, c, and d of Equations 4.13 and 4.14 by occasionally collecting the IRI data along parts of the other lanes.

Any SHA or highway authority could use the developed methodology to estimate the IRI of multi-lane facilities provided that the agency has time-series IRI data from at least one lane (typically the driving lane) and LDF data.

The step-by-step procedure for the implementation of the methodology is detailed below.

1. Model the driving lane time-series IRI data using Equation 4.12, and determine the values of the regression parameters α and β of the driving lane.

2. Use Equation 4.16, with the proper regression parameters α_{DL} and β_{DL} of the driving lane and LDF values to calculate the time-series IRI data of the passing lanes. Equation 4.16 was obtained by substituting the values of the parameters a, b, c, and d of Equations 4.13 and 4.15 obtained in the above analysis into Equation 4.15. Note that, the parameter α_{DL} in Equation 4.16 denotes the initial IRI value at time zero after construction. Hence, if the initial IRI value (after construction) of the passing lane is known or is measured in the quality control program, the value could be used to replace the entire first term (the multiplier of the exponential function) of Equation 4.16.

$$IRI = \left[\alpha_{DL}\left(\frac{7.68(LDF_i)+34}{7.68(LDF_{DL})+34}\right)\right] * \exp\left[\beta_{DL}\left(\frac{0.035(LDF_i)+0.14}{0.035(LDF_{DL})+0.14}\right)t\right] \quad \text{Equation 4.16}$$

Where, IRI is the International Roughness Index;

α_{DL} and β_{DL} are the regression parameters of the driving lane;

LDF_i is the lane distribution factor for any lane;

LDF_{DL} is the lane distribution factor for the driving lane;

t is the elapsed time (year)

3. The values of the parameters a, b, c, and d were developed based on the driving and passing lanes IRI data measured at the MnROAD test facility. It is advisable for the user to calibrate the parameters by collecting and using limited passing lane IRI data. Alternatively, the users could collect detailed and comprehensive passing lane IRI data and develop an algorithm for estimating the passing lane IRI following the procedures outlined in this subsection.

It is important to note that the parameters α_{DL} and β_{DL} of Equation 4.16 should be obtained by using Equation 4.12 and the time-series IRI data of the driving lane adjacent to the passing lane whose IRI data are being estimated. The pavements of the two adjacent lanes should have similar structure. Otherwise, the assumption of similar material related pavement deterioration of adjacent lanes is violated when one lane (typically the driving lane) is subjected to treatment, yielding dissimilar pavement structures. However, in this scenario, the regression parameters α_{DL} and β_{DL} from the pavement condition and distress data collected in the driving lane prior to treatment may continue to be used to estimate the passing lane conditions and distresses until the passing lane is treated. This is illustrated in Figure 4.54 where the driving lane was subjected to treatment while the passing lane was not. The regression parameters of the driving lane that were calculated using the pavement condition data that were collected several years before treatment were used to estimate the passing lane conditions. Alternatively, if the passing lane is subjected to treatment prior to the driving lane the methodology will no longer apply. Finally, after all lanes have been treated such that the pavement structures of the two lanes are dissimilar, the methodology will no longer be applicable.

Further, the methodology is not specific to Interstate facilities and could be applied to any road with two or more lanes in one direction where the condition and distress data along one lane are collected. The factors affecting multi-lane facilities within city borders could be different from one lane to the next. Such factors include manholes, curbs and gutters, utility trenches, intersections, and so forth. Hence, the methodologies should be verified and/or calibrated before they are applied to multi-lane city streets. The resulting equations and procedures could be used to accurately estimate passing lane pavement conditions for any multi-lane facility.

4.6.2 Rut Depth Model

The methodology for estimating the rut depth of the passing lanes using the measured time-series rut depth data of the driving lane was developed using the following steps:

1. The time-series rut depth data measured along the driving and passing lanes of each HMA pavement of the mainline cells of the MnROAD test facility were modeled using the power function of Equation 4.17. Figure 4.63 depicts the measured rut depth data along the driving and passing lanes of cell 3. The solid curves in the figure are the best-fit curves that were obtained using Equation 4.17, and the values of the regression parameters γ and ω for each lane were determined and are included in the figure.

$$RD = \gamma \, t^{\omega} \qquad \text{Equation 4.17}$$

   Where, RD is the rut depth;

   γ and ω are regression parameters;

   t is the elapsed time (year)

2. Step 1 was repeated for the twelve cells used in developing the methodology (cells 3 and 4 and 14 through 23). After obtaining the values of the parameters γ and ω of each cell, the average values of the driving and passing lanes were then calculated. The average values of γ and ω of the driving and passing lanes were then expressed as functions of the LDF (0.8 for the driving lane and 0.2 for the passing lane) using Equations 4.18 and 4.19, respectively.

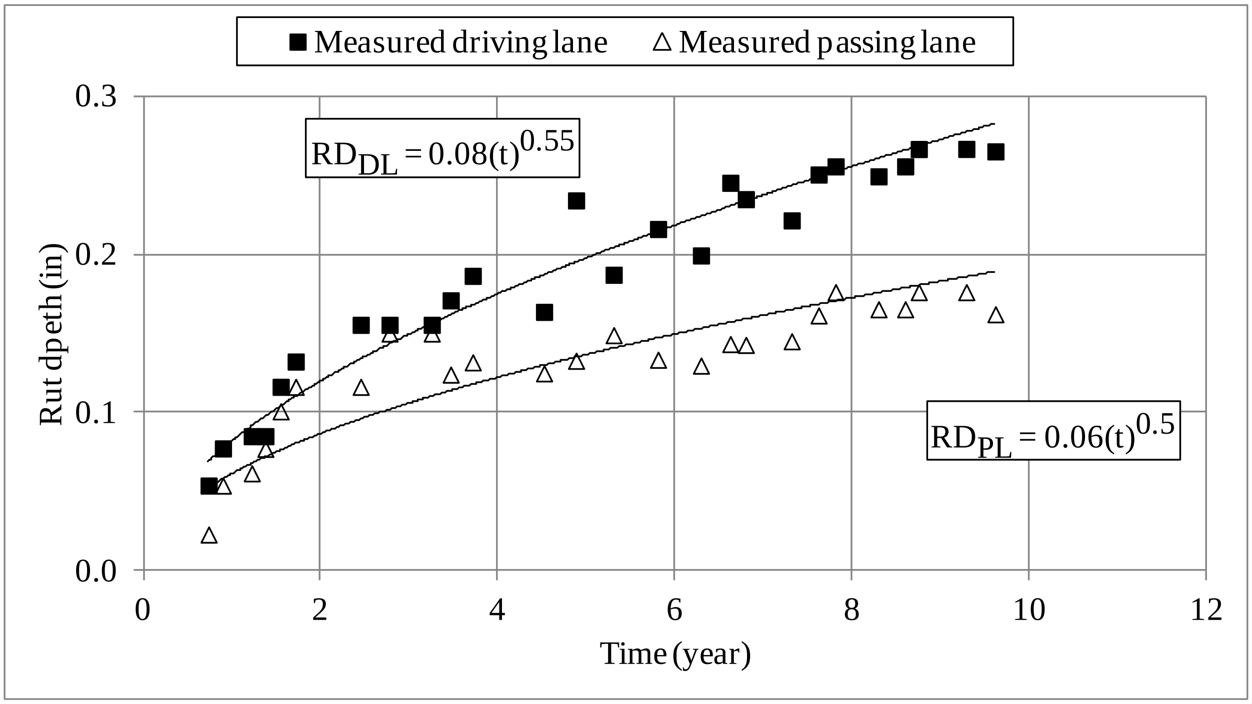


Figure 4.63 Measured time-series rut depth data along the driving and passing lanes and the best-fit curves, cell 3

Since rutting is a function of axle loads and the numbers of their repetition, the rut depth would be zero for a zero traffic lane. Hence, the power function of Equation 4.18 was used to model the parameter γ as a function of three LDF values (0.8, 0.2, and 0.0). Such power function passes through the origin where the value of γ and LDF are zero (which implies zero rut depth corresponds to zero traffic). The ω parameter, on the other hand, was modeled using the linear function of Equation 4.19. Finally, the parameters of Equations 4.18 and 4.19 (e, f, g, and h) were determined.

$$\gamma_i = e\left(LDF_i\right)^f \qquad \text{Equation 4.18}$$

$$\omega_i = g\left(LDF_i\right) + h \qquad \text{Equation 4.19}$$

Where, i is either the driving or the passing lane;

γ_i is the average regression parameter of either the driving or the passing lane;

ω_i is the average regression parameter of either the driving or the passing lane;

LDF_i is the lane distribution factor for either the driving or the passing lane;

e, f, g, and h are regression parameters

3. Equations 4.17, 4.18, and 4.19 were then combined to obtain the rut depth estimation model shown in Equation 4.20 for any LDF value. The inputs to Equation 4.20 are the γ and ω parameters of the driving lane (γ_{DL} and ω_{DL}) and the LDF of the driving and the other lanes (LDF_{DL} and LDF_i, respectively). The equation modifies the values of the parameters γ and ω of the driving lane for any lane with known LDF value. Note that if the LDF_i is assigned a value of zero, the equation would estimate the rut depth for a zero traffic lane, which is zero rut depth at any time.

$$RD = \left[\gamma_{DL}\left(\frac{e(LDF_i)^f}{e(LDF_{DL})^f}\right)\right] * t^{\left[\omega_{DL}\left(\frac{g(LDF_i)+h}{g(LDF_{DL})+h}\right)\right]} \qquad \text{Equation 4.20}$$

Where, RD is rut depth in inches;

LDF_i is the lane distribution factor (LDF = 0.0 to 1.0);

LDF_{DL} is the lane distribution factor of the driving lane;

γ_{DL} and ω_{DL} are the regression parameters of the driving lane;

LDF_i, e, f, g, and h are as before

4. The time-series rut depth data of the passing and zero traffic lanes was estimated using Equation 4.20 and LDF_i values of 0.2 and zero, respectively. The estimated rut depths for

cell 3 are plotted against time in Figure 4.64. The open triangles in the figure indicate the measured passing lane time-series rut depth data, the dashed and the dotted curves are the estimated time-series rut depths curves for the passing lane and for a zero traffic lane, respectively. Note that the driving lane rut depths could be estimated using Equation 4.20 as well; the estimation however would be equivalent to fitting the measured data to the model (γ and ω are known).

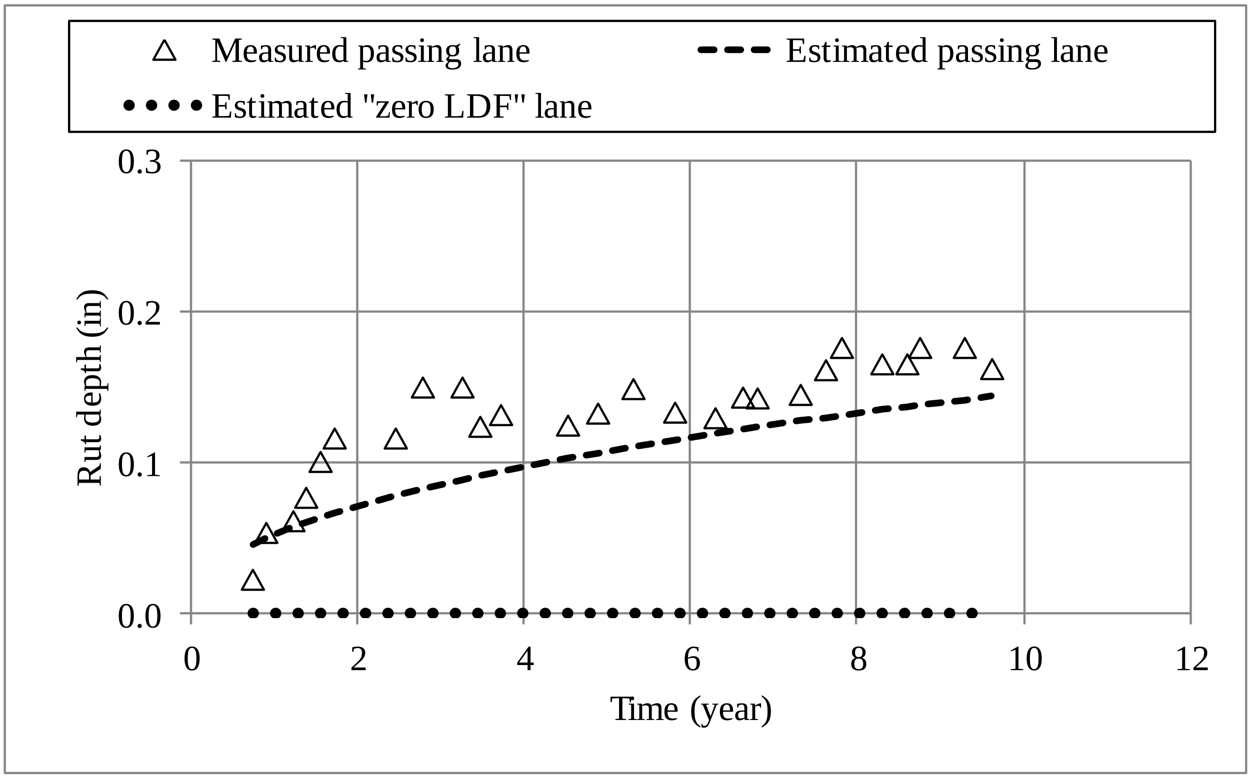


Figure 4.64 Measured and estimated rut depth curves for passing and zero traffic lanes, cell 3

5. The accuracy of the estimated rut depths data was checked by plotting the estimated rut depths versus the measured rut depths data for the passing lane of all the cells used in developing Equation 4.20, as shown in Figure 4.65. The diagonal line in the figure is the line of equality between the two data sets. The data in the figure indicate that the estimated rut depth values are similar to the measured ones with the standard error of the estimates of only

0.04 inch. This result was expected because the parameters of Equations 4.17 through 4.20 were obtained using the measured rut depth values of the driving and passing lanes of those cells. The true test of the accuracy of the estimated values is presented in item 6 below.

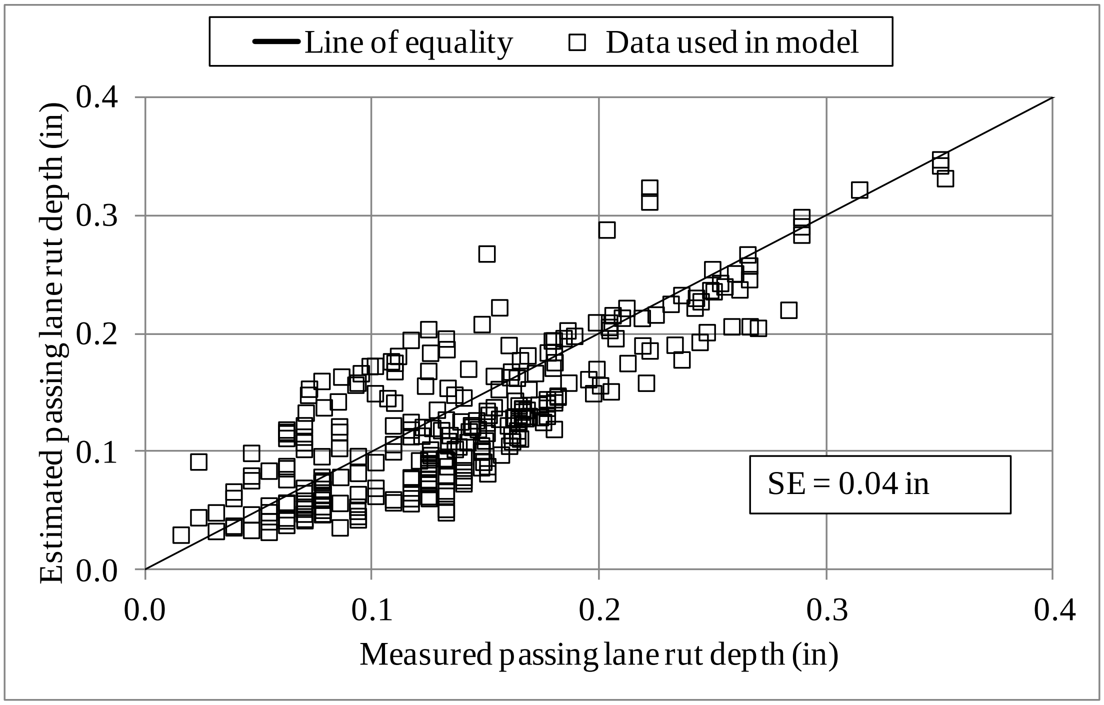


Figure 4.65 Measured vs. estimated rut depth data for cells used in developing the model

6. The accuracy of the estimated rut depth values, and hence of Equation 4.20, was further scrutinized as discussed below.

   a. Recall that the rut depth data of cells 1 and 2 were not included in the development of the model. Thus, Equation 4.20 and the rut depth data of the driving lane of each of the two cells were used to estimate the rut depth values of each of the corresponding passing lanes of the two cells. Further, the time-series rut depth values for a zero traffic lane (LDF = zero) were also estimated. As stated earlier, the rut depth is a function of traffic load repetitions and hence is always zero for a LDF value of zero. The estimated time-series rut depth values for the passing lane and for a zero traffic lane are shown in Figures

4.66 and 4.67 for cells 1 and 2, respectively. The open triangles in the figures indicate the measured time-series rut depth data of the passing lane, while the dashed and dotted curves are the estimated time-series rut depth curves of the passing and zero traffic lanes, respectively. The data in Figures 4.66 and 4.67 indicate that the estimated rut depth curve represents the measured rut depth data very well with the standard errors of the estimates of only 0.026 and 0.011 inch, respectively.

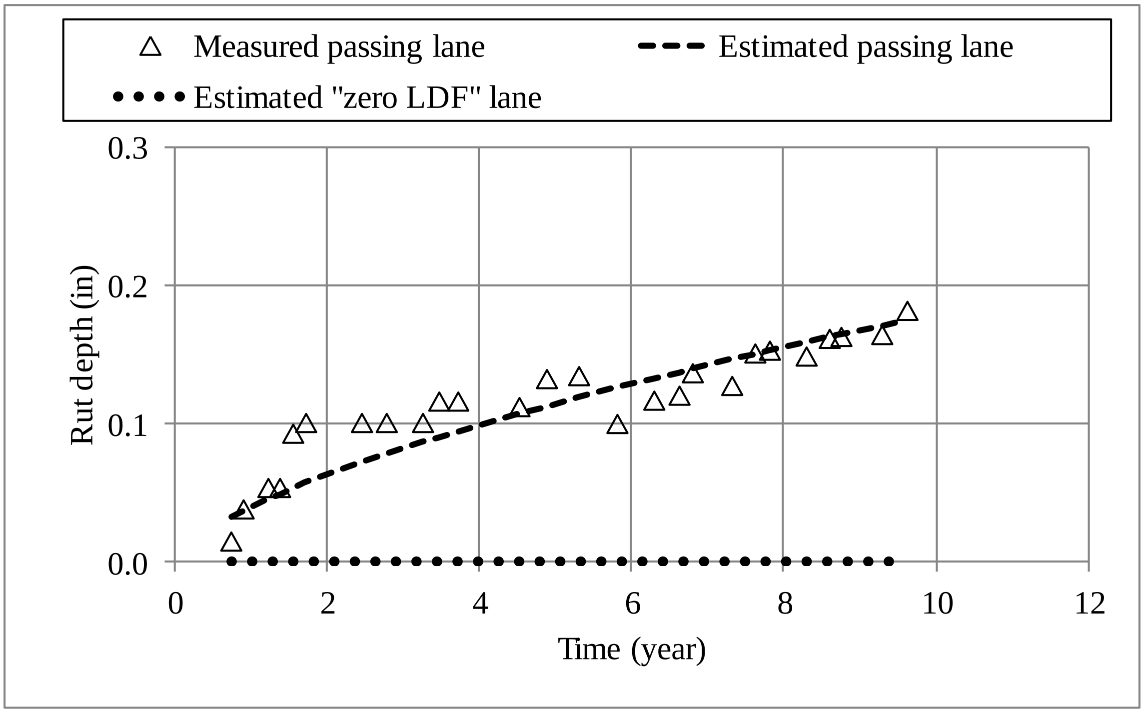


Figure 4.66 Measured and estimated rut depth curves for passing and zero traffic lanes, cell 1

b. In addition, the time-series rut depth data for the passing lanes of cells 50 and 51 were also estimated using Equation 4.20 and the driving lane data for each cell. The pavements of the two cells were subjected to HMA overlay in 1997 and the rut depth data were measured since then. The estimated rut depth values for the passing lane and for a zero traffic lane (LDF = zero) are shown in Figures 4.68 and 4.69 for cells 50 and 51, respectively. The open triangles in the figures indicate the measured time-series rut depth

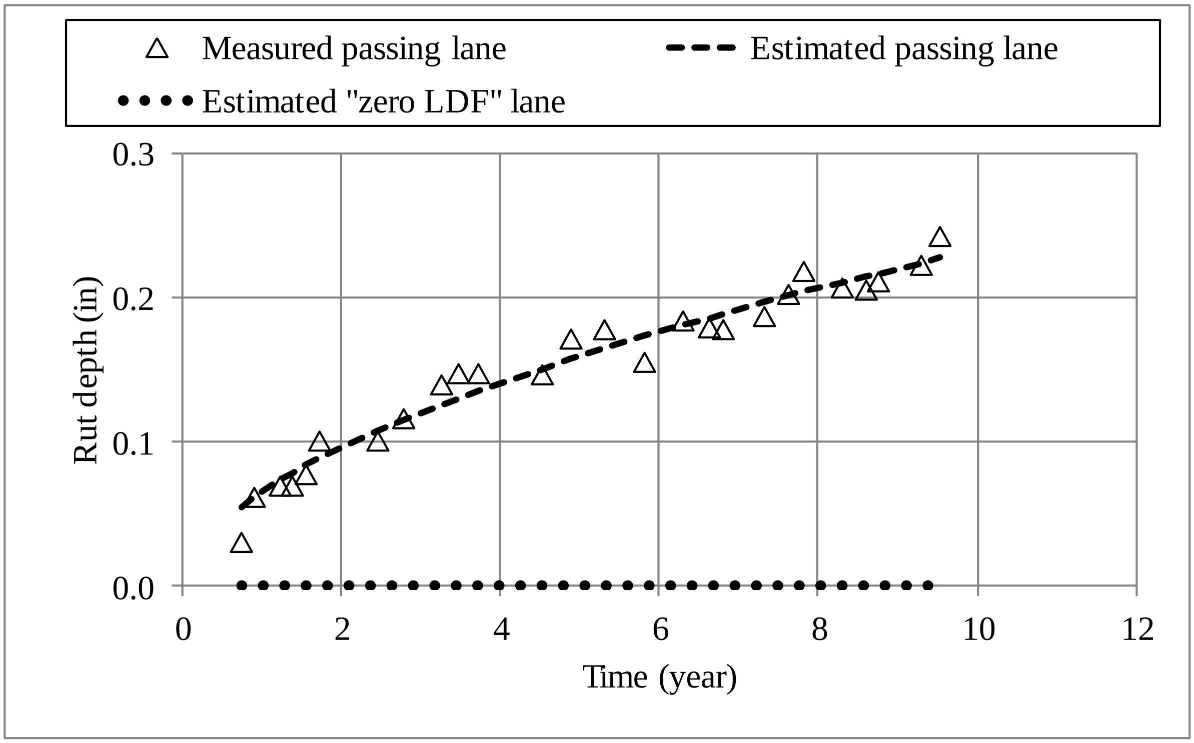


Figure 4.67 Measured and estimated rut depth curves for passing and zero traffic lanes, cell 2

data of the passing lane. The dashed and dotted curves are the locus of the estimated (using Equation 4.20) time-series rut depth of the passing and zero traffic lanes, respectively. The data in Figures 4.68 and 4.69 indicate that the estimated curves for the passing lanes simulate the measured rut depth data very well with the standard errors of the estimates of 0.018 and 0.017 inch, respectively.

c. Finally, the measured and estimated rut depths data for all cells, those used in developing Equation 4.20 and the verification cells are plotted in Figure 4.70. The diagonal line in the figure is the line of equality indicating a perfect estimation of the measured data. The data in the figure indicate that the estimated rut depth values are similar to the measured ones with a standard error of the estimates of only 0.037 inch.

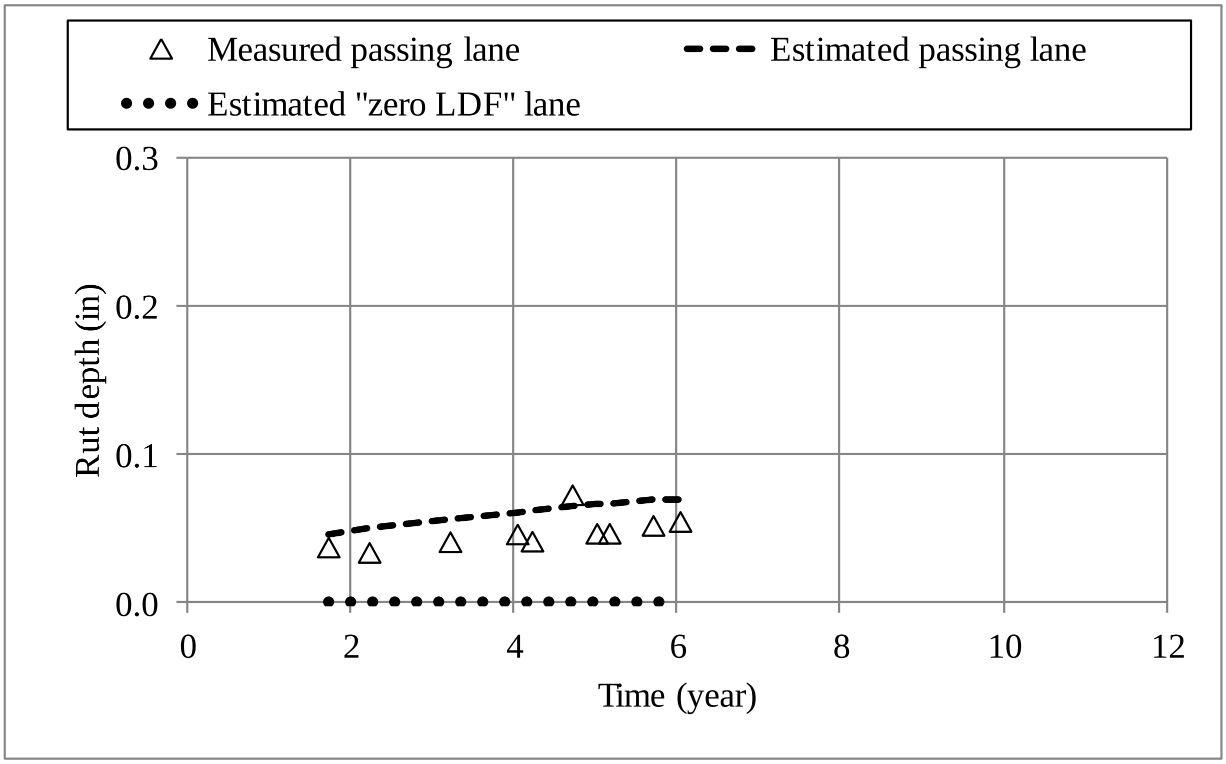


Figure 4.68 Measured and estimated rut depth curves for passing and zero traffic lanes, cell 50

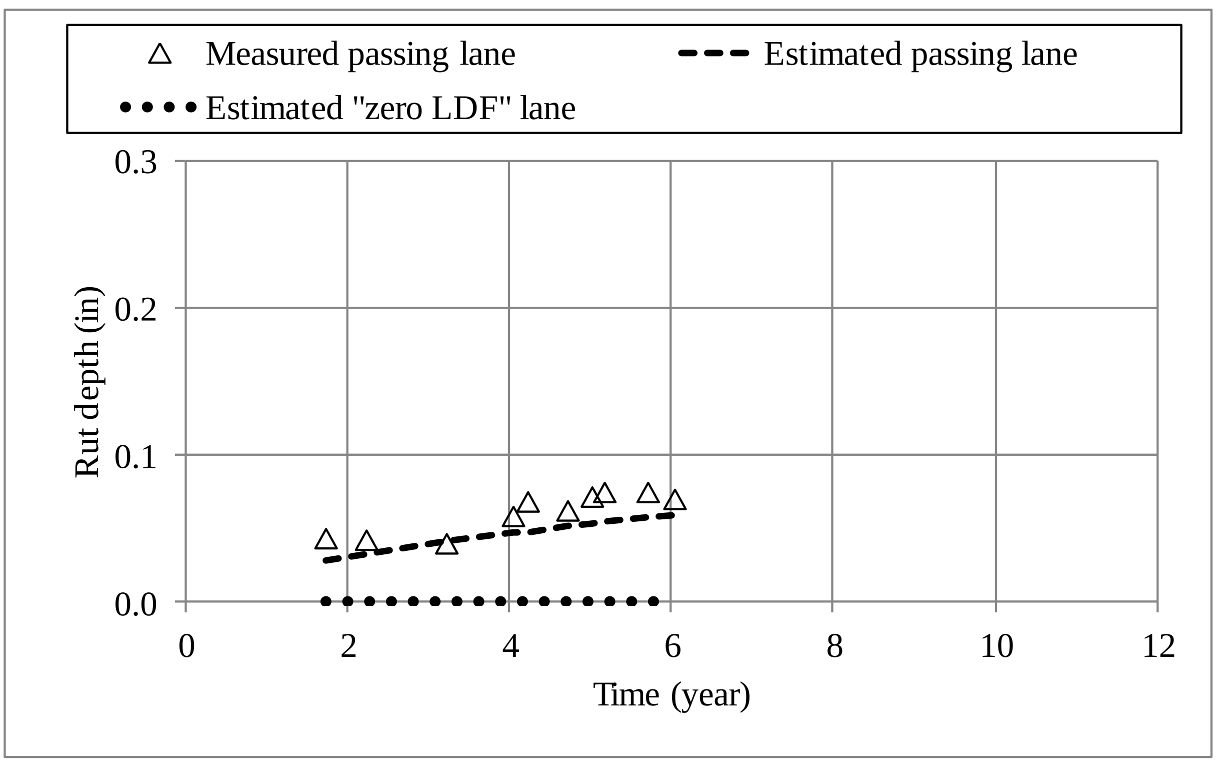


Figure 4.69 Measured and estimated rut depth curves for passing and zero traffic lanes, cell 51

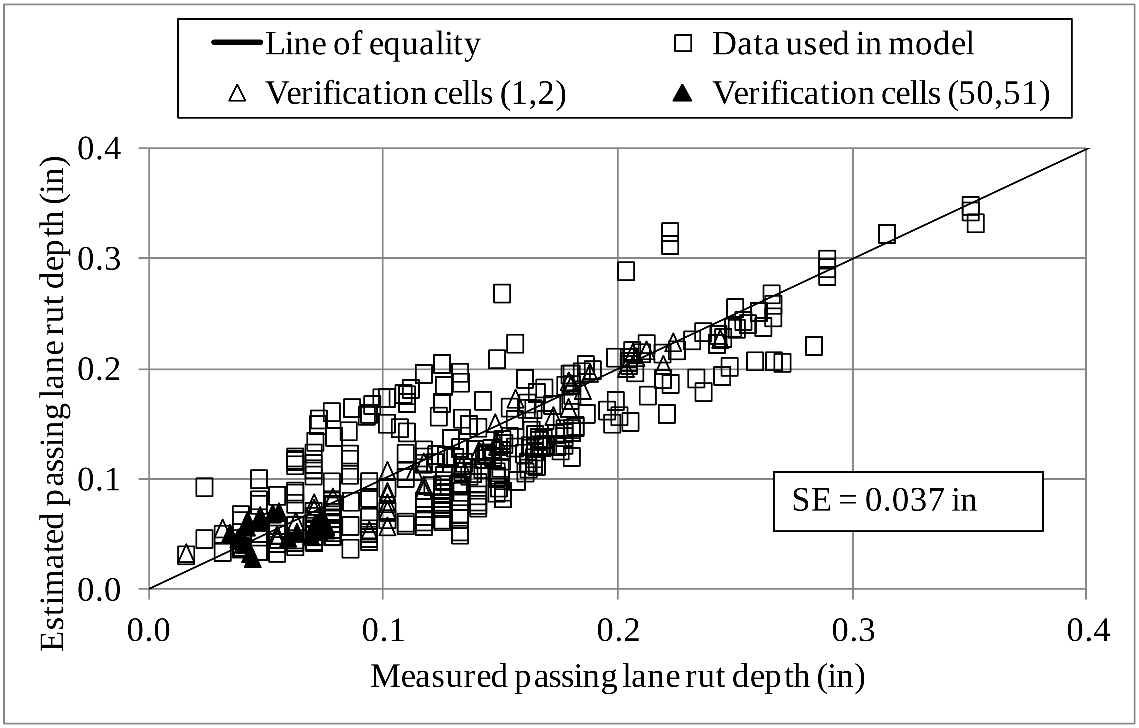


Figure 4.70 Measured vs. estimated rut depth for all cells

Nevertheless, SHAs could use the above method to estimate the time-series passing lane rut depth data if the regression parameters (γ and ω) of Equation 4.17 of the measured rut depth data of the driving lanes are known. However, the exact relationships between the regression parameters (γ and ω) and the LDF could vary from one pavement network to another. For more than two lane roads, more than two values of the LDF should be available. Hence, the SHA could use the LDF data to verify or to calibrate the parameters e, f, g, and h of Equations 4.18 and 4.19.

Any SHA or highway authority could use the developed methodology to estimate the rut depth of multi-lane facilities provided that the agency has time-series rut depth data from at least one lane (typically the driving lane) and LDF data.

The step-by-step procedure for the implementation of the methodology is detailed below.

1. Model the driving lane time-series rut depth data using Equation 4.17, and determine the values of the regression parameters γ and ω of the driving lane.

2. Use Equation 4.21, with the proper regression parameters γ_{DL} and ω_{DL} of the driving lane and LDF values to calculate the time-series rut depth data of the passing lanes. Equation 4.21 was obtained by substituting the values of the parameters e, f, g, and h of Equations 4.18 and 4.19 obtained in the above analysis into Equation 4.20.

$$RD = \left[\gamma_{DL} \left(\frac{0.32(LDF_i)^{0.11}}{0.32(LDF_{DL})^{0.11}} \right) \right] * t^{\left[\omega_{DL} \left(\frac{0.21(LDF_i)+0.52}{0.21(LDF_{DL})+0.52} \right) \right]} \qquad \text{Equation 4.21}$$

   Where, RD is the rut depth;

   γ_{DL} and ω_{DL} are the regression parameters of the driving lane;

   LDF_i is the lane distribution factor for any lane;

   LDF_{DL} is the lane distribution factor for the driving lane;

   t is the elapsed time (year)

3. The values of the parameters e, f, g, and h were developed based on the driving and passing lanes rut depth data measured at the MnROAD test facility. It is advisable for the user to calibrate the parameters by collecting and using limited passing lane rut depth data. Alternatively, the users could collect detailed and comprehensive passing lane rut depth data and develop an algorithm for estimating the passing lane rut depth following the procedures outlined in this subsection.

It is important to note that the parameters γ_{DL} and ω_{DL} of Equation 4.21 should be obtained by using Equation 4.17 and the time-series rut depth data of the driving lane adjacent to the passing lane whose rut depth data are being estimated. The pavements of the two adjacent lanes should have similar structure. Otherwise, the assumption of similar material related pavement deterioration of adjacent lanes is violated when one lane (typically the driving lane) is subjected to treatment, yielding dissimilar pavement structures. However, in this scenario, the regression parameters γ_{DL} and ω_{DL} from the pavement condition and distress data collected in the driving lane prior to treatment may continue to be used to estimate the passing lane conditions and distresses until the passing lane is treated. Alternatively, if the passing lane is subjected to treatment prior to the driving lane the methodology will no longer apply. Finally, after all lanes have been treated such that the pavement structures of the two lanes are dissimilar, the methodology will no longer be applicable.

Further, the methodology is not specific to Interstate facilities and could be applied to any road with two or more lanes in one direction where the condition and distress data along one lane are collected. The factors affecting multi-lane facilities within city borders could be different from one lane to the next. Such factors include manholes, curbs and gutters, utility trenches, intersections, and so forth. Hence, the methodologies should be verified and/or calibrated before they are applied to multi-lane city streets. The resulting equations and procedures could be used to accurately estimate passing lane pavement conditions for any multi-lane facility.

4.6.3 Pavement Cracking

Although the MnROAD database also includes data for alligator, longitudinal, transverse, and block cracking; unfortunately, the data are very limited and express insignificant cracking. Hence, the cracking data were not included in the analyses. If enough cracking data were available, similar analyses as those presented in subsections 4.6.1 and 4.6.2 could be carried out. It should be noted that transverse cracks in HMA pavements are typically caused by environmental effects on the pavement materials (also known as temperature cracks due to contraction and expansion) and are, in general, not load related. Therefore, the transverse cracking length should be similar in both the driving and passing lanes as the cracks will propagate the entire width of the pavement. The severity of the transverse cracks may be worse in the driving lane due to the increased traffic load repetitions but the occurrence should be almost equal. Similarly, block cracking is caused by oxidation and aging of the pavement surface and is not load related. Therefore, the block cracking area should be almost equal in the driving and passing lanes.

4.7 Hypotheses Verification

The first hypothesis detailed in section 4.2 was partially verified by the analyses and discussions presented in this chapter. The effectiveness of thin HMA overlay of asphalt surfaced pavement, thick HMA overlay of asphalt surfaced pavement, single chip seal, double chip seal, thin mill and fill, thick mill and fill were determined in three states based on the BT condition states. The results were used to populate convenient T^2Ms. Methodology and selection tables were developed to assist in the identification of candidate pavement treatment types based on the pavement conditions and their causes. Algorithms were developed to estimate the pavement conditions of multi-lane facilities. Methodologies were developed to use the information in the T^2Ms to analyze pavement network treatment strategies and to select the most cost-effective strategies and pavement treatment type, time, and project boundaries.

Unfortunately, the availability of detailed pavement treatment cost data was extremely limited. To fully verify the hypothesis, detailed cost data for each 0.1 mile long segment of the analyzed pavement sections are required. Nevertheless, the available total cost data were used in subsection 4.5.9 to show relationships between the ratio of treatment cost to treatment benefit and the application timing (condition states) of each of the six treatments. This data could be used with the objective function to determine the most cost-effective pavement treatment types, timing, and project boundaries. The methodologies and data analyses included in this dissertation support the selection of effective pavement treatments and treatment strategies, and provide the framework for the selection of cost-effective pavement treatments if the detailed cost data become available in the future.

The second hypothesis detailed in section 4.2 was verified by the analyses and discussions presented in section 4.6 of this chapter. New methodologies were developed and verified to use measured driving lane IRI and rut depth data along with the LDF values to estimate the IRI and rut depths of passing and zero traffic lanes. Further, the steps in developing the methodology and suggestions for SHA implementation were provided.

CHAPTER 5

IMPLEMENTATION GUIDELINES AND PROCEDURES FOR THE SELECTION OF COST-EFFECTIVE PAVEMENT TREATMENT TYPE, TIME, AND PROJECT BOUNDARIES

5.1 Foreword

This chapter presents step-by-step guidelines, procedures, and some recommendations for the selection of cost-effective pavement treatment type, time, and project boundaries. The guidelines and procedures are generic in nature and the interested highway authorities (HAs) should tailor them to its needs, organizational structure, goals, policies, and objectives. They could be implemented by HAs who have pavement management system (PMS) in place and by those who decided to start PMS. The formers could use the guidelines and procedures to perhaps improve their PMS, whereas the latter to build the foundation for a PMS. The guidelines and procedures are intended to assist the HAs to accomplish their goals and objectives. They could be fully or gradually implemented depending on the desire, resources, and constraints of the agencies. Therefore, the guidelines and procedures are divided into the ten topics listed below and detailed in the next section.

1. Pavement Condition and Distress Data Collection Policy
2. Pavement Condition and Distress Data Collection
3. Pavement Condition and Distress Data Analysis and Treatment Transition Matrices (T^2Ms)
4. Pavement Treatment Strategy Analysis
5. Identification of Candidate Project Boundaries
6. Forensic Investigation
7. Selection of Treatment Types

8. Calculation of Treatment Cost and Benefits
9. Selection of the Preferred Set of Pavement Projects
10. Study and Investigate Pavement Performance

At the outset, it cannot be overemphasized that, for all HAs, the success of establishing, or improving the existing, PMS operation is mainly a function to the degree of commitment of their administrators as well as their employees to continuously use and improve the system. It cannot be overemphasized that the main objectives of a successful PMS are:

1. Enhance the intra agency communications by providing electronic mean to share data and information and hence to decrease existing turf problems.
2. Plan, maintain, and/or improve the health of the pavement asset in cost-effective manners based on pavement needs.
3. Improve the capability and the state-of-the-practice through the feedback loop where the impacts and outcomes of the past practice could be analyzed and improved.
4. Allow the employees of the HAs to learn from each other's experience by documenting the problems they faced and the solutions they made and whether or not the outcomes were successful and cost-effective.

5.2 Step-by-Step Guidelines and Procedures Leading to the Selection of Pavement Treatment Type, Time, and Project Boundaries

A successful and cost-effective pavement condition and distress data collection and analysis system that assists in the selection of pavement treatment alternatives based on pavement conditions and distresses and their causes, and that meets the needs of the HA should be structured and implemented in various steps including:

Step 1 - Pavement Condition and Distress Data Collection Policy - A comprehensive, detailed, and clear pavement condition and distress data collection policy should be established that can be supported by administrators and easily implemented by the employees of the organization. The policy should address the various topics presented below.

1. Pavement Condition and Distress Types - Establish the types of pavement condition and distress data and severity levels to be collected. The chosen condition and distress types should reflect the common distresses observed on the surface of the pavement network. For consistency and accuracy purposes, the HAs should clearly define the severity levels of each condition and distress type in a distress identification manual. For guidance, consult the pavement condition manuals (if applicable) of the American Society for Testing and Materials (ASTM), the Federal Highway Administration (FHWA), and/or the Strategic Highway Research Program (SHRP) (SHRP 1993, ASTM 1999, FHWA 2003). It should be noted that some pavement condition and distress types, such as the IRI and rut depth, may not be assigned severity levels; the data are typically collected by sensors and reduced using a continuous scale.
2. Units of Measurement - Determine the units of measurement (such as length, area, or number) to be used for each pavement condition and distress type. Some pavement distress types, such as transverse cracks, could be assigned two measurement units; length and count. The selected measurement units should be the same as those of the pavement condition and distress models used in the pavement design process and should be consistent throughout the HA.
3. Distress Manual - Publish pavement condition manual where each condition and distress type, severity level, and unit of measurements are clearly defined. It is advisable to include in

the manual several photographs of each condition and distress type and severity level. This would help in training personnel and promoting consistency. Examples of such manuals include the ASTM, FHWA, and SHRP (SHRP 1993, ASTM 1999, FHWA 2003).

4. Data Collection Procedures - Establish procedures for pavement condition and distress data collection. The procedures should address the following topics:
   a. Identify common and accurate location reference system to be used by all divisions such as maintenance, construction, rehabilitation, design, and so forth within the agency. It is strongly recommended that a GPS (Global Positioning System) or Geographical Information System (GIS) be used alone or in conjunction with the linear location referencing system such as the beginning and ending mile point system used by many HAs.
   b. Establish the pavement condition and distress data collection methods to be used, such as sensors for longitudinal and transverse profile and automated or semi-automated image recording techniques.
   c. Define the length of the pavement segment for condition survey and for storing the data in the database. It is strongly recommended that 0.1 mile long or shorter pavement survey segments be used.
   d. Determine, if desired, the sampling technique to be used for collecting pavement condition and distress data. It is strongly recommended that all condition and distress data be collected on a continuous basis.
   e. Decide on the pavement condition and distress data collection frequency (every year or every other year) to be implemented. It is strongly recommended that all pavement condition and distress data be collected on a yearly basis.

5. Threshold Values **-** Establish, for each pavement condition and distress type, a distress level or a threshold value. Pavement sections having worse conditions or distress levels than any threshold value are considered to provide substandard level of service and are in need of treatment. For example, a pavement section having IRI value higher than 200 inch/mile, the threshold value, is considered to provide substandard ride quality. Likewise, an asphalt pavement section having 0.5-inch rut depth or more is considered to provide substandard services relative to safety and is in need of treatment. All threshold values and their unit of measurements should be compatible with those used in the pavement design process and with the condition and distress data stored in the database. Nevertheless, the threshold value for each condition and distress type could be based on engineering criteria addressing the safety, ride quality, and structural capacity of the pavement. The engineering criteria may differ from one road class to another and from one traffic level to the next. For example, higher roughness may be acceptable on farm-to-market type roads than on the Interstate. Table 4.5 in Chapter 4 provides a list of threshold values for various pavement condition and distress types that can be used as starter threshold values while the agency debates the final threshold values to be adopted.
6. Pavement Rating Scale - Define a pavement rating scale such as zero to 100 to calculate pavement condition and distress indices (such as structural index, rut index, transverse crack index, and so forth). It would be desirable to establish one common scale for all pavement condition and distress types. The calculation of each pavement condition and distress index should be based on the established threshold values and the corresponding measured condition and distress data. The indices could be used for communication with the general public and/or the legislators. If desired, the index could be divided into discrete zones (such

as 90 to 100, 80 to 90, and so forth) and each zone could be used to establish descriptive terms such as excellent, very good, good, and so forth for easy communication with the general public.

It is important to note that the value of any distress index expresses one snap shot in time corresponding to that particular data collection cycle. The value does not include the pavement rate of deterioration and cannot be used to accurately rank the various pavement sections. For example, two pavement sections may have the same distress index in 2005 and substantially different indices in 2006 due to their significantly different rates of deterioration. When sufficient time-series pavement condition and distress data become available (three or more data points) they should be used to model the data, calculate the pavement rate of deterioration, and estimate the time (the remaining service life (RSL)) until the pavement section in question would reach the threshold value. For planning purposes, it is very important to determine how long the pavement will last until treatment is required and hence, it is highly recommended that the RSL be used to define the condition states, as described in subsection 4.5.3 of Chapter 4.

7. Weight Factors - Establish weight factors between the various pavement distress severity levels and between the various distress types. If desired, combine the various severity levels of each distress using the weight factors to calculate the distress index. The weight factors between the various distress types could be used to calculate composite pavement indices. The individual and composite pavement distress indices should be used for communication with the public only. It is strongly recommended that the data for each distress type and for each severity level be stored in the database in separate fields. The data could be useful to estimate the amount of work to be done and hence the cost and to determine the appropriate

pavement treatment types. For pavement performance modeling and analysis purposes, the various crack severity levels should be combined using uniform weight factor or simply added up. The reason is that most crack models used in the pavement design process such as those embedded in the mechanistic-empirical pavement design guide (M-E PDG) do not differentiate between severity levels.

Step 2 - Pavement Condition and Distress Data Collection – This step consist of various activities including:

1. Collect and digitize the pavement condition and distress data according to the established pavement condition and distress manuals and procedures.
2. Perform extensive quality control during the data collection and digitization process, and quality assurance afterward. The latter could be done in-house or by another contractor independent from the one who collects and digitizes the data. The quality control and/or quality assurance processes could be based on the fact that the severity and extent of each pavement condition and distress type should be consistent from one year to the next. In general, the pavement condition and distress data either remain the same or increase from one year to the next with some expected variability unless the pavement segment in question was subjected to maintenance, preservation, and/or rehabilitation actions. Such check for consistency in the time-series pavement condition data could be computerized. Unexpected changes in the sensor measured pavement condition or distress data relative to earlier measurements should be flagged and re-evaluated as close to real-time as possible. Likewise, while storing the digitized data on the computer, if significant differences or inconsistencies are detected, the computer software could display on the monitor a warning message for

checking the accuracy of the data. If data inconsistency is detected, the data should be scrutinized as follows:

a. Check the construction and maintenance records for possible treatments. It is strongly recommended that for comprehensive and complete PMS, such records be stored in the PMS database using one common location reference system.
b. Examine the quality control and quality assurance procedures for data collection. This should include a check on whether or not the condition data are stored under the correct condition or distress type and location reference point. It should be noted that some of the popular linear systems used by HAs, such as mile point, may produce as much as ± 528 ft error from one year to the next, as described in subsection 4.5.10 of Chapter 4. If linear systems are used, the system should be checked using fixed landmarks such as markers, bridges, railroad crossings, intersections, and so forth.

3. Store the pavement condition and distress data in a relational database. Relational databases store and link the various data elements (such as distress, conditions, rut, design, materials, maintenance, pavement surface age, cost, and so forth) of one pavement segment using a common location reference system such as GPS or mile point. Examples of relational databases include Oracle and d-road.

Step 3 – Pavement Condition and Distress Data Analysis and Treatment Transition Matrices (T^2Ms) – A comprehensive analyses of the measured time-series pavement condition and distress data include the activities presented below:

1. For each pavement survey segment and for each pavement condition and distress type, download all available time-series pavement condition and distress data. It is important to note that a minimum of three data points (three data collection cycles) during which no

pavement preservation or rehabilitation actions were taken are required to properly model the data and to estimate the pavement rate of deterioration and RSL. In general, the pavement surface age since the last pavement treatment (if known) could be used to estimate certain pavement conditions to be used as an additional data point in the analysis. For example, the pavement rut depth and cracking are reduced to zero after the application of hot-mix asphalt (HMA) overlay or mill and fill treatments. Hence, at an early surface age (such as 0.1 year after treatment) the rut depth and cracking length could be relatively accurately assumed to be 0.01-inch and 0.1-ft, respectively. It should be noted that most pavement models (mathematical functions) do not accept zero values. Hence the pavement surface age after treatment could be assigned a small value such as 0.1 year and another small value for the corresponding condition such as rut depth and cracking. Likewise, the International Roughness Index (IRI) could be assumed to be equal to or less than the maximum specified value after construction.

Certain pavement sections may be excluded from the data analysis and the consequent selection of treatment if the pavement condition or distress has passed certain safety threshold (such as low friction or deep rutting). Such pavement conditions should be corrected as soon as possible, hence, for these pavement sections, skip to Step 7 to identify and select the candidate pavement treatment type(s).

2. Perform statistical analysis and model the time-series data using the proper mathematical function to, statistically, determine the parameters of the best fit models. For each pavement condition and distress type, the model should be selected based on known trends in the pavement deterioration (see Table 4.1 of Chapter 4 for recommended functions).

3. For each pavement condition and distress type and for each pavement survey segment (0.1 mile, 1 mile, etc…), use the model (item 2) and the established threshold value (Step 1, item 5) to calculate the RSL of that segment. The RSL could be calculated based on the pavement condition and distress data or the condition indices (see items 6 and 7 of Step 1). It is strongly recommended that one RSL value be calculated for each distress and condition type. Recall that the maximum calculated RSL value should be equal to or less than the design service life (DSL) of the pavement segment in question minus the pavement surface age (SA) as defined in subsection 2.2.4 of Chapter 2.
4. After calculating the RSL of each pavement survey segment for each condition and distress type, select the lowest RSL value as the controlling RSL for that segment.
5. Establish various condition states where each state is based on a range of RSL values. It is recommended that five condition states be adopted as follows:
   - Condition state 1 (CS-1) where the RSL value ranges from 0.0 to 2 years.
   - CS-2 where the RSL value ranges from 3 to 5 years.
   - CS-3 where the RSL value ranges from 6 to 10 years.
   - CS-4 where the RSL value ranges from 11 to 15 years.
   - CS-5 where the RSL value ranges from 16 to 25 years.

   The reason the RSL ranges increase with increasing RSL values is that the accuracy of the calculated RSL decreases as the condition and distress data is predicted for the more distant future.
6. For all pavement segments that were subjected to treatments, do items 1 through 5 twice; once using the time-series pavement condition and distress data before treatment (BT) and once for after treatment (AT).

7. For each pavement treatment type, list the results of item 8 in T^2Ms as outlined in subsection 4.5.3 of Chapter 4.
8. Calculate the number of 0.1 mile long pavement segments and the percent of the pavement network within each condition state. This would yield the distribution of the pavement network in the various condition states. Note that at the network level, the percent of the network within each condition state consists of flexible, rigid, and composite pavement segments of various roads. If desired, the distribution could be divided into several groups, one group per pavement type and road class. Further, the number of pavement segments within a given condition state can be considered to be uniformly distributed along the RSL range.
9. Calculate the weighted average RSL value of the pavement network as discussed in subsection 2.2.4 of Chapter 2, and repeated in Equation 5.1 for convenience.

$$RSL_{(network)} = \frac{\sum_{i=1}^{n} (RSL_i)(SL_i)}{\sum_{i=1}^{n} SL_i} \qquad \text{Equation 5.1}$$

Where, i is the "i^{th}" pavement segment;

n is the total number of pavement segments in the network;

RSL is the remaining service life in years;

SL is the segment length

Step 4 – Pavement Treatment Strategy Analysis – Pavement treatment strategy analysis entails modeling many different network treatment strategies and studying the effects on the pavement network. The treatment strategy is expressed by certain percentages of the pavement network to be treated with certain treatment types and is constrained by the annual budget allocation of the HA. Hence, by analyzing different strategies (many combinations of percentages of the pavement network to be subjected to various treatments) the impact of the various strategies on the longevity of the pavement network could be studied. Once again, the costs of each of the analyzed treatment strategies are the same and limited to the annual budget allocation. However, their benefits are not the same. The network level benefit of any given treatment strategy could be expressed by determining its impact on the value of the pavement network. For example, the value of a 10,000 lane-mile pavement network having weighted average RSL of 5 years is 50,000 lane-mile-years of service. If no actions are taken the network value will decrease by 10,000 lane-mile-year of service for each physical year. Therefore, to retain the status quo, the pavement treatment strategy must add 10,000 lane-mile-year of service each year. If the available budget allocation allows the addition of more than 10,000 lane-mile-years of service, the network longevity will improve and its value will increase, otherwise, it will decrease. Expressing the value of the pavement network in terms of lane-mile-years of service can easily be communicated to engineers, managers, politicians, and the public, and hence, it is a powerful and useful tool.

The distinction between network and project level treatment type and timing selection is critical to the analysis of the pavement network treatment strategies. The network treatment strategy of the HA could be based on maximizing the improvement of the overall condition state of the pavement network for the given budget allocation and/or satisfying other objectives. The project level analyses are conducted to select the specific pavement sections for treatment to satisfy the network treatment strategy. Therefore, the two analyses should be conducted simultaneously or in a series to maximize the benefit to the network and to select the most appropriate pavement sections for treatment. Note that optimization of the treatment strategy implies providing the maximum benefit to the pavement network for the available budget allocation. The network strategy analysis is based on a modified Markov Chain Algorithm (MCA) and consists of the various activities presented below (Kuo et al. 1992, Beach et al. 2012).

1. Divide the pavement network into groups based on the condition state of each pavement segment or analysis length. Each group consists of certain number of miles or percentage of the pavement network within the given range of RSL that are distributed throughout the pavement network. One condition state does not imply uniform conditions or distress, rather it implies consistent remaining service life. For example, two pavement segments could be in condition state 1 where the RSL of one ranges from 0.0 to 2 years due to high roughness and the other due to deep rut depth. Alternatively, one segment could have high roughness and low rate of deterioration and the other medium roughness and high rate of deterioration. In either scenario, the condition state is the same (RSL between 0.0 and 2 years); however the most cost-effective treatments may not be the same. Once the distribution of condition state along the pavement network has been established, the information in the T^2Ms could be used to analyze pavement network treatment strategies and estimate future pavement condition states, as discussed below.
2. Develop and analyze thousands of pavement treatment strategies such that the total cost of each strategy is the same and equal to the available budget allocation. Five examples of treatment strategies for a $400 million budget allocation are listed in Table 5.1. The data in the table indicate that the percent of the network subjected to each treatment type varies from one treatment strategy to another although the total cost of each strategy is fixed at $400

million. Note that the strategy does not specify the particular pavement sections to be treated or its CS.

3. For each treatment type identified in the network treatment strategy, multiply the percent of the network to be treated by the network mileage and by the average cost of the treatment per mile. The sum of all treatment costs should be equal to the allocated budget, if not the percent should be modified to satisfy the allocated budget.
4. For each treatment type identified in the network treatment strategy, multiply the percent of the pavement network to be treated by the network mileage (assume 10,000 lane-miles for this example) and by the BT distribution of the CSs listed in the corresponding T^2Ms in Table 5.2 (the data in Table 5.2 is for the State of Colorado). For example, treatment strategy 1 of Table 5.1 includes 2% thin HMA overlay of asphalt surfaced pavement. Multiply 0.02 by 10,000 and then by the percent of CS 1 in Table 5.2 of 62 percent, which yields 124 lane-miles. Likewise, the numbers of lane-miles in BT CS 2, 3, 4, and 5 are 28, 16, 3, and 8, respectively.
5. For each treatment type identified in the network treatment strategy, use the lengths of pavement treated from each BT CS (see the previous item) and the AT distribution in the T^2M to determine the number of lane-miles transitioned to each AT CS. For example, the 124 lane-miles will be transitioned from BT CS 1 to the five AT CSs according to the distribution listed in the T^2M of Table 5.2; which yields 10, 39, 42, 14, and 19 lane-miles to condition states 1, 2, 3, 4, and 5, respectively.

Table 5.1 Examples of treatment strategies (percent of the pavement network)

Treatment types	Treatment strategies (percent of the network)														
	Strategy 1			Strategy 2			Strategy 3			Strategy 4			Strategy 5		
	%	BT	AT	%	BT	AT	%	BT	AT	%	BT	AT	%	BT	AT
Thin (< 2.5 inch) HMA overlay	2.0	3	4	1.0	3	4	1.5	3	4	1.75	3	4	3.0	3	4
Thick (≥ 2.5 inch) HMA overlay	2.5	2	4	3.0	2	4	2.0	2	4	2.25	2	4	4.0	2	4
Single chip seal	1.0	2	3	0.5	2	3	1.25	2	3	1.0	2	3	2.0	2	3
Double chip seal	2.0	2	3	2.5	2	3	1.0	2	3	1.25	2	3	0.0	2	3
Thin (< 2.5 inch) mill & fill	0.25	3	4	0.35	3	4	0.15	3	4	0.0	3	4	2.0	3	4
Thick (≥ 2.5 inch) mill & fill	0.10	2	4	0.20	2	4	0.15	2	4	0.06	2	4	0.05	2	4
Reconstruction	0.5	1	5	1.0	1	5	0.75	1	5	0.25	1	5	0.0	1	5
Steady state network RSL (year)	7.2			7.0			6.5			6.8			7.0		
Treatment strategy benefit (years)	2.2			2.0			1.5			1.8			2.0		
BT and AT columns contain the average before and after condition states, see the T^2Ms for detailed condition state distributions.															

Table 5.2 T^2M for thin HMA overlay, State of Colorado

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the percent of the 0.1 mile long pavement segments transitioned from each BT CS or RSL bracket to the indicated AT CS or RSL brackets					Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)		
CS or RSL bracket number	RSL bracket range (year)	0.1 mile long pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	TL	SLE	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	543	62		**8**	31	34	11	15	5	8	9
2	3 to 5	124	14		1	**33**	44	7	15	3	5	9
3	6 to 10	71	8		0	34	**35**	7	24	2	2	10
4	11 to 15	38	4		0	26	29	**18**	26	2	-2	11
5	16 to 25	102	12		0	19	34	10	**37**	2	-8	12
Total/average		878/	100/		5/	30/	35/	10/	19/	/4	/5	/9

Alternatively, the T^2M could be slightly modified to present the data listed in Table 5.2 as the percent of the sum of all pavement projects subjected to the same treatment as listed in Table 5.3 for thin HMA overlay treatment in Colorado. The data listed in Table 5.3 could be used directly to determine the AT distribution of 0.1 mile long pavement segments. For example, for the 10,000 lane-mile pavement network example, the 2 percent to be subjected to thin HMA overlays (see Table 5.1) is 200 lane-miles. After treatment, the 200 lane-miles could be distributed to the AT CSs based on the transitions listed in Table 5.3. This yields 10, 50, 70, 20, and 40 lane-miles transitioned to condition states 1, 2, 3, 4, and 5, respectively.

Each untreated pavement segment (do-nothing alternative) will lose one year of service life for each physical year, thus, subtract one year from the corresponding BT RSL. It is important to note that the RSL is a linear function of time and it decreases by one year for each elapsed year. RSL is commonly mistaken as a non-linear function due to the non-linear pavement condition and distress functions from which it is typically calculated. However, regardless of the form of the curve, the RSL is linear. Having said that, the accuracy of the calculated RSL is as good as the accuracy and the variability of the pavement condition and distress data used in the RSL calculation. Such data inaccuracy and variability, however, could be minimized through good quality control and quality assurance practices, as discussed in subsection 2.5.3.2 of Chapter 2. The reduction in RSL may or may not change the condition state. For example, a pavement segment with a BT RSL of 6 (condition state 3) would have an AT RSL of 5 and would transition to condition state 2. On the other hand, a pavement segment with a BT RSL of 8 (condition state 3) would have an AT RSL of 7 and would remain in condition state 3.

Table 5.3 T^2M for thin HMA overlay, (the AT section is listing the percent of the treated pavement segments transitioned to each CS or RSL bracket, State of Colorado

<table>
<tr><th colspan="13">Condition/distress type: condition/distress causing the minimum RSL</th></tr>
<tr><td colspan="5" rowspan="2">Before treatment (BT) data</td><td colspan="8">After treatment (AT) data</td></tr>
<tr><td colspan="5">CS or RSL bracket number and range in years, the SE per CS or RSL bracket, and the percent of the 0.1 mile long pavement segments transitioned from the pavement network to the indicated AT CS or RSL brackets</td><td colspan="3">Treatment benefits in terms of treatment life (TL), service life extension (SLE), and RSL of the treatment (year)</td></tr>
<tr><td rowspan="3">CS or RSL bracket number</td><td rowspan="3">RSL bracket range (year)</td><td colspan="2" rowspan="2">0.1 mile long pavement segments</td><td rowspan="8">Standard error (SE) (cannot be calculated for the minimum RSL)</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td rowspan="3">TL</td><td rowspan="3">SLE</td><td rowspan="3">RSL</td></tr>
<tr><td>0 to 2</td><td>3 to 5</td><td>6 to 10</td><td>11 to 15</td><td>16 to 25</td></tr>
<tr><td>Number</td><td>Percent</td><td colspan="5">SE (cannot be calculated for the minimum RSL)</td></tr>
<tr><td>1</td><td>0 to 2</td><td>543</td><td>62</td><td>5.24</td><td>19.13</td><td>21.18</td><td>6.83</td><td>9.45</td><td>5</td><td>8</td><td>9</td></tr>
<tr><td>2</td><td>3 to 5</td><td>124</td><td>14</td><td>0.11</td><td>4.67</td><td>6.15</td><td>1.03</td><td>2.16</td><td>3</td><td>5</td><td>9</td></tr>
<tr><td>3</td><td>6 to 10</td><td>71</td><td>8</td><td>0.00</td><td>2.73</td><td>2.85</td><td>0.57</td><td>1.94</td><td>2</td><td>2</td><td>10</td></tr>
<tr><td>4</td><td>11 to 15</td><td>38</td><td>4</td><td>0.00</td><td>1.14</td><td>1.25</td><td>0.80</td><td>1.14</td><td>2</td><td>-2</td><td>11</td></tr>
<tr><td>5</td><td>16 to 25</td><td>102</td><td>12</td><td>0.00</td><td>2.16</td><td>3.99</td><td>1.14</td><td>4.33</td><td>2</td><td>-8</td><td>12</td></tr>
<tr><td colspan="2">Total/average</td><td>878/</td><td>100/</td><td></td><td>5.1/</td><td>30.1/</td><td>35.4/</td><td>10.4/</td><td>19.0/</td><td>/4</td><td>/5</td><td>/9</td></tr>
</table>

7. Calculate the after treatment network RSL using Equation 5.1.
8. Repeat items 3 through 7 until the weighted average RSL of the pavement network reaches steady state. The steady state is reached when the distribution of the pavement network in the various condition states becomes constant. When the steady state condition is reached, the longevity of the pavement network becomes constant. For example, the steady state network RSL of network treatment strategy 1 of Table 5.1 is 6.5 years of service.
9. Calculate the benefit of the pavement network treatment strategy using Equation 5.2. For example, assuming the before treatment RSL value of 7.2 years, the benefit of network treatment strategy 1 is 2.2 years of service, as listed in Table 5.1. This could also be expressed in terms of the value of the pavement network. Doing so, the benefits of the 10,000 lane-mile pavement network example is 22,000 lane-mile-years.

$$TSB = RSL_{\text{AT Network, Steady State}} - RSL_{\text{BT Network}} \qquad \text{Equation 5.2}$$

Where, TSB is the Treatment Strategy Benefit expressed as the gain or loss in the longevity of the pavement network (year of service);

$RSL_{\text{AT Network, Steady State}}$ is the weighted average RSL of the pavement network when the steady state condition is reached (year of service) due to the application of the strategy;

$RSL_{\text{BT Network,}}$ is the initial before treatment weighted average RSL of the pavement network (year of service)

10. Repeat items 3 through 9 many times. After sufficient iterations to analyze different network treatment strategies, select the strategies that produce the highest benefit (typically more than 1 strategy produces the same maximum benefits). The preferred strategy could then be selected from these strategies.

A common misconception about the effectiveness of a given network treatment strategy (such as those listed in Table 5.1) is that the steady state weighted average RSL of the network is dependent on the pavement conditions and its rate of deterioration or the RSL values BT. The long term gain or loss in the weighted average RSL (benefit) of a pavement network subjected to a treatment strategy is independent of the condition states of the pavement network prior to the application of the strategy. The network treatment strategy benefit is only a function of the effectiveness of the strategy itself, which is dependent on the state-of-the-practice and the available budget allocation. Figure 5.1 illustrates the constant long term weighted average RSL for a given network treatment strategy applied to a pavement network having four different BT weighted average RSL values. The data in the figure clearly show, for the given network treatment strategy, that the steady state weighted average RSL is the same (about 7.6 years of service) and independent of the weighted average BT RSL value. Note that the steady state weighted average network RSL of 7.6 years of service could be increased if the budget allocation were increased as discussed below.

The steady state weighted average RSL of a pavement network could also be used as a tool to quantify the effect of budget allocation on the longevity of the pavement network. This could be accomplished by developing one “optimum” pavement network treatment strategy for various budget allocations. This is shown in Figure 5.2 where the BT weighted average RSL of the pavement network was held constant and the steady state weighted average RSL was calculated based on four budget allocations; \$50,000,000, \$120,000,000, \$200,000,000, and \$250,00,000. As is shown in the figure, the steady state weighted average RSL of the network increased from 2.2 to 3.5, to 4.5, and to about 7.2 years, respectively. This procedure could also

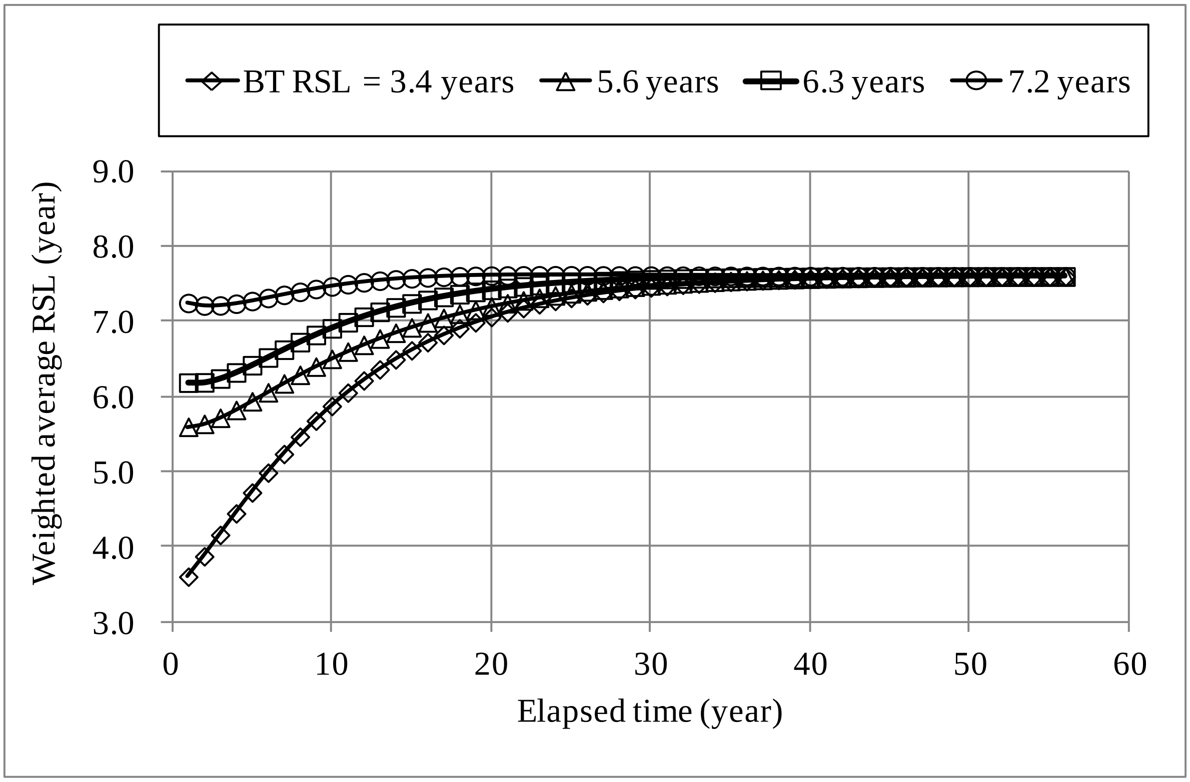


Figure 5.1 Weighted average RSL versus time for a budget allocation and four BT pavement network conditions

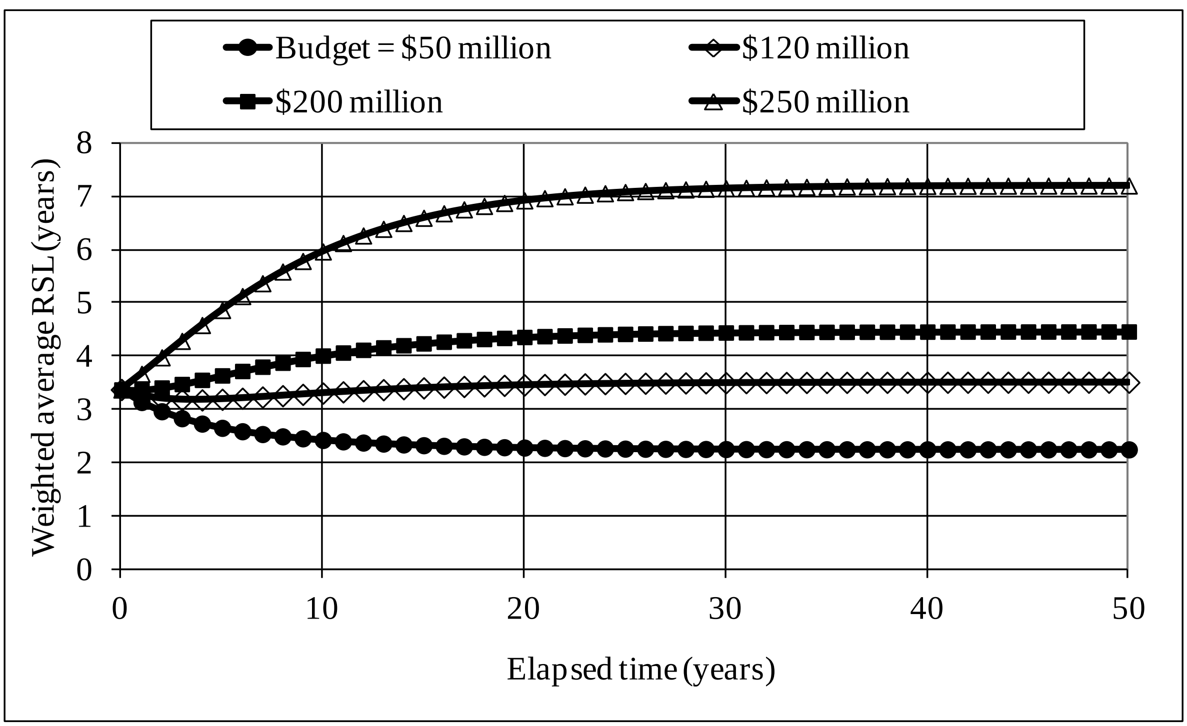


Figure 5.2 Weighted average RSL of a pavement network versus four budget allocations

be used as a simple yet powerful tool to determine the required budget allocation for a desired level of future RSL value.

Step 5 - Identification of Candidate Project Boundaries - A comprehensive identification of candidate project boundaries include the activities presented below:

1. Consult the HA policy regarding the minimum and maximum lengths of a pavement project. The minimum length should be established on the basis of costs (mobilization and actual cost of the treatment), and the maximum length based on the availability of HA staff, the budget allocation, and the project needs. Use the established budget allocation formula (based on pavement class, traffic, etc.) to determine the percent of the total budget allocation to be distributed to each region or district. Based on the outcome, determine the percent of the preferred treatment strategy to be treated in each region. For example, consider treatment strategy 1 listed in Table 5.1 and a HA district which receives 20% of the total HA budget allocation. The district should perform 20% of the specified strategy.
2. Use the existing methods (AASHTO, in-house methods, etc.) to delineate the pavement network into candidate pavement sections for treatment based on the condition states. The selected pavement treatment strategy, see Step 4 item 10 above, determines the length of pavement to be subjected to each of the pavement treatments. These lengths could be widely distributed along the pavement network or clustered in few areas. The pavement mileage to be treated would be distributed along the network in a matter consistent with the annual budget allocation per district. Upon completion of this step a list of all candidate project boundaries should be generated for further analyses.

The selection of the final project boundaries for the projects which satisfy the treatment strategy for each district or region and subsequently for the pavement network will be conducted as part of Step 9. The appropriate treatment types must be determined for each of the candidate pavement sections obtained in this step as discussed in Step 7. Note that in some HA, the regional or the district manager makes the final selection of the project boundaries in his/her region for various reasons. The locations of the selected projects are inconsequential to the network treatment strategy and could be determined based on any set of objectives. Each project and the project boundaries should be identified by length, reference locations, and the pavement treatment type to be applied based on the preferred treatment strategy.

Step 6 - Forensic Investigation - For each candidate project, download and study all available data from the PMS database. Based on the condition and distress data and available forensic investigation data, determine the causes of each observed or measured pavement condition and distress. The causes of pavement conditions and distresses may be identified by completion of one or more of the following forensic investigation activities:

1. Review available results, if any, of past forensic investigations into the causes of the conditions and distresses on the pavement section in question.
2. Download from the pavement inventory or the PMS database the pavement type, the treatment history, maintenance history, layer types and thicknesses, material types, drainage, and so forth. The data are needed to determine possible design issues and for the backcalculation of layer moduli, if pavement deflection data are available or will be obtained.
3. Visit the project to:
    a. Verify the available pavement conditions and distress data and the extent of the distresses.

b. Conduct visual forensic investigation of the pavement surface and its surroundings (see Table 5.4) to determine the causes of pavement conditions and distresses. During the visual investigation:

Table 5.4 Visual forensic investigations

Observation	**Indication**	**Causes**	**Possible conditions and distress**
Drainage	Water in drainage ditches or lines and culverts	Low permeability material, lack of edge drainage and/or drainage ditch	Weakened lower pavement layers, cracking, rutting, roughness
Segregation	Non-uniform surface	Particle or temperature segregation during construction	Raveling, cracking
Shoving and corrugation	Displacement of top asphalt layer	Soft asphalt mix, improper asphalt grade, high asphalt content	Roughness, potholes
Potholes	Hole in asphalt layer	Frost susceptible materials	Safety issue
Edge cracking	Wash out of edge materials, longitudinal edge cracking	Lack of lateral support	Longitudinal edge cracking
Faulting	Differential elevation at joints	Lack of load transfer devices	Roughness
Spalling	Deterioration around cracks	High moisture in the vicinity of cracks/joints	Roughness
Loss of friction	Smooth macro-texture	High soft aggregate content	Safety issue
Popouts	Aggregate loss in surface	Dirty aggregates	Poor ascetics

i. Assess the quality of drainage; sitting water in the drainage ditches, in the drainage line, or in culverts may indicate saturated pavement layers.

ii. Observe the pavement surface for possible aggregate segregation (which may cause raveling and cracking), shoving and corrugation (which is typically caused by unstable asphalt mixtures), or other defects that are typically not recorded during the regular pavement condition and distress survey.

iii. Examine the status of the lateral support and the stability of the embankment. Unstable embankments could lead to longitudinal edge cracking.

iv. Examine the quality of the shoulder and whether or not improvement is needed as a part of the project work.

c. Determine the extent and types of work (such as signs, guard rails, shoulder improvement, drainage improvement, and so forth) to be included in the project contract.

4. Obtain pavement cores for laboratory testing to determine the HMA or Portland cement concrete (PCC) contents, the percent air voids, density, unconfined compressive strength, etc. The cores could also be used to:

   a. Measure and verify the thicknesses of the pavement layers.

   b. Determine the type of cracks (top-down or bottom-up), and the depth of each crack. Shallow top-down cracks could be treated by milling the asphalt layer (to the crack depth) and filling.

   c. Inspect the cores for possible delamination and honeycombing (asphalt cores), which may cause rutting, stripping, and/or cracking.

5. Perform non-destructive testing such as:

   a. Non-destructive deflection testing (NDT) using Falling Weight Deflectometer (FWD) to measure the pavement deflection. The deflection data could be analyzed to determine the pavement structural capacity, the load transfer efficiency at joints, the presence of voids, and/or for backcalculation of the pavement layer moduli and the design of the thickness of the overlay.

   b. Ground penetrating radar (GPR) to estimate the pavement layer thicknesses.

   c. Sounding to determine the layer thicknesses and their stiffness.

Step 7 – Selection of Treatment Types - In general, each deteriorated pavement segment along the network could be preserved or rehabilitated using the proper treatment alternatives (see Table 5.5). Some of the pavement treatment alternatives are effective whereas others are not. In certain scenario, a feasible treatment could be the combination of two or more treatments. For example, improving drainage, sealing the cracks and/or joints, and grouting could address the causes of, and solve the, pumping problem in concrete pavements. It should be noted that the alternative pavement treatment types should address both the pavement conditions and distresses and their causes (see Table 5.6). Nevertheless, in this step, all pavement treatment alternatives are selected using the treatment selection matrices provided in Tables 5.7 through 5.12 below. Use the pavement conditions and distresses and their causes (see Step 6) to determine all pavement treatment alternatives that address most, if not all, the measured or observed pavement conditions and distresses and their causes. The desired activities leading to the selection of pavement treatment types are enumerated below.

1. Determine the alternative treatment types for each candidate project identified in Step 5. Treatment type selection tables were developed to assist in the selection of candidate pavement treatments based on pavement conditions and distresses and their causes identified by forensic investigation (see Step 6). The tables include the pavement treatment types and the pavement conditions and distresses defined below. Note that the lists are not exhaustive and may use terms which are different from those used by some HAs to describe similar pavement treatments or pavement conditions or distresses.

   After identifying the causes of pavement conditions and distresses (forensic investigation), the pavement treatments which will correct the causes of the pavement conditions and distresses should be identified. Those treatments are known or should be

Table 5.5 Some pavement treatment types

Treatment type	Treatment details
Chip seal	Single or double asphalt and stone chips (Baladi et al. 1992).
Crack and seat & overlay	Cracking the PCC slab in-place and compacting it as a base to support an HMA or PCC overlay.
Crush & shape & overlay	The crushing and re-shaping of the HMA pavement layer followed by HMA or PCC overlay.
Diamond grinding	The removal of the top layer (up to 0.75 inch) of the PCC surface with diamond embedded saw blades to improve the ride quality.
Dowel bar retrofit	The installation of dowel bars in existing PCC pavements where load transfer is low or non-existent, such as across transverse cracks.
Mill and fill	Milling 1 to 4-inch and replacing it with HMA.
Patching	The partial or full-depth removal of limited pavement area and replacement with new or recycled HMA or PCC.
PCC overlay	PCC overlay of an existing HMA (white topping) or PCC surface.
Reconstruction	Removal of the surface material or the existing pavement structure and replacement with new or recycled or virgin materials.
Rubbelize and overlay	The in-place crushing of PCC slab for use as an aggregate base to support an HMA or PCC overlay.
Surface seal (various types)	The application of a mixture of asphalt emulsion, with or without aggregates (Baladi et. al. 1992).
HMA overlay	Nonstructural (< 2.5-inch) and structural (≥ 2.5 inch) HMA overlay over PCC or HMA surface.

Table 5.6 Possible causes of pavement condition and distress types

Condition or distress type	Possible causes of pavement conditions and distresses
Map cracking	Alkali-silica reaction (ASR)
Block cracking	Aging and oxidation of the surface of the asphalt layer
Durability (D-cracking)	Non-durable aggregate
Fatigue (alligator) cracking	Low tensile strength in the asphalt layer and/or under design
Faulting	Lack of load transfer mechanism
International Roughness Index (IRI)	Curling, warping, corrugation, shoving, texture roughness, cracks, and joints
Longitudinal cracking	Construction quality, delayed longitudinal joint saw cutting
Rutting	Studded tire, snow removal, soft asphalt mix, poor compaction, soft and saturated base, subbase, and/or roadbed soil, poor drainage
Transverse cracking	Temperature expansion and contraction, delayed transverse joint sawing, for PCC fatigue cracking

Table 5.7 Common asphalt surfaced pavement condition and distress types, the causes of the pavement condition or distress, and the alternative pavement treatments

Row designation	Pavement condition and distress type	Causes of pavement condition or distress										
		Column designation										
		A	B	C	D	E	F	G	H	I	J	K
		F1	F2	F3	F4	F5	F6	F7	F8	F9	F10	F11
1	Alligator (fatigue) cracking[1]	TF5	TF5	TF5	TF2, 3[1], 4, or 5	TF2, 3[1], 4, or 5	TF2, 3[1], 4, or 5	-	TF5	-	-	TF2 to 5, 7, or 8
2	Block cracking	-	-	-	-	-	-	TF1 to 8	-	-	-	TF1 to 8
3	International Roughness Index (IRI)	TF5	TF5	TF5	TF2, 4, or 5	TF2, 4, or 5	TF2, 4, or 5	TF2 to 5	TF5	-	TF2, 4, or 5	TF2 to 5, 7, or 8
4	Longitudinal cracking	TF5	TF2, 4, or 5	TF2 to 5	TF2, 4, or 5	TF2, 4, or 5	TF2, 4, or 5	-	TF5	-	TF2, 4, or 5	TF2 to 5, 7, or 8
5	Patching	TF4 or 5	TF4 or 5	TF5	TF4 or 5	TF4 or 5	TF4 or 5	TF4 or 5	TF5	TF4 or 5	TF4 or 5	TF2 to 5, 7, or 8
6	Rutting	TF5	TF2, 4, or 5	TF5	TF2, 4, or 5	TF2, 4, or 5	TF1, to 5, 7, or 8	-	TF5	TF2, 4, or 5	-	TF2 to 5, 7, or 8
7	Transverse cracking	-	TF2, 4, or 5	-	TF2, 4, or 5	TF2, 4, or 5	TF2, 4, or 5	-	-	-	TF2, 4, or 5	TF2 to 5, 7, or 8

[1] If alligator cracking is top-down then mill and fill to the depth of the cracks is an option

See Tables 5.9 and 5.11 for the pavement condition and distress type and their causes coding

Table 5.8 Common PCC surfaced pavement condition and distress types, the causes of the pavement condition or distress, and the alternative pavement treatments

Row designation	Pavement condition and distress types	Causes of pavement condition or distress												
		Column designation												
		A	B	C	D	E	F	G	H	I	J	K	L	M
		R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Alkali-silica reaction (ASR), map cracking	-	-	-	-	-	-	TR5	-	-	-	TR5	-	TR1 or 4 to 7
2	Durability cracking (D-cracking)	-	-	-	-	-	-	TR5	-	-	-	-	TR5	TR1 or 4 to 7
3	International Roughness Index (IRI)	TR5	TR1, 2, or 5	TR5	TR1, 4, or 5	TR1, 4, or 5	TR1 or 3 to 5	TR1, 4, or 5	TR1, 4, or 5	TR5	TR1 or 5	TR5	TR5	TR1, 2, or 4 to 7
4	Faulting	TR5	-	TR5	-	-	-	-	-	TR1, 4, or 5	-	-	-	TR1 to 7
5	Longitudinal cracking	TR1, 4, or 5	-	TR5	-	-	-	TR1, 4, or 5	TR1, 4, or 5	TR5	TR1, 4, or 5	-	-	TR1 or 4 to 7
6	Patching	TR4	TR2 or 4	TR5	TR4	-	TR4	TR4	TR4	TR5	TR4	TR4	TR4	TR1 or 4 to 7
7	Transverse cracking	TR1, 4, or 5	-	TR5	TR1, 4, or 5	TR1, 4, or 5	-	TR1, 4, or 5	TR1, 4, or 5	TR5	TR1, or TR5	-	-	TR1 or 4 to 7
See Tables 5.10 and 5.12 for the pavement condition and distress type and their causes coding														

Table 5.9 Common causes of asphalt surfaced pavement condition and distress

Code	Cause of pavement condition and distress
F1	Base/subbase material properties (gravel < 15,000 psi/sand < 8,000 psi)[1]
F2	Construction problems (particle and temperature segregation, insufficient compaction, etc…)
F3	Drainage
F4	HMA mix design
F5	HMA material properties
F6	HMA thickness
F7	Oxidation/ hardening of the asphalt binder
F8	Roadbed soil material properties (< 3,000 psi)[1]
F9	Soft aggregate
F10	Temperature expansion & contraction[2]
F11	Unknown cause

[1] Leading to reduced support under the asphalt mat or reduced edge support
[2] If composite pavement, temperature expansion and contraction is in the underlying PCC

Table 5.10 Common causes of PCC surfaced pavement condition/distress

Code	Cause of pavement condition and distress
R1	Construction problems (late sawing of joints, hot/dry environment, segregation, etc…)
R2	Curling and/or warping (temperature and/or moisture gradient)
R3	Drainage
R4	Fatigue[1]
R5	Joint spacing
R6	Joint width
R7	PCC mix design
R8	PCC material properties
R9	PCC slab support
R10	PCC thickness
R11	Reactive aggregate
R12	Soft aggregate
R13	Unknown cause

[1] Transverse cracking could be caused by the material properties or under design

Table 5.11 Common asphalt surfaced pavement treatment types

Code	Pavement treatment type
TF1	Chip seal (single or double)
TF2	Crush and shape and overlay
TF3	Mill and fill
TF4	Patching
TF5	Reconstruction
TF6	Surface seal (various types)
TF7	Thick HMA overlay (≥ 2.5 inch)
TF8	Thin HMA overlay (< 2.5 inch)
Improving drainage and/or the application of pre-treatment repairs (when applicable) is recommended with any treatment	

Table 5.12 Common PCC surfaced pavement treatment types

Code	Pavement treatment type
TR1	Crack and seat and overlay or rubbelize and overlay[1]
TR2	Diamond grinding
TR3	Dowel bar retrofit
TR4	Patching
TR5	Reconstruction
TR6	Thick HMA or PCC overlay (≥ 2.5 inch)
TR7	Thin HMA overlay (< 2.5 inch)
[1] If composite pavement, HMA must be removed prior to crack and seat or rubbelization	
Improving drainage and/or the application of pre-treatment repairs (when applicable) is recommended with any treatment	

developed a priori by the HA and could be stored in matrices or decision trees. Example of treatment selection matrices based on some causes of pavement conditions in asphalt and in PCC surfaced pavements are included in Tables 5.7 through 5.12. Similar matrices could be developed by each HA based on their previous experience and pavement performance. Tables 5.7 and 5.8 list the pavement conditions and distresses and their causes and the alternative pavement treatments. The codes in the corresponding cells are the alternative treatment types for each pair of condition or distress and its causes. The key to the codes in Tables 5.7 and 5.8 are listed in Tables 5.9 and 5.10 for the causes of pavement conditions and distresses and Tables 5.11 and 5.12 for the treatment types, for both asphalt and PCC surfaced pavements, respectively. The treatment type selection tables could be used following the steps outlined below. Note that the pavement condition and distress survey and forensic investigation should be completed prior to the use of the selection tables.

a. Identify the applicable pavement condition and distress types on the left side of Table 5.7 or 5.8 for asphalt or PCC surfaced pavements, respectively.
b. Identify the applicable causes of the pavement conditions and distresses in Tables 5.9 or 5.10 for asphalt or PCC surfaced pavements, respectively. If the causes of pavement condition and distress are unknown or the HA does not wish to address the causes, one may choose option F11 or R13 for asphalt and PCC surfaced pavement, respectively to improve the conditions and distresses only.
c. Record the codes corresponding to the outcomes of parts a and b from Table 5.7 or 5.8. Locate these codes in Tables 5.11 or 5.12 for asphalt or PCC surfaced pavements, respectively.
d. If multiple pavement conditions or distresses are present, identify the treatment type(s) that were most commonly identified.

For example, consider a flexible pavement section with alligator cracking caused by HMA material properties or under design. The codes in cell E1 of Table 5.7 indicate that, for alligator cracking stemming from HMA material properties, four candidate treatment types exist (TF2, 3, 4, or 5). Alligator cracking typically starts from the bottom of the asphalt layer and has penetrated the entire asphalt layer before it is visible on the surface. The crack is moving upward and will propagate to the surface of any pavement treatment in relatively short time. For this reason the asphalt layer must be replaced, and hence crush and shape and overlay, patching, and reconstruction are the only options to address the condition and its causes. The particular treatment could be selected based on the extent of the distress. For example, low to medium severity top-down cracking could be treated by milling and filling to the crack depth. On the other hand, high severity top-down cracking has likely penetrated the entire asphalt surface and would require replacement. Note that other treatments could be applied to address the pavement condition, such as HMA overlay or mill and fill, but they will not address the causes of the condition; and hence the condition will likely return expediently.

2. List the alternative treatment types for each candidate pavement section.

Step 8 - Calculation of Treatment Cost and Benefits – A comprehensive evaluation of pavement treatment costs and benefits includes:

1. For each pavement treatment alternative identified in Step 7:
    a. Perform or select preliminary treatment design, which must include sufficient details to predict the cost based on the pavement conditions and what actions are required (including pre-treatment repairs, repair of non-pavement assets, and the construction/ traffic control methods).
    b. Calculate the agency costs. It should be noted that comprehensive and accurate cost data are essential for determining the cost of pavement treatments. The same treatment applied to different pavements in different condition states and physical locations and at different time may have substantially different costs. The most reasonably accurate estimate of cost is required in order to select between alternative treatment types and project boundaries.

c. Calculate the user costs. It should be noted that comprehensive and accurate user cost data are essential for determining the cost of pavement treatments. The same treatment applied to pavements in rural areas and urban locations will have substantially different user costs due to the traffic control and volume.

d. Estimate the treatment benefits based on the pavement condition states, the treatment type, and the T^2M. If such T^2Ms are not available, the expected treatment benefits could be based on the HAs experience.

2. Perform life cycle cost analysis (LCCA). A pavement section will typically be subjected to several series of pavement treatments in its life including all preservation, maintenance, and rehabilitation activities between major rehabilitation or reconstruction. LCCA entails estimating the pavement life or the remaining pavement life and the various treatments which could be applied over that life to maintain serviceability. In this case, the pavement life is considered to come to an end when major rehabilitation or reconstruction is the most cost-effective treatment alternative. The most cost-effective series of treatments during a pavement life could be developed in similar way to that previously discussed for a single treatment. The main difference is that the costs and benefits of several actions must be estimated. The difficulty lies in predicting the future pavement conditions or distresses and hence the candidate treatment types.

The data in the T^2Ms express the expected service life, as discussed above, while the treatment type could be determined by the expected AT controlling pavement condition or distress type, as shown in figures such as Figure 4.16 of Chapter 4. The data listed in Figure 4.16 indicate that the RSL after the application of thin HMA overlay of asphalt surfaced pavement in the State of Colorado most often is controlled by alligator cracking AT. Hence, if thin HMA overlay is selected as a treatment the subsequent treatment should address the anticipated alligator cracking, such as patching, crush and shape and overlay, or reconstruction. In this way the cumulative treatment costs and their benefits could be estimated over the pavement life cycle. Several project treatment strategies could be analyzed with varying treatment types and application timing. The total treatment cost could be calculated as the sum of all treatment costs, and the total treatment benefit could be calculated as the sum of all treatment benefits.

3. Calculate the cost to benefit ratio in $/lane-mile-year of service for each treatment alternative. The agency cost should be used in this ratio. The user cost should not be included in the cost to benefit ratio but rather used as a “tie-breaker” to help decide between competitive treatment alternatives.
4. Rank the alternative treatments relative to their cost to benefit ratio.
5. Finally, the treatment type which satisfies the objective function, Equation 4.1 of Chapter 4, should be selected as the most cost-effective treatment for the pavement section.

Step 9 – Selection of the preferred set of pavement projects - Satisfy the selected pavement network treatment strategy by selecting the sets of pavement projects that are the most cost-effective and that suit the HA objectives. Recall that the network strategy may need to be subdivided along with the budget allocation by region or district, and that each region or district should select projects to satisfy their portion of the network strategy. In this way, the accumulation of all regional or district pavement treatment selections will satisfy the network strategy. The project selectors should bear in mind that they are selecting pavement projects to best improve the entire pavement network and not just the partial network for which they are responsible. In other words, the network strategy trumps the regional, district, or project level decisions.

Candidate pavement projects could be differentiated by the expected cost-effectiveness. For example, two candidate pavement projects which were identified for thin HMA overlay will likely have varying BT condition state distributions and will therefore have different cost-effectiveness. Recall that the treatment type which satisfies the objective function, Equation 4.1 of Chapter 4, was selected in Step 8; however the cost-effectiveness of one candidate project may be different from that of a similar candidate project. One candidate project may yield cost/benefit ratio of $25,000/year of service while the other $23,000/year of service. The candidate pavement projects with the absolute minimum values of the objective functions, of all candidate treatments, and which satisfy the network strategy, should be chosen for treatment. Alternatively, the candidate pavement projects could be differentiated in the same way as above except by considering the results of LCCA. Additionally, the project length may be modified to adjust the cost and to satisfy the designated budget allocation.

Finally, Figure 5.3 recaps the general project selection flow.

Step 10 - Study and Investigate Pavement Performance – A comprehensive feedback system includes the following activities:

1. Study the various pavement segments in the network over time to investigate the reasons why some pavements fail prematurely, others perform as designed, and still others perform better than designed. Pavements should be studied and tested from design, to construction, to maintenance/rehabilitation, and to reconstruction. The knowledge gained from observation and testing of the various pavement types and designs could lead to better pavement design,

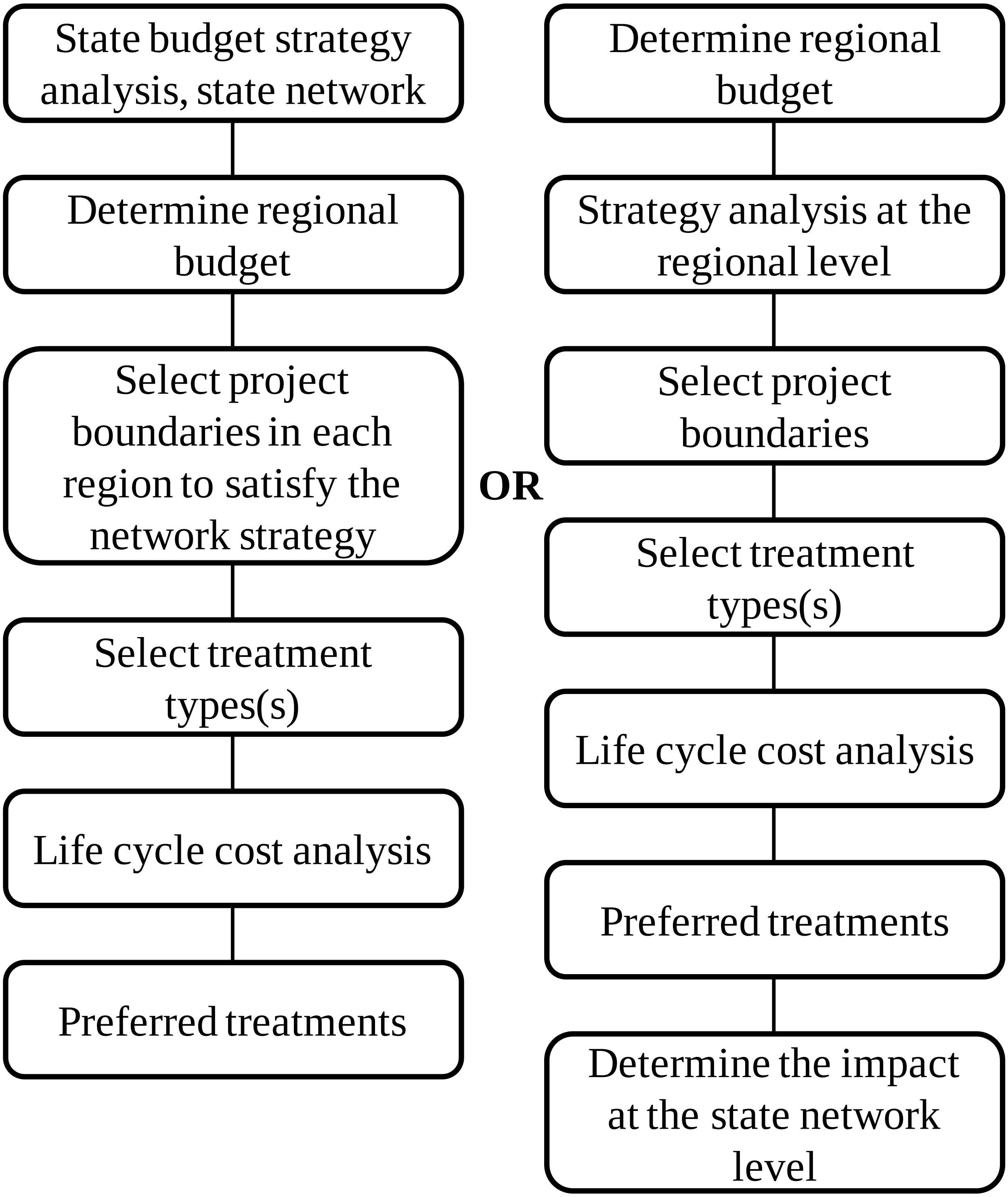


Figure 5.3 Network treatment strategy to project selection flowchart

maintenance, and construction. It should be noted that “good” pavements should be studied as much as “poor” pavements, if not more.

2. Establish feedback loops to review and reform the procedures and decisions made based on the guidelines and procedures presented. The post-treatment pavement conditions and distresses should be monitored with time. Analyses of the post-treatment pavement conditions and distress will provide the data necessary to create a feedback loop for the methodology and to update and calibrate the T^2Ms. Any discrepancies between the predicted and observed pavement conditions or distresses could be used to improve the predictive models. The following components of the methodology could be modified based on the feedback:
    a. The pavement condition threshold values (what defines acceptable levels of service).
    b. The treatment type selection based on pavement conditions.
    c. The treatment type selection based on the causes of pavement conditions.
    d. The expected immediate condition improvement due to application of the treatments.
    e. The agency and user costs of the treatments.
    f. The T^2Ms.

CHAPTER 6

SUMMARY, CONCLUSIONS, & RECOMMENDATIONS

6.1 Summary

State Highway Agencies (SHAs) select pavement treatments based on varying criteria and with mixed results. In this study, it was hypothesized that for a given pavement network and budget, there exists a cost-effective pavement treatment strategy consisting of treatment types, times, and project boundaries that would minimize the objective function by minimizing the cost to benefits ratio.

To verify the hypothesis, pavement management system (PMS) databases were obtained from four SHAs; Colorado, Louisiana, Michigan, and Washington and the Minnesota Road Research (MnROAD) test facility. The databases were scrutinized and pavement sections which were treated in the past were identified and the time-series pavement condition and distress data were modeled before and after treatment. The models were used to calculate the remaining service lives (RSL) and treatment benefits for six common pavement treatment types, thin and thick hot-mix asphalt overlays of asphalt surfaced pavements, single and double chip seals, and thin and thick mill and fill. Treatment transition matrices (T^2Ms) were developed to display the results of the analyses in a convenient and implementable format. The data in the T^2Ms were used to study the intrastate and interstate states-of-the-practices and to determine the benefits of the treatments. Methodologies and procedures were developed to use the data listed in the T^2Ms to estimate future pavement condition states and to perform pavement network treatment strategy analyses.

At both the network and the project levels, detailed step-by-step procedures and treatment matrices were developed to guide SHAs in the selection of cost-effective pavement treatment time, type, and project boundaries based on the pavement conditions and distresses and their causes and on the results of the analysis listed in the T^2Ms.

Further, to improve cost-effectiveness, recently, many SHAs began to treat different pavement lanes of multi-lane facilities at different times. This new way of thinking and the selection of cost-effective pavement treatment type, time, and project boundaries for each lane individually require knowledge of the pavement conditions and distresses and its rates of deterioration along each lane. Based on this need a new methodology was developed and verified to predict the conditions and distresses of the passing lanes. The methodology is based on the measured time-series pavement condition and distress data along the driving lane and the load distribution factors between the lanes of MnROAD test facility.

6.2 Conclusions

Based on the results of the analyses conducted in this study and on the analyses of the existing state-of-the-practice, the following conclusions were drawn:

- Certain data elements (such as treatment cost and location and pavement surface age) are currently not included in the PMS database.
- Some of the pavement condition and distress data are missing or highly variable and they do not support the analyses of pavement rate of deterioration.
- The three pavement treatment benefits used in this study, treatment life (developed in this study), service life extension, and the after treatment RSL, could be used collectively to express the treatment benefits.

- The existing time-series pavement condition and distress and treatment histories do not support the total benefit and its modification (developed in this study).
- Housing the analysis results in the T^2M format (developed in this study) present a snap-shot of the historical state-of-the-practice in pavement preservation and rehabilitation.
- For each treatment type, the data in the T^2Ms can be used to:
    - Study the distribution of the before and after treatment condition states along the projects.
    - Analyze the differences in the state of the practice between the various regions or districts.
    - Analyze the benefits of the treatment strategy at the project and network levels.
    - Assess the impacts of treatment time on its benefits.
    - Provide feedback to the state highway agency.
- Linear location referencing systems causes up to ± 528 feet error in locating a point along a pavement structure.
- The IRI and rut depths of the passing lanes of multi-lane facilities could be estimated using the measured driving lane data and the two methodologies developed in this study.

6.3 Recommendations

Based on the results of the analyses and conclusions, and for cost-effective selection of pavement treatment type, time, and project boundaries it is strongly recommended that:

- The PMS databases be expanded to include the comprehensive pavement data listed in Table 2.35 (pre-treatment repairs, materials, treatment thickness or application rates, safety improvements, treatment costs (materials, labor, traffic control, equipment mobilization), etc.).
- Express the before and after treatment condition states in T^2M format.

- Use the treatment life, service life extension, and the after treatment RSL express the pavement treatment benefits.
- Utilize Global Positioning System (GPS) or equivalent system in addition to the linear referencing system to locate the boundaries of various pavement segments.
- Use relational database to link the various data elements based on one common and accurate location reference system.
- Use the methodologies developed in this study to estimate the IRI and rut depths of the passing lanes of multi-lane facilities based on the measured driving lane IRI and rut depth data and the traffic lane distribution factors.
- Tailor the guidelines, procedures, and recommendations listed in Chapter 5 for the selection of pavement treatment type, time, and project boundaries to the states needs.

APPENDIX A

PMS DATA

The PMS databases collected for this study are colossal and would take many thousands of pages to include on paper. Hence, the databases are available, by request, from the Department of Civil & Environmental Engineering at Michigan State University. To request the data, please contact:

Department of Civil & Environmental Engineering

C/O Dr. Gilbert Baladi

3546 Engineering Building

Michigan State University

East Lansing, MI 48824-1226

Tel: (517) 355-5107 Fax: (517) 432-1827 e-Mail: cee@egr.msu.edu

For convenience, examples of the pavement condition and distress data are included below. Tables A.1 and A.2 contain the measured and formatted condition and distress data (see Chapter 3) for a section of I-70 in Colorado from the Colorado Department of Transportation (CDOT). Similar examples of the formatted data are listed in Tables A.3 through A.6 for the Louisiana Department of Transportation and Development (LADOTD), the Michigan Department of Transportation (MDOT), the Washington State Department of Transportation (WSDOT), and the Minnesota Road Research facility (MnROAD).

Table A.1 Measured pavement condition data along I-70 in Colorado

Route	Control section	Mile points		Direction to survey	Year	IRI (in/mi)	Rut depth (in)	Alligator cracking (ft^2)	Longitudinal cracking (ft)	Transverse cracking (Count)
		Beginning	Ending							
070A	-	4.8	4.9	1	1998	98	0.4	0	3	1
070A	-	4.9	5	1	1998	114	0.4	0	1	1
070A	-	5	5.1	1	1998	94	0.4	0	8	2
070A	-	5.1	5.2	1	1998	99	0.4	0	1	0
070A	-	5.2	5.3	1	1998	98	0.3	0	0	0
070A	-	5.3	5.4	1	1998	78	0.3	0	7	0
070A	-	5.4	5.5	1	1998	80	0.3	0	0	1
070A	-	5.5	5.6	1	1998	89	0.3	0	0	0
070A	-	5.6	5.7	1	1998	100	0.3	0	0	0
070A	-	5.7	5.8	1	1998	104	0.3	0	0	0
070A	-	5.8	5.9	1	1998	95	0.3	31	0	0
070A	-	5.9	6	1	1998	86	0.3	0	0	0
070A	-	6	6.1	1	1998	112	0.3	0	0	0
070A	-	6.1	6.2	1	1998	81	0.3	0	3	0
070A	-	6.2	6.3	1	1998	85	0.3	0	0	0
070A	-	6.3	6.4	1	1998	85	0.3	0	0	1
070A	-	6.4	6.5	1	1998	103	0.3	0	2	0
070A	-	6.5	6.6	1	1998	78	0.3	0	2	0
070A	-	6.6	6.7	1	1998	93	0.4	100	14	0
070A	-	6.7	6.8	1	1998	86	0.4	0	0	1
070A	-	6.8	6.9	1	1998	71	0.4	0	0	0
070A	-	6.9	7	1	1998	92	0.4	0	0	1
070A	-	7	7.1	1	1998	95	0.4	2	0	0
070A	-	7.1	7.2	1	1998	90	0.4	0	0	0
070A	-	7.2	7.3	1	1998	76	0.4	0	0	0
070A	-	7.3	7.4	1	1998	93	0.3	0	0	0
070A	-	7.4	7.5	1	1998	74	0.4	0	3	1

Table A.2 Formatted pavement condition for Table A.1

Route	Control section	Mile points		Direction to survey	Year	IRI (in/mi)	Rut depth (in)	Alligator cracking (ft)	Longitudinal cracking (ft)	Transverse cracking (ft)
		Beginning	Ending							
I-70	070A	4.8	4.9	1	1998	98	0.4	0	3	12
I-70	070A	4.9	5	1	1998	114	0.4	0	1	12
I-70	070A	5	5.1	1	1998	94	0.4	0	8	24
I-70	070A	5.1	5.2	1	1998	99	0.4	0	1	0
I-70	070A	5.2	5.3	1	1998	98	0.3	0	0	0
I-70	070A	5.3	5.4	1	1998	78	0.3	0	7	0
I-70	070A	5.4	5.5.	1	1998	80	0.3	0	0	12
I-70	070A	5.5	5.6	1	1998	89	0.3	0	0	0
I-70	070A	5.6	5.7	1	1998	100	0.3	0	0	0
I-70	070A	5.7	5.8	1	1998	104	0.3	0	0	0
I-70	070A	5.8	5.9	1	1998	95	0.3	2.6	0	0
I-70	070A	5.9	6	1	1998	86	0.3	0	0	0
I-70	070A	6	6.1	1	1998	112	0.3	0	0	0
I-70	070A	6.1	6.2	1	1998	81	0.3	0	3	0
I-70	070A	6.2	6.3	1	1998	85	0.3	0	0	0
I-70	070A	6.3	6.4	1	1998	85	0.3	0	0	12
I-70	070A	6.4	6.5	1	1998	103	0.3	0	2	0
I-70	070A	6.5	6.6	1	1998	78	0.3	0	2	0
I-70	070A	6.6	6.7	1	1998	93	0.4	8.3	14	0
I-70	070A	6.7	6.8	1	1998	86	0.4	0	0	12
I-70	070A	6.8	6.9	1	1998	71	0.4	0	0	0
I-70	070A	6.9	7	1	1998	92	0.4	0	0	12
I-70	070A	7	7.1	1	1998	95	0.4	0.3	0	0
I-70	070A	7.1	7.2	1	1998	90	0.4	0	0	0
I-70	070A	7.2	7.3	1	1998	76	0.4	0	0	0
I-70	070A	7.3	7.4	1	1998	93	0.3	0	0	0
I-70	070A	7.4	7.5	1	1998	74	0.4	0	3	12

Table A.3 Formatted pavement condition data along US-80 in Louisiana

Route	Control section	Mile points		Direction to survey	Year	IRI (in/mi)	Rut depth (in)	Alligator cracking (ft)	Longitudinal cracking (ft)	Transverse cracking (ft)
		Beginning	Ending							
US-80	001-05	0	0.1	1	2005	172	0.11	69	100	404
US-80	001-05	0.1	0.2	1	2005	171	0.1	120	140	470
US-80	001-05	0.2	0.3	1	2005	138	0.08	110	87	345
US-80	001-05	0.3	0.4	1	2005	136	0.06	51	148	278
US-80	001-05	0.4	0.	1	2005	241	0.15	43	194	400
US-80	001-05	0.5	0.6	1	2005	105	0.07	349	56	304
US-80	001-05	0.6	0.7	1	2005	127	0.11	381	0	330
US-80	001-05	0.7	0.8	1	2005	135	0.21	35	71	120
US-80	001-05	0.8	0.9	1	2005	136	0.14	86	38	56
US-80	001-05	0.9	1	1	2005	74	0.21	7	9	59
US-80	001-05	1	1.1	1	2005	105	0.25	55	0	31
US-80	001-05	1.1	1.2	1	2005	109	0.14	29	0	33
US-80	001-05	1.2	1.3	1	2005	119	0.15	54	38	81
US-80	001-05	1.3	1.4	1	2005	120	0.15	4	13	77
US-80	001-05	1.4	1.5	1	2005	107	0.12	0	16	79
US-80	001-05	1.5	1.6	1	2005	102	0.08	13	0	28
US-80	001-05	1.6	1.7	1	2005	118	0.12	134	35	93
US-80	001-05	1.7	1.8	1	2005	114	0.22	102	161	204
US-80	001-05	1.8	1.9	1	2005	-	0.32	12	86	64
US-80	001-05	1.9	2	1	2005	140	0.23	216	125	176
US-80	001-05	2	2.1	1	2005	192	0.21	49	0	24
US-80	001-05	2.1	2.2	1	2005	163	0.3	62	0	0
US-80	001-05	2.2	2.3	1	2005	217	0.41	48	0	8
US-80	001-05	2.3	2.4	1	2005	194	0.35	72	0	33
US-80	001-05	2.4	2.5	1	2005	191	0.31	0	0	0
US-80	001-05	2.5	2.6	1	2005	244	0.24	0	0	0
US-80	001-05	2.6	2.7	1	2005	259	0.4	12	0	0

Table A.4 Formatted pavement condition data along I-69BL in Michigan

Route	Control section	Mile points		Direction to survey	Year	IRI (in/mi)	Rut depth (in)	Alligator cracking (ft)	Longitudinal cracking (ft)	Transverse cracking (ft)
		Beginning	Ending							
I69BL	33043	0.000	0.100	1	2000	202	0.94	-	25	22
I69BL	33043	0.100	0.200	1	2000	143	0.59	-	41	18
I69BL	33043	0.200	0.300	1	2000	175	0.59	-	68	37
I69BL	33043	0.300	0.400	1	2000	258	0.58	-	80	36
I69BL	33043	0.400	0.500	1	2000	182	0.67	-	76	29
I69BL	33043	0.500	0.600	1	2000	188	0.65	-	81	33
I69BL	33043	0.600	0.700	1	2000	170	0.69	-	74	47
I69BL	33043	0.700	0.800	1	2000	112	0.36	-	76	38
I69BL	33043	0.800	0.900	1	2000	144	0.42	-	103	30
I69BL	33043	0.900	1.000	1	2000	189	0.61	-	106	56
I69BL	33043	1.000	1.100	1	2000	207	0.86	-	106	35
I69BL	33043	1.100	1.200	1	2000	166	0.67	-	34	35
I69BL	33043	1.200	1.300	1	2000	232	0.30	-	99	45
I69BL	33043	1.300	1.400	1	2000	173	0.52	-	59	9
I69BL	33043	1.400	1.500	1	2000	174	0.61	-	64	1
I69BL	33043	1.500	1.600	1	2000	89	0.33	-	57	-
I69BL	33043	1.600	1.700	1	2000	307	1.30	-	74	1
I69BL	33043	1.700	1.800	1	2000	243	1.23	-	73	34
I69BL	33043	1.800	1.900	1	2000	200	0.76	-	54	27
I69BL	33043	1.900	2.000	1	2000	100	0.18	-	35	4
I69BL	33043	2.000	2.100	1	2000	74	0.15	-	32	16
I69BL	33043	2.100	2.200	1	2000	116	0.16	-	61	-
I69BL	33043	2.200	2.300	1	2000	67	0.20	-	54	3
I69BL	33043	2.300	2.458	1	2000	61	0.43	-	107	14
I69BL	33043	2.458	2.558	1	2000	202	0.94	-	2	22
I69BL	33043	2.558	2.658	1	2000	143	0.59	-	25	18

Table A.5 Formatted pavement condition data along I-5 in Washington

Route	Control section	Mile points		Direction to survey	Year	IRI (in/mi)	Rut depth (in)	Alligator cracking (ft)	Longitudinal cracking (ft)	Transverse cracking (ft)
		Beginning	Ending							
I-5	-	5.3	5.4	1	2003	94	0.04	0	0	0
I-5	-	5.4	5.5	1	2003	123	0.04	0	132	0
I-5	-	5.5	5.6	1	2003	67	0.16	0	359.04	0
I-5	-	5.6	5.7	1	2003	68	0.31	0	73.92	0
I-5	-	5.7	5.8	1	2003	95	0.35	0	0	0
I-5	-	5.8	5.9	1	2003	77	0.31	0	0	0
I-5	-	5.9	6	1	2003	75	0.20	0	21.12	0
I-5	-	6	6.1	1	2003	81	0.31	137.28	15.84	0
I-5	-	6.1	6.2	1	2003	85	0.31	42.24	0	0
I-5	-	6.2	6.3	1	2003	91	0.46	0	42.24	0
I-5	-	6.3	6.4	1	2003	131	0.41	0	21.12	48
I-5	-	6.4	6.5	1	2003	89	0.24	0	163.68	12
I-5	-	6.5	6.6	1	2003	68	0.28	0	68.64	0
I-5	-	6.6	6.7	1	2003	67	0.24	0	63.36	0
I-5	-	6.7	6.8	1	2003	78	0.35	0	52.8	0
I-5	-	6.8	6.9	1	2003	73	0.28	0	0	0
I-5	-	6.9	7	1	2003	74	0.20	0	52.8	0
I-5	-	7	7.1	1	2003	95	0.12	0	132	0
I-5	-	7.1	7.2	1	2003	67	0.12	0	0	12
I-5	-	7.2	7.3	1	2003	85	0.12	0	0	0
I-5	-	7.3	7.4	1	2003	81	0.20	0	0	0
I-5	-	7.4	7.5	1	2003	106	0.31	0	0	12
I-5	-	7.5	7.6	1	2003	84	0.35	0	0	24
I-5	-	7.6	7.7	1	2003	85	0.24	10.56	42.24	48
I-5	-	7.7	7.8	1	2003	80	0.16	0	5.28	12
I-5	-	7.8	7.9	1	2003	68	0.04	0	0	0
I-5	-	7.9	8	1	2003	86	0.12	0	0	0

Table A.6 Formatted pavement condition data along cell 1 at MnROAD

Cell	Date	IRI (in/mi)	Rut depth (in)	Alligator cracking (ft)	Longitudinal cracking (ft)	Transverse cracking (ft)
1	11-FEB-94	94	0.04	0	0	0
1	21-OCT-94	123	0.04	0	0	0
1	20-APR-95	67	0.16	0	0	0
1	04-NOV-95	68	0.31	0	0	0
1	13-FEB-96	95	0.35	0	0	170
1	01-MAR-96	77	0.31	0	0	184
1	13-MAR-96	75	0.20	0	0	184
1	18-APR-96	81	0.31	0	0	190
1	13-NOV-96	85	0.31	0	0	190
1	30-APR-97	91	0.46	0	0	190
1	15-NOV-97	131	0.41	0	0	190
1	29-APR-98	89	0.24	0	0	191
1	02-OCT-98	68	0.28	0	0	2
1	20-APR-99	67	0.24	0	0	2
1	22-OCT-99	78	0.35	0	0	2
1	16-MAY-00	73	0.28	0	0	2
1	04-JAN-01	74	0.20	0	0	2
1	30-APR-01	95	0.12	0	0	2
1	23-OCT-01	67	0.12	0	0	2
1	20-APR-02	85	0.12	0	0	14
1	08-OCT-02	81	0.20	0	0	14
1	18-APR-03	106	0.31	0	40	38
1	15-OCT-03	84	0.35	0	20	38
1	21-APR-04	85	0.24	0	20	160
1	14-APR-05	80	0.16	0	5	204
1	17-OCT-05	68	0.04	0	5	208

APPENDIX B

ANALYZED DATA

The data analyzed as part of this study are colossal and would take many thousands of pages to include on paper. Hence, the analyzed data are available, by request, from the Department of Civil & Environmental Engineering at Michigan State University. To request the data, please contact:

Department of Civil & Environmental Engineering

C/O Dr. Gilbert Baladi

3546 Engineering Building

Michigan State University

East Lansing, MI 48824-1226

Tel: (517) 355-5107 Fax: (517) 432-1827 e-Mail: cee@egr.msu.edu

For convenience, all analyzed data for one project are included below. The analyzed data for the thin (< 2.5 inch) hot-mix asphalt (HMA) overlay project detailed in subsection 4.5.3 of Chapter 4 are provided below. The pavement project is located along a 9.5 mile long section of asphalt surfaced pavement (beginning mile points (BMPs) 198.1 to 207.6) of US-385, control section 385C, direction 1, in Colorado. The data analyses for the IRI are provided in Chapter 4, while the remaining pavement distresses and the controlling remaining service life (RSL) are detailed below. The three types of treatment transition matrices (T^2Ms) are listed, for the rut depth, alligator, longitudinal, and transverse cracking, and the controlling RSL, in Tables B.1 through B.15.

Table B.1 T^2M (rut depth, number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: rut depth												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the number of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (in)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (in)							
							0.09	0.11	0.10			
1	0 to 2	0	0		**0**	0	0	0	0			
2	3 to 5	0	0		0	**0**	0	0	0			
3	6 to 10	0	0		0	0	**0**	0	0			
4	11 to 15	2	4	0.04	0	0	0	**0**	2	5	7	20
5	16 to 25	52	96	0.02	0	0	3	2	**47**	2	-1	19
Total/average		54/	100/		0/	0/	3/	2/	49/	/2	/-1	/19

Table B.2 T^2M (rut depth, percent of the number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: rut depth												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (in)	1	2	3	4	5	Treatment life	Service life extension	RSL
		Number	Percent		0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
					SE of each RSL bracket (in)							
							0.09	0.11	0.10			
1	0 to 2	0	0									
2	3 to 5	0	0									
3	6 to 10	0	0									
4	11 to 15	2	4	0.04	0	0	0	**0**	100	5	7	20
5	16 to 25	52	96	0.02	0	0	6	4	**90**	2	-1	19
Total/average		54/	100/		0/	0/	6/	4/	91/	/2	/-1	/19

Table B.3 T^2M (rut depth, percent of the number of pavement segments of the treatment network), US-385, control section 385C, direction 1, Colorado

Condition/distress type: rut depth												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from the pavement network to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (in)	1	2	3	4	5	Treatment life	Service life extension	RSL
		Number	Percent		0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
					SE of each RSL bracket (in)							
							0.09	0.11	0.10			
1	0 to 2	0	0									
2	3 to 5	0	0									
3	6 to 10	0	0									
4	11 to 15	2	4	0.04	0.00	0.00	0.00	**0.00**	3.70	5	7	20
5	16 to 25	52	96	0.02	0.00	0.00	5.56	3.70	**87.04**	2	-1	19
Total/average		54/	100/		0/	0/	6/	4/	91/	/2	/-1	/19

Table B.4 T²M (alligator cracking, number of pavement segments), US-385, control section 385C, direction 1, Colorado

<table>
<tr><th colspan="14">Condition/distress type: alligator cracking</th></tr>
<tr><td colspan="5" rowspan="2">Before treatment (BT) data</td><td colspan="9">After treatment (AT) data</td></tr>
<tr><td colspan="5">RSL bracket number and range in years, the standard error per RSL bracket, and the number of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets</td><td colspan="3">Treatment life, service life extension, and RSL of the treatment (year)</td></tr>
<tr><td rowspan="4">RSL bracket number</td><td rowspan="4">RSL bracket range (year)</td><td colspan="2" rowspan="2">0.1 mile pavement segments</td><td rowspan="4">Standard error (SE) (ft)</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td rowspan="4">Treatment life</td><td rowspan="4">Service life extension</td><td rowspan="4">RSL</td></tr>
<tr><td>0 to 2</td><td>3 to 5</td><td>6 to 10</td><td>11 to 15</td><td>16 to 25</td></tr>
<tr><td rowspan="2">Number</td><td rowspan="2">Percent</td><td colspan="5">SE of each RSL bracket (ft)</td></tr>
<tr><td></td><td>30</td><td>13</td><td>10</td><td>10</td></tr>
<tr><td>1</td><td>0 to 2</td><td>30</td><td>70</td><td>9</td><td>0</td><td>8</td><td>10</td><td>3</td><td>9</td><td>7</td><td>10</td><td>11</td></tr>
<tr><td>2</td><td>3 to 5</td><td>4</td><td>9</td><td>1</td><td>0</td><td>1</td><td>0</td><td>1</td><td>2</td><td>5</td><td>10</td><td>14</td></tr>
<tr><td>3</td><td>6 to 10</td><td>3</td><td>7</td><td>0</td><td>0</td><td>0</td><td>1</td><td>0</td><td>2</td><td>6</td><td>8</td><td>16</td></tr>
<tr><td>4</td><td>11 to 15</td><td>2</td><td>5</td><td>1</td><td>0</td><td>0</td><td>1</td><td>0</td><td>1</td><td>4</td><td>1</td><td>14</td></tr>
<tr><td>5</td><td>16 to 25</td><td>4</td><td>9</td><td>1</td><td>0</td><td>0</td><td>1</td><td>1</td><td>2</td><td>4</td><td>-5</td><td>15</td></tr>
<tr><td colspan="2">Total/average</td><td>43/</td><td>100/</td><td></td><td>0/</td><td>9/</td><td>13/</td><td>5/</td><td>16/</td><td>/6</td><td>/8</td><td>/12</td></tr>
</table>

Table B.5 T²M (alligator cracking, percent of the number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: alligator cracking													
Before treatment (BT) data					After treatment (AT) data								
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
		Number	Percent		0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
					SE of each RSL bracket (ft)							
						30	13	10	10			
1	0 to 2	30	70	9	**0**	27	33	10	30	7	10	11
2	3 to 5	4	9	1	0	**25**	0	25	50	5	10	14
3	6 to 10	3	7	0	0	0	**33**	0	67	6	8	16
4	11 to 15	2	5	1	0	0	50	**0**	50	4	1	14
5	16 to 25	4	9	1	0	0	25	25	**50**	4	-5	15
Total/average		43/	100/		0/	21/	30/	12/	37/	/6	/8	/12

Table B.6 T²M (alligator cracking, percent of the number of pavement segments of the treatment network), US-385, control section 385C, direction 1, Colorado

Condition/distress type: alligator cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from the pavement network to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
						30	13	10	10			
1	0 to 2	30	70	9	**0.00**	18.60	23.26	6.98	20.93	7	10	11
2	3 to 5	4	9	1	0.00	**2.33**	0.00	2.33	4.65	5	10	14
3	6 to 10	3	7	0	0.00	0.00	**2.33**	0.00	4.65	6	8	16
4	11 to 15	2	5	1	0.00	0.00	2.33	**0.00**	2.33	4	1	14
5	16 to 25	4	9	1	0.00	0.00	2.33	2.33	**4.65**	4	-5	15
Total/average		43/	100/		0/	21/	30/	12/	37/	/6	/8	/12

Table B.7 T^{2}M (longitudinal cracking, number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: longitudinal cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the number of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
							45	26	77			
1	0 to 2	11	61	59	**0**	0	1	3	7	8	16	17
2	3 to 5	2	11	15	0	**0**	1	0	1	6	10	14
3	6 to 10	2	11	7	0	0	**1**	1	0	4	3	11
4	11 to 15	0	0		0	0	0	**0**	0			
5	16 to 25	3	17	15	0	0	1	0	**2**	2	-4	16
Total/average		18/	100/		0/	0/	4/	4/	10/	/6	/11	/16

Table B.8 T^2M (longitudinal cracking, percent of the number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: longitudinal cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
							45	26	77			
1	0 to 2	11	61	59	**0**	0	9	27	64	8	16	17
2	3 to 5	2	11	15	0	**0**	50	0	50	6	10	14
3	6 to 10	2	11	7	0	0	**50**	50	0	4	3	11
4	11 to 15	0	0									
5	16 to 25	3	17	15	0	0	33	0	**67**	2	-4	16
Total/average		18/	100/		0/	0/	22/	22/	56/	/6	/11	/16

Table B.9 T^{2}M (longitudinal cracking, percent of the number of pavement segments of the treatment network), US-385, control section 385C, direction 1, Colorado

Condition/distress type: longitudinal cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from the pavement network to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
							45	26	77			
1	0 to 2	11	61	59	**0.00**	0.00	5.56	16.67	38.89	8	16	17
2	3 to 5	2	11	15	0.00	**0.00**	5.56	0.00	5.56	6	10	14
3	6 to 10	2	11	7	0.00	0.00	**5.56**	5.56	0.00	4	3	11
4	11 to 15	0	0									
5	16 to 25	3	17	15	0.00	0.00	5.56	0.00	**11.11**	2	-4	16
Total/average		18/	100/		0/	0/	22/	22/	56/	/6	/11	/16

Table B.10 T^2M (transverse cracking, number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: transverse cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the number of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
					70	8	127	87	58			
1	0 to 2	20	47	123	**1**	1	5	4	9	7	13	14
2	3 to 5	7	16	42	0	**0**	3	1	3	2	10	14
3	6 to 10	9	21	26	0	0	**2**	0	7	1	9	17
4	11 to 15	3	7	18	0	0	1	**0**	2	2	3	16
5	16 to 25	4	9	45	0	0	0	0	**4**	1	0	20
Total/average		43/	100/		1/	1/	11/	5/	25/	/4	/10	/15

Table B.11 T^2M (transverse cracking, percent of the number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: transverse cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
					70	8	127	87	58			
1	0 to 2	20	47	123	**5**	5	25	20	45	7	13	14
2	3 to 5	7	16	42	0	**0**	43	14	43	2	10	14
3	6 to 10	9	21	26	0	0	**22**	0	78	1	9	17
4	11 to 15	3	7	18	0	0	33	**0**	67	2	3	16
5	16 to 25	4	9	45	0	0	0	0	**100**	1	0	20
Total/average		43/	100/		2/	2/	26/	12/	58/	/4	/10	/15

Table B.12 T^2M (transverse cracking, percent of the number of pavement segments of the treatment network), US-385, control section 385C, direction 1, Colorado

Condition/distress type: transverse cracking												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from the pavement network to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (ft)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE of each RSL bracket (ft)							
					70	8	127	87	58			
1	0 to 2	20	47	123	**2.33**	2.33	11.63	9.30	20.93	7	13	14
2	3 to 5	7	16	42	0.00	**0.00**	6.98	2.33	6.98	2	10	14
3	6 to 10	9	21	26	0.00	0.00	**4.65**	0.00	16.28	1	9	17
4	11 to 15	3	7	18	0.00	0.00	2.33	**0.00**	4.65	2	3	16
5	16 to 25	4	9	45	0.00	0.00	0.00	0.00	**9.30**	1	0	20
Total/average		43/	100/		2/	2/	26/	12/	58/	/4	/10	/15

Table B.13 T^{2}M (controlling RSL, number of pavement segments), US-385, control section 385C, direction 1, Colorado

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the number of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	52	59		**1**	20	27	1	3	3	6	7
2	3 to 5	17	19		0	**3**	11	3	0	2	4	8
3	6 to 10	6	7		0	0	**3**	1	2	3	5	13
4	11 to 15	2	2		0	0	1	**1**	0	-4	-3	11
5	16 to 25	11	13		0	0	2	0	**9**	3	-2	18
Total/average		88/	100/		1/	23/	44/	6/	14/	/3	/4	/9

Table B.14 T^{2}M (controlling RSL, percent of the number of pavement segments), US-385, control section 385C, direction 1, Colorado

<table>
<tr><th colspan="13">Condition/distress type: condition/distress causing the minimum RSL before and after treatment</th></tr>
<tr><td colspan="5" rowspan="2">Before treatment (BT) data</td><td colspan="8">After treatment (AT) data</td></tr>
<tr><td colspan="5">RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from each BT RSL bracket to the indicated RSL brackets</td><td colspan="3">Treatment life, service life extension, and RSL of the treatment (year)</td></tr>
<tr><td rowspan="3">RSL bracket number</td><td rowspan="3">RSL bracket range (year)</td><td colspan="2" rowspan="2">0.1 mile pavement segments</td><td rowspan="8">Standard error (SE) (cannot be calculated for the minimum RSL)</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td rowspan="3">Treatment life</td><td rowspan="3">Service life extension</td><td rowspan="3">RSL</td></tr>
<tr><td>0 to 2</td><td>3 to 5</td><td>6 to 10</td><td>11 to 15</td><td>16 to 25</td></tr>
<tr><td>Number</td><td>Percent</td><td colspan="5">SE (cannot be calculated for the minimum RSL)</td></tr>
<tr><td>1</td><td>0 to 2</td><td>52</td><td>59</td><td>2</td><td>38</td><td>52</td><td>2</td><td>6</td><td>3</td><td>6</td><td>7</td></tr>
<tr><td>2</td><td>3 to 5</td><td>17</td><td>19</td><td>0</td><td>18</td><td>65</td><td>18</td><td>0</td><td>2</td><td>4</td><td>8</td></tr>
<tr><td>3</td><td>6 to 10</td><td>6</td><td>7</td><td>0</td><td>0</td><td>50</td><td>17</td><td>33</td><td>3</td><td>5</td><td>13</td></tr>
<tr><td>4</td><td>11 to 15</td><td>2</td><td>2</td><td>0</td><td>0</td><td>50</td><td>50</td><td>0</td><td>-4</td><td>-3</td><td>11</td></tr>
<tr><td>5</td><td>16 to 25</td><td>11</td><td>13</td><td>0</td><td>0</td><td>18</td><td>0</td><td>82</td><td>3</td><td>-2</td><td>18</td></tr>
<tr><td colspan="2">Total/average</td><td>88/</td><td>100/</td><td></td><td>1/</td><td>26/</td><td>50/</td><td>7/</td><td>16/</td><td>/3</td><td>/4</td><td>/9</td></tr>
</table>

Table B.15 T^2M (controlling RSL, percent of the number of pavement segments of the treatment network), US-385, control section 385C, direction 1, Colorado

Condition/distress type: condition/distress causing the minimum RSL before and after treatment												
Before treatment (BT) data					After treatment (AT) data							
					RSL bracket number and range in years, the standard error per RSL bracket, and the percent of the 0.1 mile pavement segments transitioned from the pavement network to the indicated RSL brackets					Treatment life, service life extension, and RSL of the treatment (year)		
RSL bracket number	RSL bracket range (year)	0.1 mile pavement segments		Standard error (SE) (cannot be calculated for the minimum RSL)	1	2	3	4	5	Treatment life	Service life extension	RSL
					0 to 2	3 to 5	6 to 10	11 to 15	16 to 25			
		Number	Percent		SE (cannot be calculated for the minimum RSL)							
1	0 to 2	52	59		**1.14**	22.73	30.68	1.14	3.41	3	6	7
2	3 to 5	17	19		0.00	**3.41**	12.50	3.41	0.00	2	4	8
3	6 to 10	6	7		0.00	0.00	**3.41**	1.14	2.27	3	5	13
4	11 to 15	2	2		0.00	0.00	1.14	**1.14**	0.00	-4	-3	11
5	16 to 25	11	13		0.00	0.00	2.27	0.00	**10.23**	3	-2	18
Total/average		88/	100/		1/	26/	50/	7/	16/	/3	/4	/9

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