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16. Abstract With ever increasing traffic loadings, highway pavement maintenance needs continue to outpace the availability of resources, and transportation agencies seek cost-effective maintenance practices. This study investigated the effectiveness of maintenance treatments in the short-term and the cost effectiveness of maintenance strategies over entire pavement life. The study also analyzed the relationships and trade-offs between maintenance and capital investments such as pavement rehabilitation, and the trade-offs between preventive and corrective maintenance. These analyses were carried out through a work sequence that included analyses of historical trends, literature review, and a questionnaire survey. The study found that there are significant benefits associated with maintenance treatments, and that such short-term impacts generally involve an increase in pavement condition or a decrease in the rate of deterioration. For most treatments, a greater benefit is generally obtained for a larger effort expended on the maintenance treatment, at a given level of pavement condition, up to a point. The study also found that if chosen appropriately, maintenance strategies could be cost-effective in the long run. The most cost-effective strategy was determined for each pavement family. Finally, the study determined that trade-off relationships exist between intervals of capital investments on one hand, and maintenance, traffic loading, and weather on the other hand: up to a point, increasing maintenance leads to increased rehabilitation interval, while increasing traffic loads and weather severity leads to reduction in rehabilitation interval, albeit at different rates for each pavement family. Marginal effects models were used to determine the effect of unit changes in maintenance levels, traffic loading, and weather on changes in rehabilitation interval. This information is useful not only for pavement management, but also for policy analyses involving truck weights, and pavement repair needs assessment to reflect changing traffic and weather conditions in the long-term. The data for the study was supplied by the Indiana Department of Transportation.			
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FINAL REPORT

FHWA/IN/JTRP-2002/27

THE EFFECTIVENESS OF MAINTENANCE
AND ITS IMPACT ON CAPITAL EXPENDITURES

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Federal Highway Administration and the Indiana Department of Transportation. The report does not constitute a standard, a specification, or a regulation.

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LIST OF ACRONYMS

AAADT-	Average Annual Daily Traffic
AAMEX-	Average Annual Maintenance Expenditure
AC-	Asphaltic Concrete
DRL-	Deterioration Reduction Level
DRR-	Deterioration Rate Reduction
ESAL-	Equivalent Single Axle Load
FDA-	Full-depth Asphalt
FHWA-	Federal Highway Administration
HMAC-	Hot Mix Asphalt Concrete
ICC-	Indiana Climatic Center
INDOT-	Indiana Department of Transportation
IRI-	International Roughness Index
JPCP-	Jointed Plain Concrete Pavement
JRCP-	Jointed Reinforced Concrete Pavement
LTPP-	Long-Term Performance Project
M&R-	Maintenance and Rehabilitation
MLW-	Maintenance-Load-Weather Variable
MMS-	Maintenance Management System
PCC-	Portland Cement Concrete
PJ-	Performance Jump
PMS-	Pavement Management System
PSI-	Present Serviceability Index

CHAPTER 1 INTRODUCTION

1.1 Background and Problem Statement

The highway infrastructure network in the United States is indeed a vital factor in the nation's economic and social development. Among all modes, highways account for a significant share of total freight value and ton-miles of freight. Furthermore, given the inter-modal nature of modern passenger and freight transportation in the state, other modes of transportation depend heavily on the highway network as an origin, intermediate, or destination link in their delivery of transportation services. Among the various elements of highway infrastructure, highway pavements are associated with the highest levels of expenditure, typically accounting for about 30-50% of total highway expenditure [FHWA, 1993-1999]. After decades of use, highway pavements in the state of Indiana and the United States in general increasingly suffer from aging of pavement materials, weather effects, and high levels of accumulated usage. This situation is exacerbated by the uncertainty of sustained adequate funds for rehabilitation and maintenance.

Several recent federal legislations have emphasized the critical need of keeping existing pavements properly maintained. For example, the 1978 Surface Transportation Assistance Act authorized over \$1.2 billion in federal funds for rehabilitation, resurfacing and restoration of the Interstate system over a 3-year period. Also, relatively recent legislation such as the 1991 Intermodal Surface Transportation Efficiency Act (ISTEA), the 1995 National Highway System Designation Act, and the 1998 Transportation Equity Act for the 21st Century (TEA-21) all recognized the importance of pavement maintenance and made provisions for funding such activities on all Federal-aid highways. Thanks to such interventions, the deterioration of highway pavements over the past decade has slowed somewhat, especially after 1994. Notwithstanding such encouraging funding trends, there remains a backlog of highway pavement

maintenance and rehabilitation. Therefore pavement managers in various states face the challenge of ensuring that maintenance and rehabilitation funds are used as efficiently and effectively as possible. In Indiana, for instance, maintenance of highway pavements accounts for a large portion of state and local transportation agency budgets. Every year, the state of Indiana incurs several hundred millions of dollars in capital works and maintenance for state highway pavements [FHWA, 1993-1999]. It is not certain that the state will be able to sustain adequate funding of pavement maintenance activities to ensure acceptable levels of service on its entire network. In this respect, it is sought to identify and implement any measures aimed at increasing the cost-effectiveness of the state's pavement maintenance activities so that maximum benefit can be wrought from each dollar expended on such activities. Considering the level of funding of such activities every year in Indiana, it is apparent that even marginal improvements in the effectiveness of maintenance investments may result in very large absolute dollar savings.

Relatively little work on the assessment of benefits of maintenance (especially of preventive type) in reducing the frequency of capital investment. The present study investigates the impacts of pavement maintenance of capital investments such as resurfacing. Traditional maintenance practices have focused primarily on activities of a structural or corrective nature. However, the notion of performing maintenance prior to the onset of significant deterioration is getting increased attention among highway pavement managers because such preventive treatments not only potentially increase performance and service life, but also show much promise in reducing long-term costs of highway facilities. Analogies can be drawn in the area of vehicle maintenance, where tune-ups, oil changes and other preventive maintenance are carried out at predetermined intervals such as every year or every 3000 miles in order to ensure longer vehicle life. Another analogy can be found in the field of medicine, where the benefits of preventive health activities to the human body are all too obvious. Indeed, "a stitch in time saves nine" is an apt admonition for highway pavement managers.

If preventive maintenance is applied too frequently or is applied long before it is really needed, it is uneconomical. On the other hand, if it is delayed for too long, user benefits are reduced and repair costs increase drastically. Optimal timing of preventive maintenance therefore requires an adequate balance between sustained performance on one hand, and increased maintenance costs on the other. For the

purposes of the present study, preventive maintenance is defined as “a planned strategy of cost-effective treatments to an existing roadway system that preserves the system, retards future deterioration, and maintains and improves the functional condition of the system, without substantially increasing the structural capacity” [Geoffroy, 1996]. With the focus of preventive maintenance activities on NHS engendered by legislation, the scope and importance of preventive maintenance has burgeoned significantly. Sub-district highway agencies perform not only on-demand activities but also carry out preventive treatments such as crack sealing and seal coating. Indeed the term “routine maintenance”, in its original connotation as work done by sub-districts, may well be on its way to obsolescence, and seems to have been gradually replaced by the term “demand maintenance”.

Functional pavement distresses are surface defects that have no immediate impact on the load bearing ability of the pavement, such as cracks, raveled surfaces, and ruts. Structural, unlike functional pavement distresses (such as alligator cracking) have immediate impact on the load bearing ability of the pavement. Notwithstanding the literal implication of the word “preventive”, preventive maintenance treatments on highway pavements play roles that vary from truly preventive to remedial, as indicated in [Figure 1](#). “Remedial” in the context of preventive maintenance, refers to correction of only functional distresses, and is used in lieu of the term “corrective”, as the latter term is often associated with treatments that correct structural distresses. Also, preventive maintenance may be categorized not only by the type of treatment, but also by the role that treatment plays in relation to the condition of the pavement at the time of treatment [Mamlouk et al., 2000]. For instance, application of a chip seal to a pavement with little or no cracks may be considered as a purely preventive treatment, while the same application to a severely cracked and raveled pavement may be viewed as preventive maintenance of a remedial nature. In reality there is often a thin line between preventive and corrective maintenance. The role of certain remedial preventive maintenance treatments, such as under-sealing, borders on the corrective, while certain corrective treatments such as shallow patching, forestalls further worsening of distress and could probably be described as preventive. [Figure 1-1](#) illustrates the different roles played by various preventive and corrective maintenance activities needed to address a range of road condition levels. As it is relatively easy

to program preventive treatments compared to corrective treatments, the former are described as having a higher possibility of being programmed compared to the latter.

Thin Hot Mix Asphaltic Concrete (HMAC) overlays and seal coating (using sand or chips), are termed “major” and “moderate” preventive maintenance activities, respectively, for the purposes of the present study. Both treatments typically cover the entire surface of the pavement, while crack sealing and shallow patching, which treat only a specific distressed spot or area, are referred to as “minor” or “localized” treatments in the present study. Details of treatment types and categories are discussed in subsequent chapters of this report.

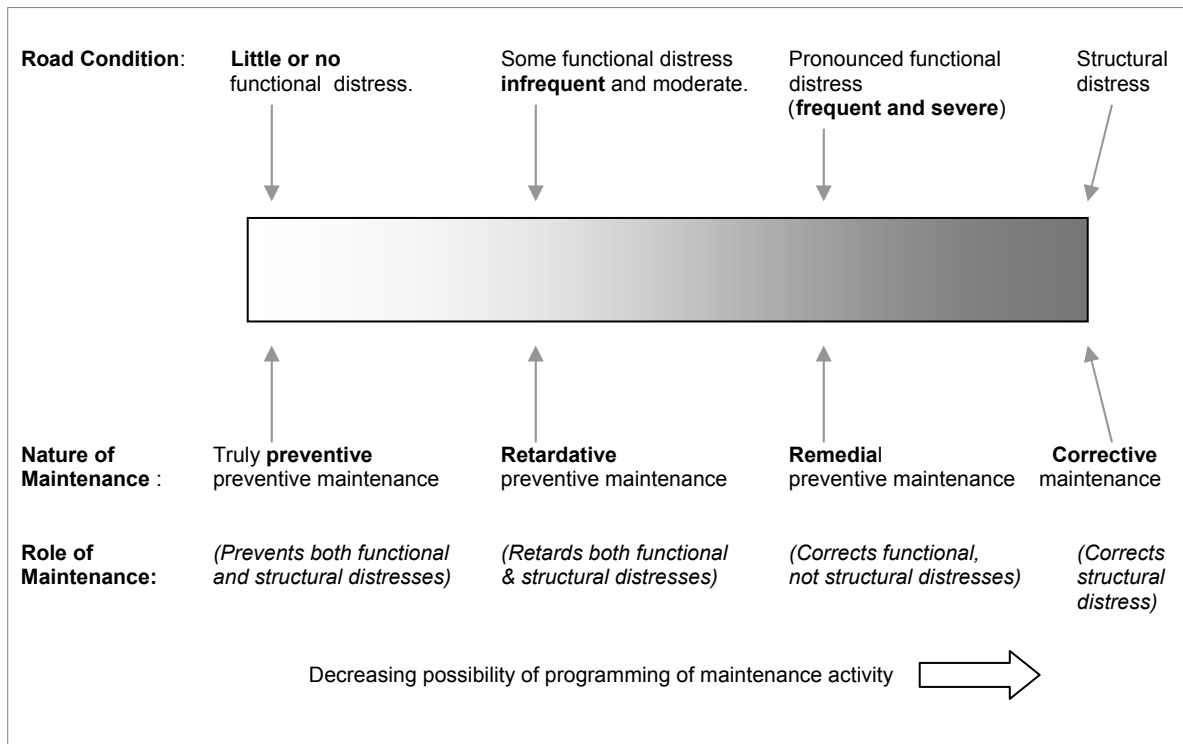


Figure 1-1: Distress/Maintenance Spectrum

Selection of the best maintenance treatment at a given time, or the best strategy over a period of time, depends not only on the type of facility, traffic, location, and environmental conditions, but may also be influenced by regional variations in maintenance policy and costs, preventive maintenance techniques, and how pavement performance is measured [Mamlouk and Zaniewski, 1998]. “Timing” of maintenance

activities may refer to the frequency of application or the distress occurrence threshold to trigger application [Peterson, 1989].

Currently, all state highway agencies invariably practice preventive maintenance. The differences in terminology and definitions used by various state and local highway agencies are worth noting [Geoffroy, 1996]. In order to avoid possible problems posed by such inconsistencies, terminologies adopted or approved by the Indiana Department of Transportation are used in the present study. For rigid pavements, typical preventive treatments include crack and joint sealing, and under-sealing and stitching, while crack sealing, grinding, sand and chip-seals, micro-surfacing and thin overlay are used on flexible pavements. At the present time, the selection of the appropriate treatment type, application timing and frequency of preventive maintenance applications are commonly made based on the experience of maintenance supervisors or district and sub-district engineers. Maintenance decisions are therefore generally made without following any established and consistent agency-wide set of guidelines. Furthermore, as no such documentation is available to train new personnel, knowledge acquired over the years may be lost due to staff turnover. An ideal standard set of guidelines could be developed from the accumulated and collective knowledge and experience of highway maintenance and management personnel and researchers throughout the country, supported by relevant research results. The use of any such guidelines would greatly facilitate the optimization of maintenance treatment type and timing, ensure consistency of practice, and could result in substantial savings. With the completion of the present study, the development of such guidelines could be facilitated.

In summation, there is a need to establish a clear definition of maintenance treatments in use today and to assess both short and long-term effectiveness of overall preventive maintenance treatments and strategies, respectively. It is also important to investigate any tradeoff relationships between maintenance and the frequency of capital investments, for various pavement families exposed to various environmental and loading conditions. This would, among others, serve as a basis for developing a set of recommendations for maintenance practices that could be used by a highway agency in the planning of its pavement repair activities.

1.2 Current Status of Pavement-related Management Systems in Indiana

1.2.1 Pavement Management System (PMS)

Indiana's current PMS applies the principles of engineering, management, information technology, and operations research in a bid to make consistent and sound decisions that would ensure maximum cost-effectiveness of maintenance and rehabilitation (M&R) activities on the state's highway pavements, and to facilitate short- and long-term planning and budgeting. The primary questions that INDOT's PMS seeks to answer are as follows:

- What kind of M&R activity should be carried out at any given section, and when?
- What should the minimum road condition be to trigger such an activity (or at what time intervals should that activity be carried out)?
- What are the consequences of carrying out (or not carrying out) a specified activity?

Indiana's PMS consists of three modules: Database, Analysis, and Feedback. The database contains data on road characteristics (geometry, condition, layer types, etc.). The present study draws on the vast amount of data made available by Indiana's PMS. In its current form, the database for INDOT's pavement management system has data on pavement section referencing, condition, and traffic volumes, but lacks some data types such as details of pavement maintenance, speeds, climate data and other data that might be useful in explaining the effectiveness of maintenance in the short- and long-term. Therefore, there was a need to develop a new database using valuable information already contained in INDOT's PMS database as a nucleus. The results of all three aspects of the present study (short-term effectiveness evaluation, long-term evaluation, and trade-off analysis) are potentially useful to the PMS.

1.2.2 The Maintenance Management System (MMS)

Indiana's Maintenance Management System is a comprehensive system that helps INDOT's Operations Support Division to plan and program various force-account pavement maintenance activities on the state road network. MMS activities are implemented at the sub-district level, and the type of work includes a diverse range of maintenance categories such as non-pavement maintenance activities (grass

mowing, culvert cleaning, etc.), pavement preventive maintenance (joint sealing, chip sealing, etc.), and pavement corrective maintenance (such as shallow patching).

Indiana's MMS helps sub-district highway departments to maintain the roadways in a safe and motorable condition on a day-to-day basis. Work may be scheduled or unscheduled. Work items under the INDOT's MMS currently include the following [Burkhardt and Goode, 1991; INDOT, 1992]:

- Developing annual work programs and budgets
- Distributing work on an annual basis
- Determining labor, equipment, and material needs
- Monitoring complaints and inspection reports
- Scheduling work
- Recording work accomplished, and evaluating performance

System features and components that assist in the execution of the above tasks include a standardized listing and description of maintenance activities in INDOT's Field Operations Manual [INDOT, 1998]. Also included in the MMS is an inventory of physical assets being maintained, performance standards (resource requirements and estimated production rates), work programs and performance budgets, work calendars, the guidelines for the use of crew day cards, and management reports.

In light of changing trends in resource availability, management and technology, most current maintenance management systems have been deemed inadequate to meet future expectations and therefore need updating with regard to three broad areas. One of such requirements is the incorporation of new types of analyses with maintenance management planning, such as trade-off between maintenance and capital activities, and the impacts of deferred maintenance [Markow et al., 1994].

It is expected that the evaluation of short-term impacts of maintenance activities, which is addressed in the first part of the present study, would especially benefit INDOT's Operations Support Division, operators of the state's Maintenance Management System.

1.2.3 The Total Highway Asset Management System (THAMS)

The concept of a total highway management system, advocated several years ago by Fwa and Sinha [1988], is finally catching on among highway administrators in the United States as the effective way to collectively and efficiently manage an increasingly complex collection of various management systems at state or local level. Several years later, NCHRP Report 363 called for an integration of all highway related management systems, in response to changing technological and managerial trends [Markow et al., 1994]. In other studies, it was stated that information collected from PMS, MMS, and Bridge Management System (BMS) are extremely important for effective overall maintenance budgeting [Reno et al., 1994; Sparks et al., 1994]. Analysis of trade-offs between maintenance and rehabilitation, and also between various maintenance categories has been touted as an important aspect of any total highway management system.

Currently, the concept of total highway asset management has not yet been implemented in Indiana, even though work on this system has begun in earnest. It is hoped that with the completion of the present study, the awareness of the benefits of a total highway asset management system to the operation of its component systems, i.e., PMS, MMS, and BMS, will be raised even further.

1.3 Objectives of the Present Study

Any maintenance effectiveness research effort is generally expected to aid in further development of existing Pavement Management Systems and Maintenance Management Systems, and enhancing the integration of both systems into a Total Highway Asset Management System, as indicated in the previous section. The realization of this general objective would ensure quick, efficient and effective development of budgets, monitoring of maintenance and rehabilitation spending, and proper administration of pavement construction and maintenance programs and resource allocation at all jurisdictional levels of highway pavement management. In the course of addressing the above issue, it is expected that several specific research objectives and issues will be realized. These include:

- Design and implementation of methodologies to evaluate short-term effectiveness of various maintenance treatments,

- Enhancement of existing pavement performance models currently used by INDOT,
- Development of annual maintenance expenditure models for each pavement type,
- Development of treatment unit accomplishment cost models for each maintenance treatment type,
- Determination of the cost effectiveness of long-term maintenance strategies for various pavement families,
- Investigation of any trade-off relationship between various levels of maintenance and the frequency of capital investment, and also between preventive and corrective maintenance,
- Investigation of marginal effects of maintenance, load and weather on the frequency of capital investment,
- Development of a set of recommendations that would assist INDOT in the selection of types and optimal timings for appropriate maintenance treatments, for each category of pavement, such that overall cost-effectiveness is maximized in the long-run.

1.4 Scope of the Study

The scope of the study is as follows:

- *Coverage*: The present study focuses on the state highway system in Indiana, and evaluates maintenance effectiveness on a project-level basis. A sample of 5,000+ one-mile pavement segments (representing over 50% of the entire state highway system) is used, and each 1-mile segment is used as the primary statistical unit for the analyses.
- *Analysis Period*: The study period ranges from 1991 to 1999, as this was the common overlap of availability of existing data from various sources. In some cases, data was obtained for the period prior to 1991 in order to carry out the trade-off analyses between rehabilitation interval and maintenance.
- *Pavement Type*: With regard to surface layer material, pavement types considered are: full-depth asphaltic (FDA) concrete, rigid pavement (plain or reinforced, and jointed or

continuous concrete), and PCC-over-AC overlays (OVR). Full-depth asphaltic concrete pavements were not further categorized as single-layer or multi-layer due to lack of such data.

- *Geo-climatic Region*: The present study utilized data from pavement sections that were located at various locations in the state, without any geographical restriction. Using multivariate statistical techniques, three regions of the state were identified such that within each region there is minimum variation of temperature and rainfall patterns while maximum variation exist from one region to another. The analyses yielded boundaries of an approximately horizontal nature, and the resulting regions were therefore designated as Northern, Central, and Southern. This regional demarcation of the state roughly coincides with an approximate regionalization based on terrain and soil (subgrade) type, and highway administration jurisdiction. Notwithstanding this regional demarcation, most aspects of the study utilized a single index derived to represent relative weather severity in each county.
- *Treatment Types*: On the basis of current practice, the following preventive maintenance treatment types were considered: relatively “minor” or “local” activities such as crack sealing, “moderate” maintenance activities such as chip sealing, and “major” activities such as thin HMAC overlays. Other activities such as under-sealing and stitching were not considered due to lack of data. Corrective (or reactive or on-demand) maintenance activities were not explicitly considered as they are typically carried out in response to local structural defects and are consequently not considered in strategy development.

1.5 **Overview of Study Approach**

The entire research study, the approach for which is shown as Figure 1-2, was carried out in four parts, and was designed in manner to address the stated study objectives in a sequential fashion. While an overview of each part of the study is provided below, details of the framework for each part of the study are provided in the prelude to each chapter of this report.

1.5.1 Information Search and Methodology

After establishing the study objectives, the first part of the present study included a literature review, agency survey, and data collection. Past and current preventive maintenance and rehabilitation practices by domestic and international highway agencies at various jurisdictional levels were documented and evaluated for treatment criteria, application conditions, and performance. A review was made of studies that have been carried out in this subject area by various agencies and institutions, and updated information on the causes and mechanisms of pavement deterioration was reviewed. Then, a list of standard preventive maintenance treatments for pavement under various conditions was made by documenting the nature of such practices in various INDOT districts and sub-districts with due cognizance given to differences in terminology. A survey of pavement managers at INDOT's district and sub-districts was also conducted. This was followed by a comprehensive data collection effort. Collected data was collated and synthesized using a common referencing scheme, and published as INDIPAVE 2000. The final aspect of this part of the study was the review of established theoretical concepts and methods needed to address study objectives.

1.5.2 Evaluation of Maintenance Effectiveness in the Short-term

This part of the study deals with the evaluation and comparison of the short-term effectiveness of the various major categories and types of pavement maintenance. Table 1-1 below shows the pertinent questions related to short-term effectiveness and how these questions were addressed through modeling and other means.

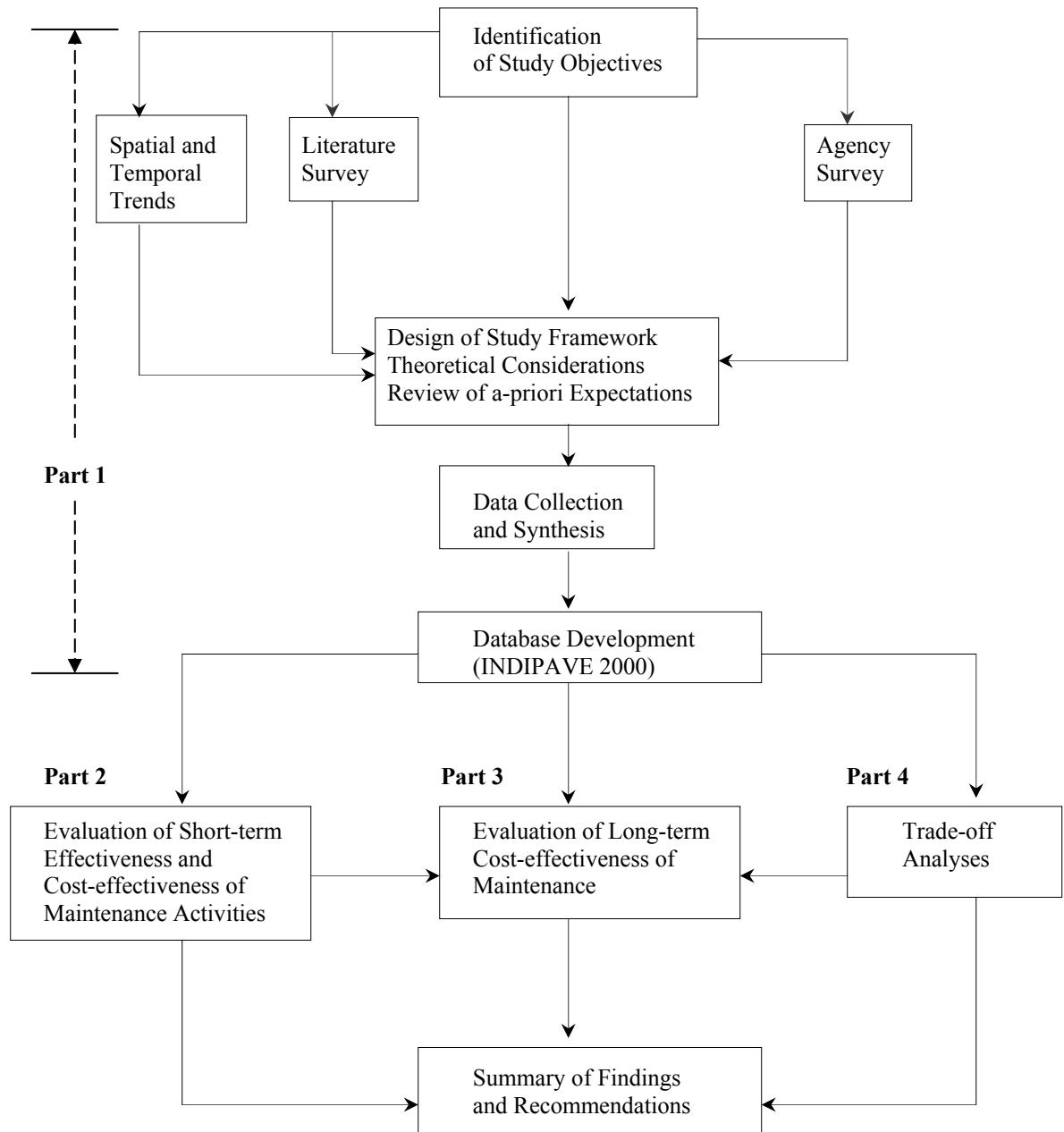


Figure 1-2: Overall Study Approach

Table 1-1: Questions Addressed by Evaluation of Maintenance Effectiveness in the Short-term

Questions	How Questions were Answered
1. What is the estimated average annual amount expended (per lane-mile) on pavement maintenance?	<u>Average Annual Pavement Maintenance Expenditure Modeling</u> Response Variable: Maintenance Expenditure (\$ per lane-mile of pavement) Explanatory Variables: Pavement Type, Age Group, Region, etc.
2. What is the estimated short-term effectiveness of each standard maintenance treatment?	
i) How do we measure effectiveness? →	Deterioration Reduction Level (DRL) vs. Deterioration Rate Reduction (DRR) vs. Performance Jump (PJ)
ii) Is maintenance effective in the short-term? (Does maintenance have a need?)	
iii) What factors affect effectiveness of maintenance in the short-term?	Descriptive Statistics of Available Data DRL/DRR due to Maintenance, by Age Group, and Pavement Type, etc.
iv) What is the relative impact of the factors identified in (iii) on maintenance effectiveness	Literature Review, Questionnaire Survey
v) Can maintenance effectiveness be modeled? If so, how can it be done?	<u>Modeling of Maintenance Effectiveness in the Short-term</u> Response Variable: Deterioration Rate Reduction (DRR) Explanatory Variables: Pavement Type, Age group, Region, etc.

1.5.3 Long-term Cost-effectiveness Evaluation

This part of the study is devoted to the development of performance models, performance jump models and cost models using the data collected. Suitable models were selected after several trials using a variety of analytical techniques and mathematical forms, and data development was guided by the experience of past researchers who have, over the years, built a variety of such models. Cost models were also developed for each maintenance treatment. Maintenance strategies were formulated and evaluated on the basis of benefits and costs. Costs associated with each strategy included the additional costs incurred by road users as a result of work zones associated with the application of maintenance treatments that comprise the strategy. The benefits of each strategy (typically viewed as a reduction in overall road user cost) were surrogated by the area under the performance curve associated with the strategy.

1.5.5 Trade-off Analyses

The last part of the study focuses on investigating whether any trade-off relationships exist between maintenance and the frequency of capital investment rehabilitation, and also between preventive maintenance and subsequent corrective maintenance, and attempts to model such relationships. The

rehabilitation/maintenance trade-off modeling involved estimating the interval of rehabilitation (in years), given the annual maintenance expenditure and other pavement performance factors. In the preventive/corrective maintenance trade-off analyses, the level of corrective maintenance for a 3-year period was estimated, given the level of preventive maintenance received by the pavement section in the preceding 3-year period.

Appropriate functional forms were selected for the trade-off models in order to facilitate engineering interpretation of the resulting functions and to provide reasonable fit to the given data. This part of the study also includes determination of the marginal effects of the trade-off modeling variables.

1.6 Organization of this Report

Chapter 1 of this report provides a brief overview of current policies and practices of pavement rehabilitation and maintenance in the state of Indiana. This section also highlights the objectives and scope of the present study, and provides an overview of the approach used in realizing the study objectives. A presentation of the temporal and spatial trends in pavement type, loading condition, unit repair costs and pavement repair expenditure is provided in Chapter 2. This chapter was included to provide a bird's eye view of the effect of variables that typically influence pavement maintenance effectiveness. The literature review in Chapter 3 presents and discusses past findings and experience of previous researchers in the areas of short-term maintenance effectiveness, long-term effectiveness, and trade-off analyses. Chapter 3 is complemented by Chapter 4, which provides results of a questionnaire survey carried out among various districts and sub-districts of INDOT, and provides a review of current state-of-practice of pavement maintenance for the state highway network, as well as the perceptions of pavement managers and engineers on the effectiveness of their pavement maintenance practices. Chapter 5 presents a framework for the analyses, discusses the various methodologies used, and provides a theoretical basis to concepts used in the present study. In Chapter 6, details of data collection and development of INDIPAVE 2000 database are briefly described. In Chapters 7 through 9, the results of the study are presented, while Chapter 10 discusses case studies of selected pavement sections whose longevity has been greatly enhanced by sustained maintenance. Chapter 11 concludes the study by presenting a summary of findings, a discussion of challenges faced during the study, implementation issues, and areas for future investigation.

CHAPTER 2: SPATIAL AND TEMPORAL TRENDS IN PAVEMENT CHARACTERISTICS

Between 1990 and 2000, there were over 11,000 miles of roads on Indiana's state highway network, constituting over 12% of the entire road network in the state. The maintenance and rehabilitation of these roads are the responsibility of the Indiana Department of Transportation. This chapter presents general trends in pavement types, climate, loading, expenditure, and other attributes on the state highway network. Such trends, which have been examined from a spatial and temporal perspective, provide a background to pavement performance modeling, maintenance effectiveness evaluation, and trade-off relationships that are investigated in subsequent chapters. A discussion of the trends of each attribute is presented below.

2.1 Distribution of Pavement Types

For the purposes of this study, pavement types are categorized based on the material comprising the surface layer only, and excludes the base and sub-base material. On this basis, there are basically 3 types of pavement on Indiana's state highways: rigid (full-depth PCC), full-depth asphaltic concrete, and overlay pavements (AC-over-PCC). "Full-depth" in this context refers only to the surface layer and not the base or sub-base material. [Figure 2-1](#) shows the various sub-types of each type of pavement, while [Figure 2-2](#) shows the average distribution of the pavement types. For purposes of the present study, overlay pavements are considered solely as AC-over-PCC pavements. Rubblized pavements are often considered AC-over-PCC overlays, even though it can be argued that rubblization yields a porous base material that cannot really be referred to as an underlying PCC layer, and therefore such pavements could be considered as having a full-depth asphaltic concrete surface layer. Other pavement types such as PCC-over-AC and PCC-on-PCC constitute a very small fraction of the network and were consequently excluded from the analyses.

The percentage of asphaltic concrete pavements increases from northern to southern Indiana, while the total mileage of AC-over-PCC overlay pavements decreases from northern to southern Indiana. The total mileage and percentage of PCC pavements are higher in central Indiana compared to the southern and northern regions of the state.

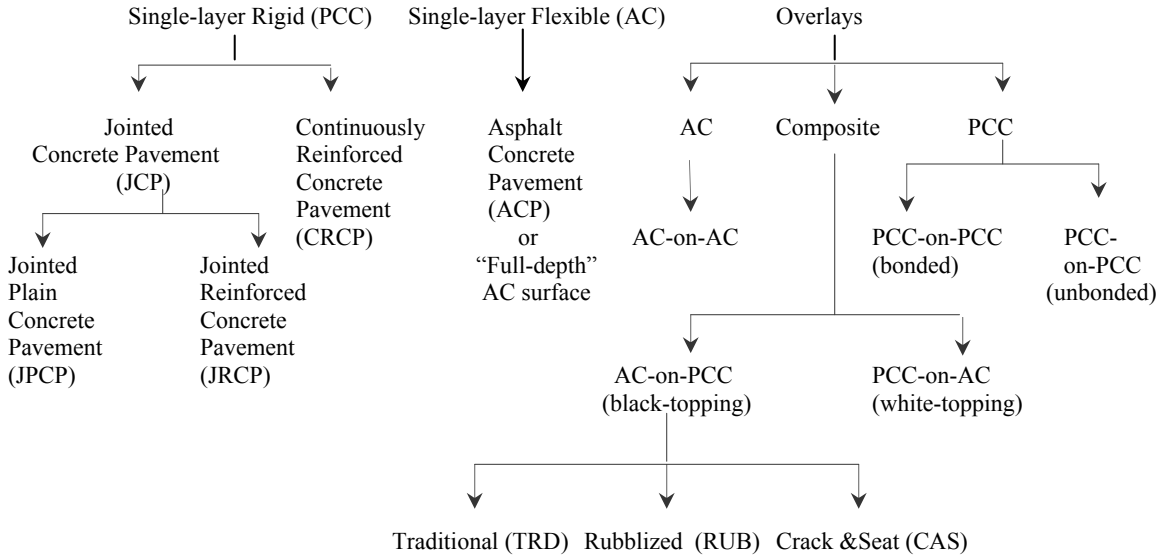


Figure 2-1: Pavement Types on the Indiana State Highway Network

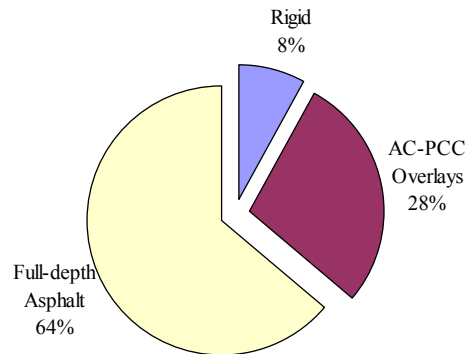


Figure 2-2: Distribution of Major Pavement Types on Indiana State Highway Network (Source: Highway Statistics, 1998, Table HM-51)

2.2 Temporal Trends in Pavement Type Distribution

Pavement maintenance practices (and consequently, resources expended for maintenance) vary by pavement type. It is therefore useful to study not only the spatial distribution of pavement surface types in the state at a given point in time, but also to investigate the change of such distribution over time. Figure 2-3 shows an increasing trend towards the use of asphalt overlays over existing concrete pavements (a practice typically termed “blacktopping”): for most part of the 1990-1999 period, approximately 200 miles of existing concrete roads received asphaltic surface overlay annually. There have been a few attempts at using PCC overlays on existing asphalt-surfaced pavements (i.e., “whitetopping”) as well as on existing concrete pavements (bonded or unbonded overlays), however the use of this relatively new technology is still limited. The mileage of rigid pavements increased slightly after 1998 as a result of the reconstruction of many sections on Interstate 465 using Portland Cement Concrete in 1999.

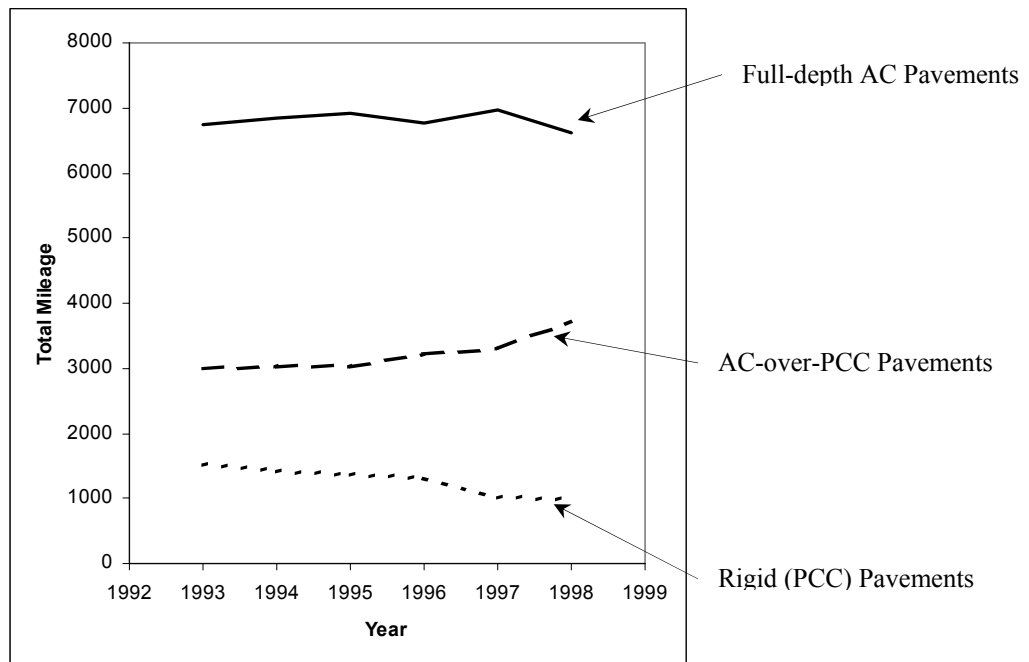


Figure 2-3: Temporal Trends in Pavement Surface Type Distribution

2.3 Pavement Loading Distributions

Ever increasing levels of pavement loading is a source of concern to highway pavement managers, as average daily loads on highway pavements continue to increase year after year. This section determines the distribution of pavement loading levels by various attributes such as highway route type, pavement type, and regional location.

After 1970, and particularly in 1994 (in wake of truck deregulation) there has been a general increase in the rate of growth in average daily loads [FHWA, 1999]. These trends are particularly seen in Indiana, where state highway pavements experience one of the highest levels of truck traffic in the country. Deregulation resulted in increased truck sizes and weight as trucking companies moved to take advantage of the economies of scale without necessarily changing their axle configurations in a manner commensurate with the increase in loading.

Levels of pavement loading (Figures 6 and 7) were calculated using the gross vehicle weights associated with each 1-mile segment as provided in INDIPAVE 2000 (the database developed for the present study). For each functional class or pavement type, the total load for all segments in that category was divided by the total length of road segments in that category.

2.3.1 Distribution of Pavement Loading by Route Type

Figure 2-4 shows the distribution of average loadings, over a three-year period (1996-1998) experienced by each route type, in terms of gross vehicle weight (GVW). In comparing the traffic loading across route type, it is deemed more useful to use gross vehicle weight rather than ESALs. This is because route types consist of different proportions of pavement types, and ESAL factors differ from one pavement type to another, even for a given traffic load. However, for comparison of traffic loads for pavement in a given type, or for modeling purposes, the use of ESALs may be deemed appropriate.

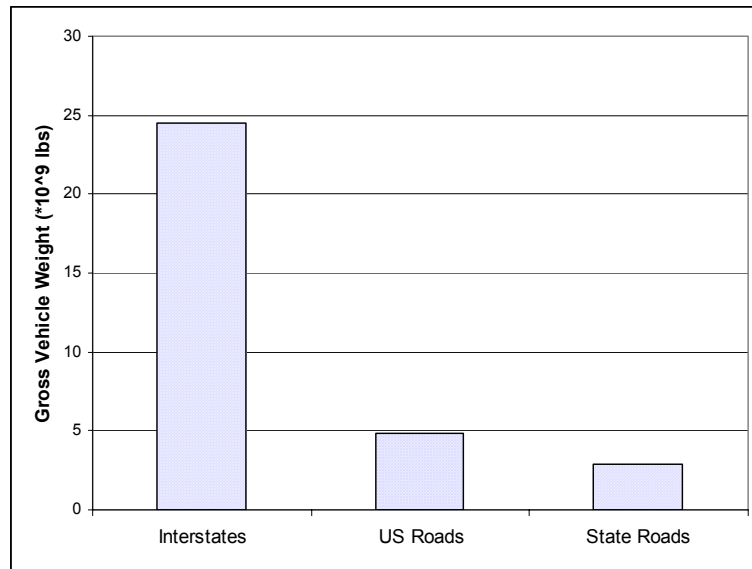


Figure 2-4: Distribution of Average Annual Loading by Route Type

Quite expectedly, Interstates are, by far, associated with the highest levels of pavement loading, as seen in Figure 2-4. Operators of larger vehicle classes (FHWA classes 4 and above) prefer such highways, which are characterized by low levels of accessibility, high levels of mobility, and superior geometric standards. State roads generally have the lowest levels of traffic loading, while the case for US roads is in between these two extremes, but nearer to that of the State roads.

2.3.2 Pavement Loading on Most Heavily-loaded Highways

The levels of average annual pavement loading (1996-1998) were determined for all roads on the state highway network. The results are shown in Figure 2-5. Interstate 80/90, which connects the eastern part of the United States to the mid-western cities of Chicago, Minneapolis and beyond, has the highest average level of traffic loading. The next most heavily loaded roadway is Interstate 465, which circumscribes the city of Indianapolis. Of non-Interstate highways, State Road 912 is most the heavily loaded. Load values are averaged over the entire stretch of a given highway. Local load concentrations are not captured by such averaging methods.

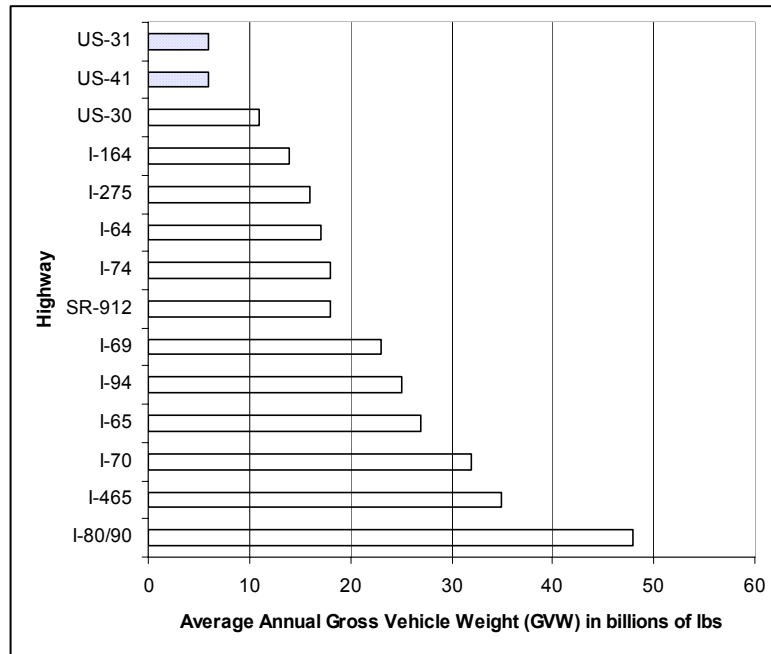
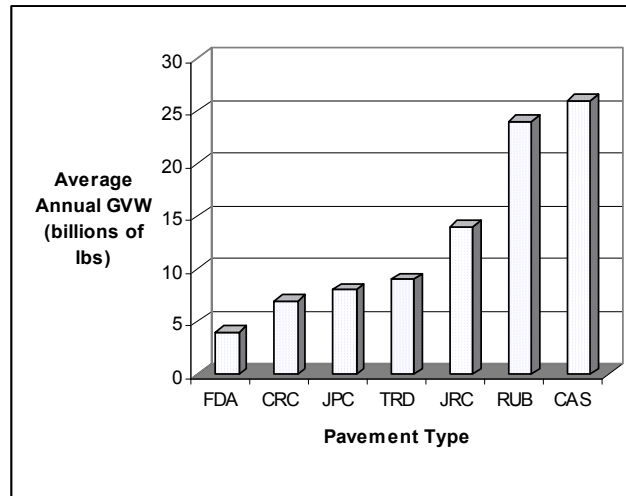


Figure 2-5: Average Annual Pavement Loading for the Most Heavily Loaded Highways

2.3.3 Pavement Loading Distribution by Pavement Type

Descriptive statistics of average annual pavement loading distribution, over a three-year period (1996-1998), by pavement type and region, shown as Figure 2-6, indicate that crack-and-seat and rubblized overlay pavements, on the average, experience the highest pavement traffic loadings, while full-depth AC pavements experience the lowest loading. It is worth noting that there are very few rubblized and crack-and-seat pavements in the state. Of the more common pavement types, jointed reinforced concrete pavements are generally the most heavily loaded, followed by traditional overlay pavements. This trend of loading is only an average picture, and it is expected that data analyses on a section-by-section basis would yield more insight into the magnitude of pavement loading and its effects on pavement performance.



Legend

FDA- Full-Depth Asphaltic Concrete pavement
 CRC- Continuously-Reinforced Concrete Pavement
 JPC- Jointed Plain Concrete Pavement
 TRD- Traditional AC-over-PCC Overlay Pavement
 JRC- Jointed Reinforced Concrete Pavement
 RUB- Rubblized PCC with AC Overlay
 CAS- Crack and Seated PCC with PCC Overlay

Figure 2-6: Average Annual Pavement Loading by Pavement Type

2.4 Trends in Climatic Features

The influence of climate on pavement performance has been well documented in literature. The rheological properties of bitumen render the stability of asphaltic concrete pavements susceptible to accelerated failure at temperature extremes. Under high temperatures, bitumen becomes increasingly viscous and undergoes plastic deformation. For this reason, rutting failures are common in the warmer regions of the United States, all else being equal. In Indiana, rutting is more common in the southern part of Indiana than it is in the north. On the other extreme, low temperatures cause asphaltic cement to become brittle under low temperatures and vulnerable to cracking under traffic loading. Therefore, cracking is less common on AC pavements in southern Indiana.

Portland Cement Concrete pavements suffer relatively less damage due to temperature extremes, but are more vulnerable to temperature changes. Expansion and contraction forces in concrete due to temperature variations cause stresses in concrete pavement that ultimately lead to failure.

For both flexible and rigid pavements, the underlying subgrade is vulnerable to temperature variations. Thawing in the spring season causes subgrades to lose strength, and this effect is especially pronounced when ice lenses are present in the subgrade [Yoder and Witczak, 1975]. For this reason, pavement deterioration is most severe in the Spring season [Allen et al., 1991]. Another aspect of the weather that accelerates pavement deterioration is precipitation: subgrade heave, resulting from expansion of clayey soils in response to increase in moisture content (arising, in part, from precipitation recharge), is a major cause of longitudinal cracking and other crack types on pavement surfaces. Also, all else being equal, pavements in regions with higher precipitation suffer more deterioration because more water enters the underlying pavement layers through any surface cracks and unsealed/deteriorated joints. This can lead to pothole formation, deterioration of existing patches, ejection of a water-fines slurry (pumping) and subsequent void formation under the pavement, and loss of base or subgrade support.

The share of non-load (typically climatic) factors in pavement deterioration and subsequent pavement repair costs typically ranges from 20%-60% compared to load factors, depending on the type of pavement [Fwa and Sinha (2), 1987; Li and Sinha, 2000]. Climatic variations within the state of Indiana are dictated by three factors: topographic features, temperature, and precipitation. These are discussed below.

Topographic Features:

The three principal land regions in Indiana are: the Great Lakes Plain in the north, the Till Plains in the center, and the Interior Low Plateau in the south. Elevations range from approximately 300 feet above sea level at the southwest corner of the state to 1200 feet in Steuben County in the northeast [Fenelon et al., 1994]. Unlike most of other areas in the state, South central Indiana did not suffer glaciation in the ice age, and therefore it has the most rugged terrain. The Kankakee valley in the northwest slopes gently towards the west and drains a large area that used to be a marsh. Northeastern Indiana is characterized by the presence of many small lakes and moraines.

Temperature:

By virtue of its location in the middle latitude in the interior of the continent, Indiana is described as having an “invigorating climate with warm summers and cool winters” [NOAA, 1995]. Against a background of daily and seasonal temperature fluctuations, temperature variations occur every few days as masses of polar air move southward or as tropical air move from south to north. The frequency and magnitude of these fluctuations are more pronounced in winter than they are in summer, and are obviously responsible for freeze-thaw cycles in the winter season. Also, the dominance of either of the polar air masses dictates whether the state will experience unusually cold or mild winters, or unusually hot or mild summers. In the southern part of the state where the terrain is hilly, temperature varies in short distances, as the valleys have lower temperatures than the slopes and tops of the surrounding hills.

Precipitation:

The interaction of southbound polar air and northbound tropical air masses is responsible for most of Indiana’s precipitation. The action between the two air masses of contrasting temperature, humidity and density results in the development of low pressure centers that generally move eastward, often passing over the state and causing abundant rainfall. Average annual rainfall ranges from 37 inches in northern Indiana to 45 inches in the south [NCDC, 1976; NCDC, 2000]. The effect of the Great Lakes on the climate of northern Indiana is most pronounced on the immediate shore of the lakes and diminishes rapidly with distance. Due to the passing of cold air over warmer lake water, the northern counties of Lake, Laporte, and Porter experience the highest levels of winter precipitation. Average annual snowfall ranges from 60 inches in northern Indiana to 10 inches in the south [NCDC, 2000]. Snowfall varies significantly from year to year depending on both temperature and frequency of winter storms. [Figure 2-7](#) presents the variation of selected pavement-related climatic features across the three regions of the state.

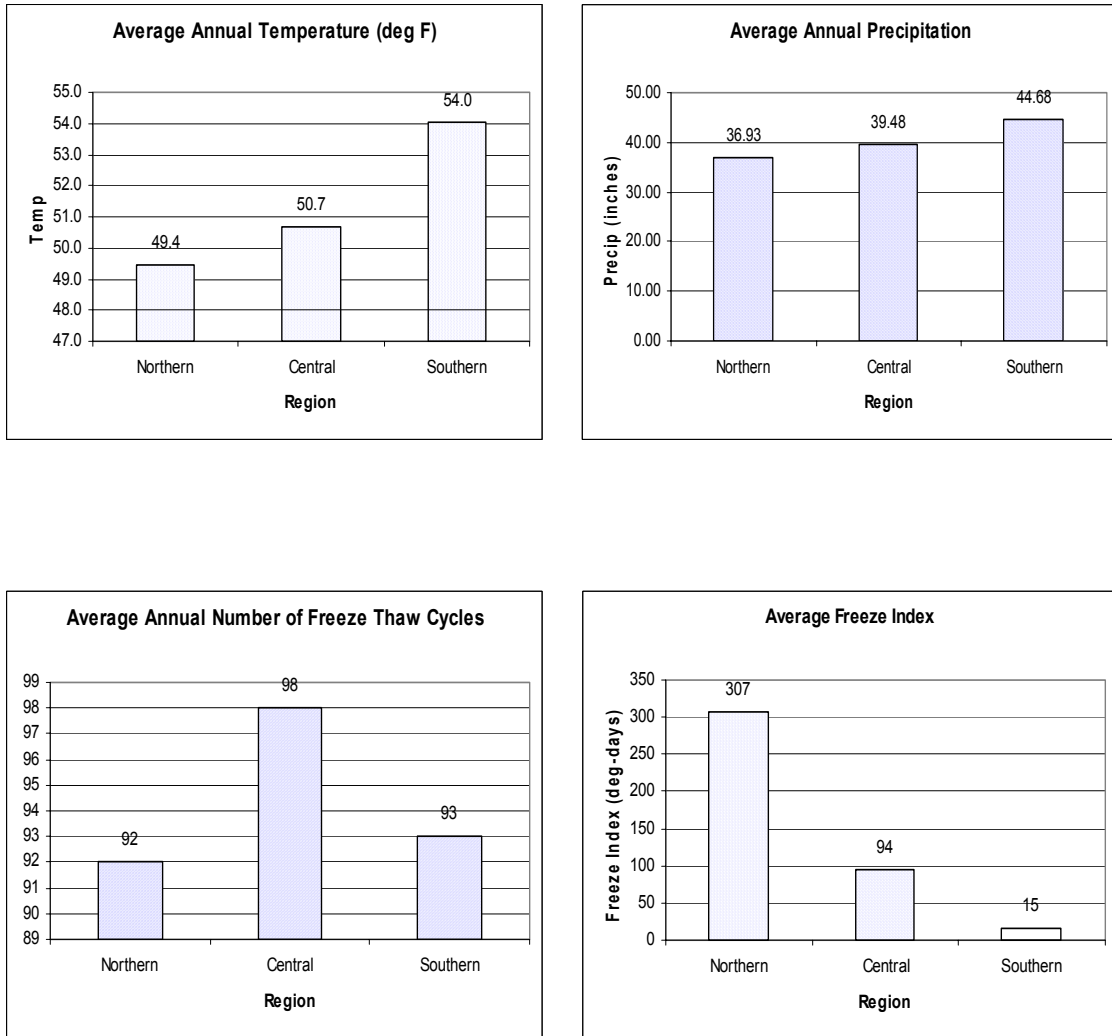


Figure 2-7: Spatial Variation of Pavement-related Climatic Features

2.5 Distribution of Surficial Soils

The state of Indiana is divided into three physiographic zones whose features dictate, to a large extent, the type of pavement subgrade soils found in those areas. These are the Northern Lake and Moraine region, the Central Drift Plain, and the Southern Bedrock Landforms region. Glacial depositional features in northern Indiana include moraines, outwash plains, kames, and lake plains. Surface geology in these areas consist of a diverse mix of sediments with highly variable hydrogeologic properties and lithographic discontinuities [Fenelon et al., 1994] and typical pavement problems include poor subgrade support. Postglacial landforms in this region include a multiplicity of lakes found in northeastern Indiana, and the frequent pockets of muck and peat bogs that arise from the damming of drainage areas. Even though the soils of the glacial drift are granular in nature, the general shallowness of the drift results in problems of drainage, particularly in areas where interbedded silts and clays are found with the drift, and are therefore generally associated with problems of weak subgrades. The northern Lake and Moraine region consists of five subregions that are characterized by lacustrine plains, morianal areas, and outwashes. A sandy lake plain overlying a basal till dominates most parts of Lake, Laporte, and Porter counties. The counties of Elkhart, Steuben, and Noble, are dominated by a layer of unconsolidated drift soils that was deposited during the advances and retreats of the Wisconsinian and older glaciations.

The Kankakee Outwash in north central Indiana is typically flat and poorly drained. Sand, deposited as outwash by glacial meltwaters, lies at or near the surface in this area, and prevailing westerly winds have “re-arranged the sand into dunes in White and Pulaski counties” [Fenelon et al., 1994]. Twenty-seven counties in central Indiana lie in the White River Basin, which is dominated by unconsolidated glacial deposits consisting of clay-rich loamy tills interbedded with stratified sands and gravels. This area includes Tipton and Delaware counties in the south, and Knox, and Pike counties in the north.

Unlike Northern and Central Indiana, the southern part of the state is dominated by soils of a residual nature, as they were derived from parent bedrocks of granites, schists and gneisses (Figure 10). These soils tend to be highly micaceous and generally have a sandy texture, with relatively deep soil mantle on top of the parent rock [Yoder and Witczak, 1975]. Surficial soils in the south-western counties of Sullivan, Posey, Vigo and Vanderburg are characterized by a thin cover of till, loess, and silt, which are

difficult to compact and are typically problematic subgrades for pavements (Figure 2-8). Notwithstanding these generalizations, there is marked local variability in surficial soils, and pavement subgrades in the state often vary considerably over short distances.

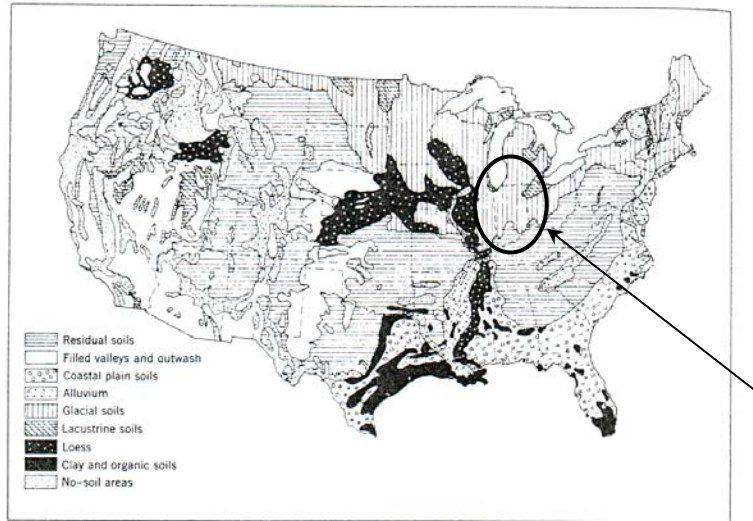


Figure 2-8: Surficial Geology in the United States [Yoder and Witczak, 1975]

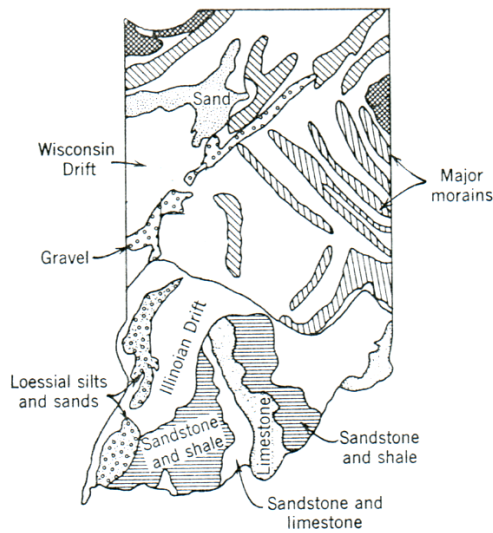


Figure 2-9: Surficial Soil Distribution in the State of Indiana [Yoder and Witczak, 1975]

2.6 Pavement Repair Expenditure Trends

2.6.1 Average Annual Distribution of Highway Pavement Repair Expenditure

Capital expenditure on highway pavements, for the purpose of the present study, consists of reconstruction of flexible or rigid pavements, and rehabilitation (resurfacing) of flexible pavements, but excludes other capital works such as new construction, relocation, and widening, which generally have little relationship with maintenance of existing pavements. Restoration of rigid pavements, such as deep patching, regardless of the scale, was considered as a maintenance activity for the present study. Between 1996 and 1998, the average annual expenditure for capital works on Indiana's highway pavements was approximately \$500 million, [FHWA, 1996-1998]. Expenditure for pavement maintenance has consistently lagged behind that for reconstruction and rehabilitation (Figure 2-10). It is believed that effective maintenance practices, particularly those of a preventive nature, would reduce the frequency of reconstruction and rehabilitation, and thus reducing the need for capital expenditure over the life cycle. Such practices would include application of maintenance treatments in a manner that maximizes cost-effectiveness.

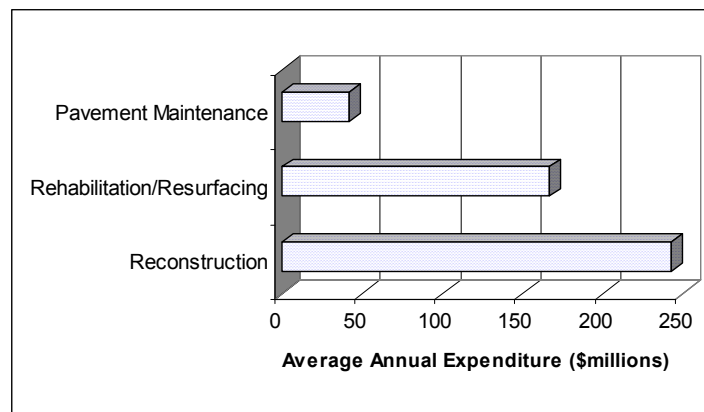


Figure 2-10: Average Annual Expenditure for Maintenance and Capital Works, for the Indiana State Highway Network

2.6.2 Temporal Trends in Pavement Capital Expenditure

Expenditure levels for capital works (reconstruction/rehabilitation only) have been somewhat erratic over the past decade. Total capital expenditure fell from a high of \$410 million in 1992 to about one-half of that amount in the following year. It rose to \$361 million in 1994 but fell in 1995 and 1996, reaching a decade low of \$218 million in 1996. 1997 and 1998 saw marked increases in total capital expenditure, reaching a level even higher than that of 1992. Figure 2-11 shows the temporal trends of unit pavement capital expenditure (per lane-mile) by road functional class. For most road functional classes, the pattern of capital expenditure was similar to the general trend described above. The only exception was Urban Interstates, for which capital expenditure reached a nadir in 1994 and 1995, and rose steadily thereafter. It appears that the level of funding (and consequently, the expenditure) for road infrastructure investments each year does not follow a quite consistent pattern. A study carried out on highways in Australia argued that the level of capital investment is the single most influential factor in network level improvement in pavement condition, and that the role of maintenance in this regard is relatively insignificant [Martin, 2000].

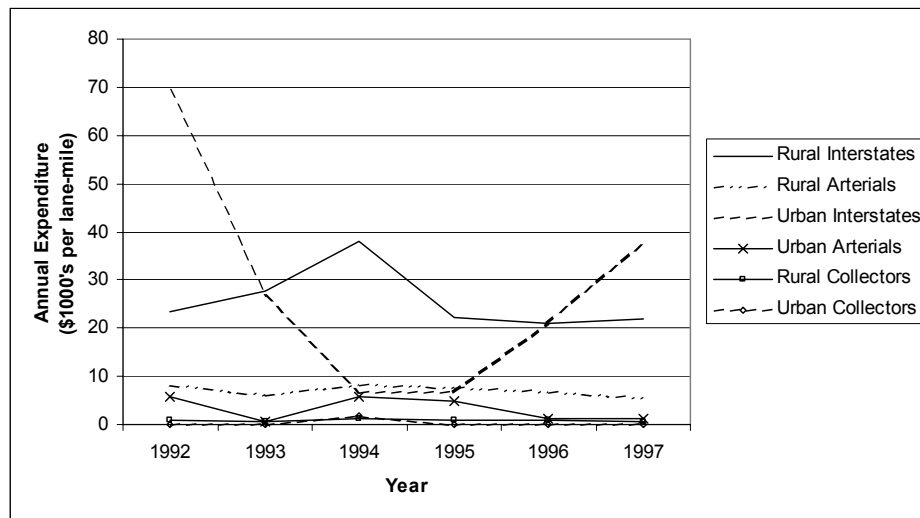


Figure 2-11: Trends in Unit Capital Expenditure by Functional Class (per lane-mile), for the Indiana State Highway Network
(Source of Data: Table SF-12A, Highway Statistics 1992-1999)

2.6.3 Temporal Trends in Pavement Maintenance Expenditure

Most of the past decade has seen fairly stable investment levels for pavement maintenance (approximately \$30 million per year). However, in 1998, the average annual expenditure for pavement maintenance increased by about 30%, for most functional classes.

2.6.4 Distribution of Pavement Maintenance Expenditure by Treatment Category

Figure 2-12 shows the distribution of average annual pavement maintenance expenditure by treatment category from 1996 to 1998. Data was obtained from the annual summaries of maintenance expenditure from the Operations Support Division [INDOT, 1994-1997] and the Program Development Division.

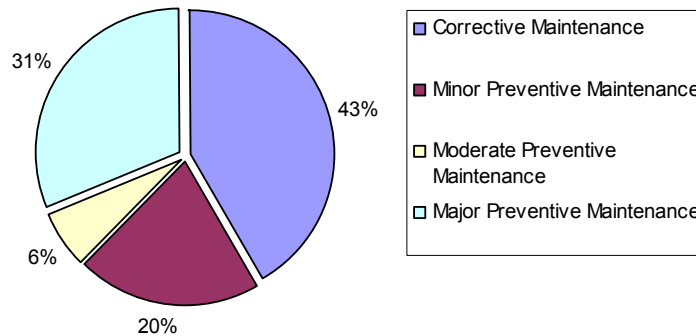


Figure 2-12: Average Annual Pavement Maintenance Expenditure by Treatment Category for the Indiana State Highway Network, 1996-1999

As seen from Figure 2-12, corrective maintenance commands the largest share of pavement maintenance expenditure, followed by major, minor and moderate preventive maintenance. It is expected that an increase in preventive maintenance will decrease the expenditure for corrective maintenance and subsequently, total maintenance expenditure.

2.6.5 Distribution of Pavement Maintenance Expenditure by Treatment Type

Figure 2-13 shows the distribution of average annual pavement maintenance expenditure by treatment type, expressed in constant (1995) dollars. It is observed that thin overlay and shallow patching, (which are major preventive and corrective maintenance activities, respectively) are associated with highest levels of pavement expenditure. Crack sealing, the next dominant activity, is a minor preventive maintenance treatment.

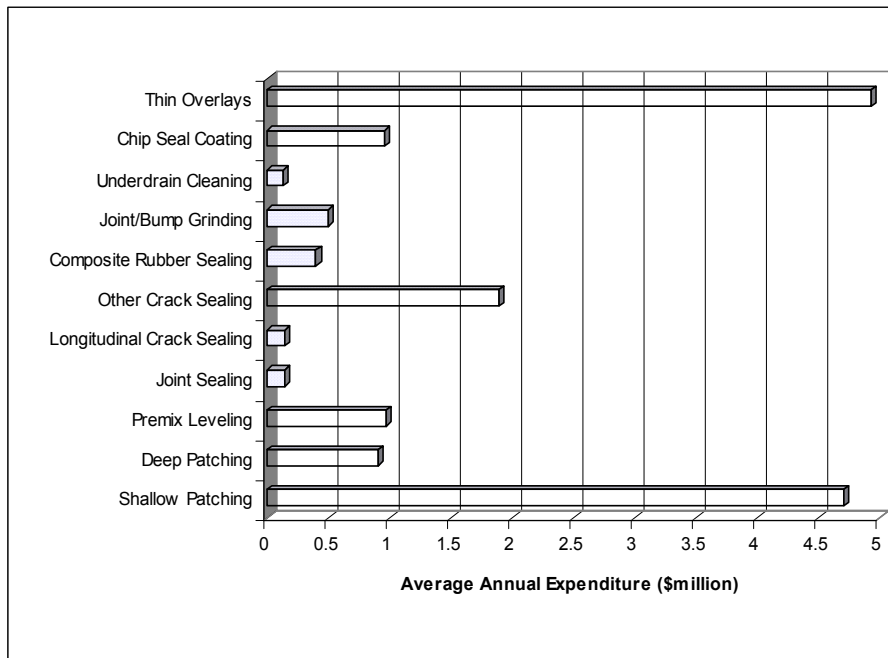


Figure 2-13: Average Annual Pavement Maintenance Expenditure by Treatment Type, for the Indiana State Highway Network, 1996-1999

2.6.6 Temporal Trends in Maintenance Expenditure by Treatment Category

Figure 2-14 shows the temporal trends in pavement maintenance expenditure by treatment category. From the figure, it appears that preventive maintenance has an overall effect on the level of subsequent corrective maintenance. From 1994 to 1995, there was an increase in statewide levels of minor preventive maintenance, and this was accompanied by a decrease in corrective maintenance. Then between 1995 and 1997, there was a general decrease in both minor and moderate preventive maintenance, and this was accompanied by an increase in the levels of corrective maintenance.

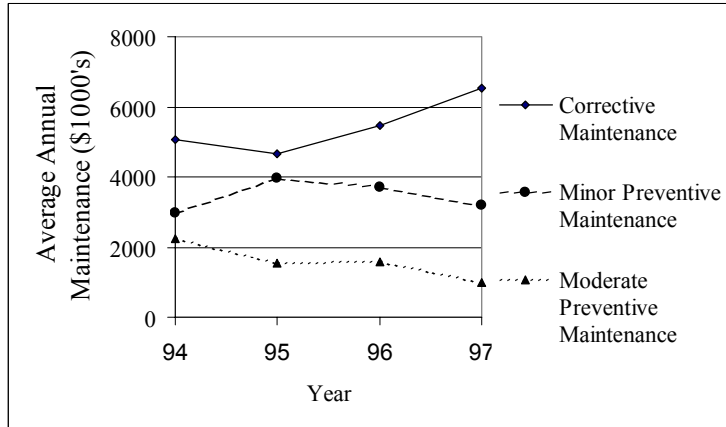


Figure 2-14: Trends in Maintenance Expenditure, by Treatment Category, for the Indiana State Highway Network
 (Source: 1993-1997 Annual Maintenance Reports, Operations Support Division, INDOT)

Any time-lag effect of the trade-off between preventive and corrective maintenance is likely to be most readily manifest on at least a half-year basis (as in-house maintenance is carried out twice in every fiscal year: from July to December, and then from January to June), and at most on a three year basis (as many sub-districts and districts operate on a recurring maintenance cycle for pavements). Figure 2-15 shows a sketch of the distribution of average pavement maintenance expenditure by region and maintenance category.

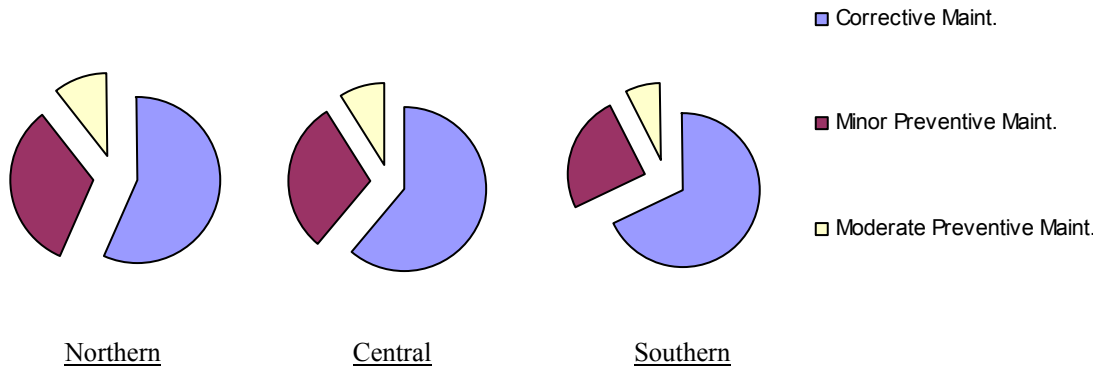


Figure 2-15: Distribution of Maintenance Expenditure by Region and Treatment Category, Indiana State Highway Network, 1996-1999

From north to south, there is increased use of corrective maintenance relative to preventive maintenance. In other words, the fraction of each maintenance dollar spent on preventive maintenance is higher in the north than it is in the south. Northern Indiana has a harsher climate than the south, from the perspective of pavement deterioration. Therefore, the fact that a higher fraction of each maintenance dollar is spent on preventive maintenance at such areas suggests that preventive maintenance is probably more needed in areas of harsh weather than it is in areas of relatively mild weather.

2.7 Trends in Pavement Condition

2.7.1 Pavement Condition by Region

Figure 2-16 presents the distribution of pavement condition and region in 1999. The figure suggests that average pavement conditions are generally better in the southern region, an observation for which reasons are offered in the subsequent paragraph.

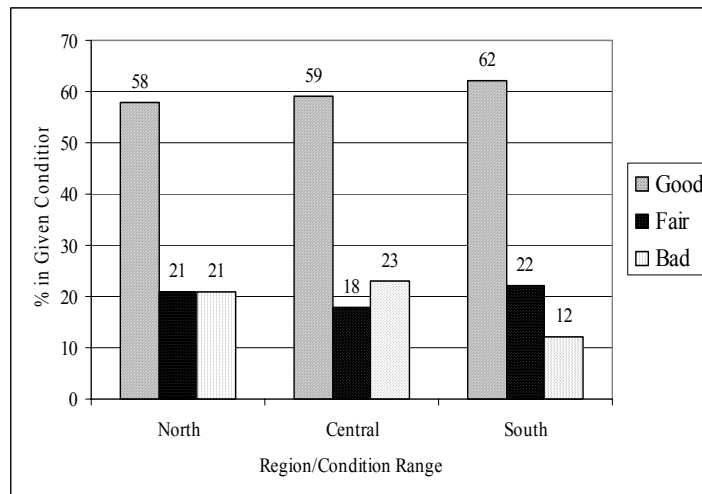


Figure 2-16: Distribution of Pavement Condition by Region, 1998
(Source: 1999 Pavement Surface Report, Program Development Division, INDOT)

Generally speaking, the relatively better condition of pavements in the southern region of the state could be attributed to i) lighter pavement loadings, (ii) hilly terrain, thus quicker surface drainage, (iii) better subgrades (most pavements are built in cut sections well below depth of weathered surface material or on engineered material at fill sections, due to hilly nature of terrain), and (iv) generally better weather (higher temperatures, little freezing). Admittedly, the south has higher non-winter precipitation and slightly higher number of freeze-thaw cycles, but these adverse features are obviously offset by other redeeming features in the south such as those discussed above. The average pavement conditions in northern Indiana are slightly better than those in the central part probably due to higher average levels of pavement loading in the central region compared to the north. From west to east, better average pavement condition is observed, except for northern region. Unlike other regions, eastern pavements in the northern region are worse than their western counterparts. A possible reason is the very poor quality of subgrades in the northeast is the nature of geologic and hydrogeologic formations (lakes, moraines, peat pockets, etc., due to glacial activities and depositions during the ice age). Differences in institutional practices from one highway district to another also account for differences in pavement condition from north to south and from east to west.

2.7.2 Pavement Condition by Highway Route Type

Average roughness values were computed for each highway route type in terms of Present Serviceability Index (PSI). It was found that Interstate roads, with a three-year average annual (1996-1998) with a PSI value of 3.16 units, have the best pavement condition (Figure 2-17). State Roads had the next highest level, followed by US Roads. This is expected, as Interstate pavements are generally of higher design and construction quality and therefore able to sustain good levels of pavement condition even though they suffer, on the average, 4-5 times as much pavement loading as non-Interstates (US and State Roads).

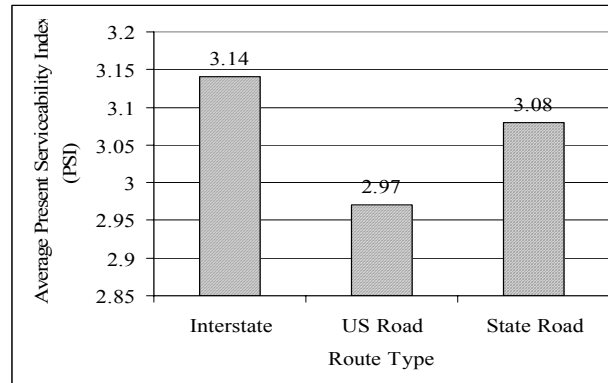


Figure 2-17: Average Annual Pavement Condition on Highway Route Type
(Source: 1999 Pavement Surface Report, Program Development Division, INDOT)

2.7.3 Temporal Trends in Pavement Condition

Figure 2-18, which is a graph of the median PSI values for Indiana, shows four stages through which the condition of the state's pavements have undergone between the period 1992 and 1998:

- ❑ Period 1 (1992-1993) indicates the tail end of a fairly stable trend, where pavement condition generally improved for all road classes.
- ❑ Period 2 (1993-1994) shows a period when Indiana's pavements experienced a precipitous drop in pavement condition. The end of this period (1994) saw most pavements reach their poorest levels of pavement condition within the entire period under consideration.
- ❑ In period 3 (1994-1997), the situation improved for all pavements. By 1995, the deterioration trend had been reversed, except for Rural non-Interstates (whose worst condition was in 1995 and started improving thereafter), and for Urban Interstates (which had peak poor levels for the most part of 1994-1997 period) but started showing an improvement at the end of this period. By the end of the 3rd period, all road classes had experienced modest improvements in pavement condition.

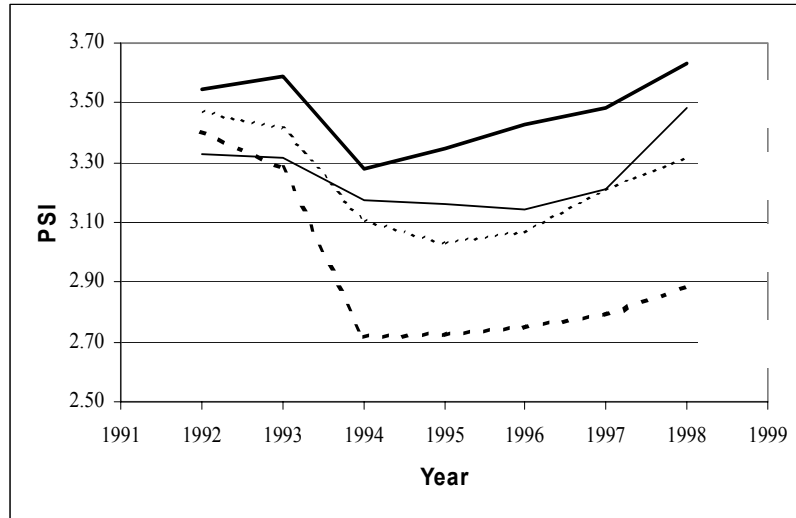


Figure 2-18: Trends in Average Pavement Condition by Functional Class in Indiana
(Source of Data: IRI data in Table SF-13, Highway Statistics 1992-1999)

- Period 4 (1997-1998) indicates a general continuation of the improvement trend started at the end of 1994. The average condition of pavements in some functional classes (Rural non-Interstates and Rural Interstates) was brought to levels near pre-1993 conditions, while the condition of Urban Interstates improved sharply and by the end of Period 4, surpassing even pre-1993 levels. The average condition of Urban and Rural non-Interstates also saw an upsurge in Period 4, albeit to levels below those prior to 1993.

2.8 Chapter Summary

The analyses of the spatial and temporal variations in pavement-related attributes showed that there is significant variation in such attributes over time and space to warrant the consideration of such attributes of pavement performance and maintenance effectiveness studies for the state of Indiana. A summary of findings is presented below.

- *Pavement Types*: Pavements in the state can be divided into three major surface types: rigid, overlay and full-depth asphalt, constituting approximately 10%, 30% and 60% of the state

highway network, respectively. Also, there is significant variation of the relative proportions of pavement type from north to south. Furthermore, the last decade has seen an increasing proportion of AC-over-PCC overlay pavements.

- *Pavement Loading:* Pavement loading is highest for Interstate pavements, and lowest for state roads, and is also highest for pavements in the central region, and lowest for those in the south. On the average, rubblized and crack and seat pavements are associated with the highest levels of pavement loadings (followed by jointed concrete and traditional overlays), while full-depth asphaltic concrete pavements experience the least loads.
- *Climate:* Climatic features, which are dictated by changes in topography, precipitation, and temperature from northern to southern Indiana, vary considerably across the state. As one goes from north to south, one encounters conditions that are generally more favorable to pavement materials. Total levels of precipitation (rainfall and snowfall combined) and the number of freeze-thaw transitions apparently do not vary significantly across the state, while freeze indices are very different, ranging from zero at the southern tip to over 600 degree-days at the northern tip of the state.
- *Subgrade:* Subgrade soils, the nature of which depend on surficial geology, differ from north to south, and in some cases from east to west. The northern part of the state is generally characterized by shallow soils of glacial origins, while the south has residual soils derived from parent bedrock.
- *Pavement Maintenance Efforts:* Annual pavement maintenance expenditure lags far behind that for capital works (rehabilitation (resurfacing) and reconstruction). Levels of capital expenditure for arterials and collectors remained fairly constant over the past decade, while Interstates saw a slight general increase especially after 1994. There appears to be no clear indication of changes in capital expenditure in response to any changes in maintenance in a previous period. Of all maintenance treatment categories, corrective maintenance appears to have the highest levels of expenditure (40% of total expenditure), followed by major preventive maintenance (30%). Minor and moderate preventive maintenance constitute approximately 20% and 10%, respectively. There appears to be a trade-off between preventive maintenance and corrective maintenance for all

regions, and particularly for the northern region. Also, from north to south, a lesser share of the maintenance dollar is expended on preventive maintenance compared to corrective maintenance, which is probably suggestive of the greater need for preventive maintenance in areas of harsher weather.

- *Pavement Condition:* Pavement condition levels in the south are slightly in better condition than their northern or central counterparts. The plausible explanation for this is the more favorable environmental (climate, terrain, and subgrade) features and lighter traffic loadings in that part of the state. This probably explains why southern pavements are generally associated with lower levels of pavement maintenance expenditure, as found in several past studies in Indiana. Pavement condition on Interstate pavements is better than that of non-Interstate pavements, in spite of their higher traffic loadings, probably due to their superior design and construction features. Temporal trends in condition of all state highways indicate increasing pavement condition up to 1992, a sharp decrease in condition from 1992-1994, and a gradual increase in condition thereafter.

The spatial and temporal trends in pavement attributes that were investigated in this chapter provide a useful insight into the variation of such attributes and their impact on pavement performance modeling, evaluation of pavement maintenance effectiveness in the short and long-term, and pavement maintenance-rehabilitation trade-off analyses.

CHAPTER 3: LITERATURE REVIEW

3.1 Introduction

Any maintenance effectiveness study should be preceded by review of available literature related to pavement performance, failure and repair, state-of-practice regarding pavement maintenance and rehabilitation (in the State of Indiana in particular, and the United States in general) and various methods of evaluating cost-effectiveness of maintenance activities. In the present study, a systematic literature review was undertaken with the following goals:

- To obtain current understanding about the causes, mechanisms, and the short- and long-term effects of various modes of distress in rigid and flexible pavements and their relation to standard pavement preventive and corrective maintenance treatments.
- To identify and evaluate state-of-the-practice preventive and corrective maintenance treatments and timings of preventive maintenance used by various state highway agencies in the United States. This was done to acquire further knowledge about the efficacy of various maintenance activities, to acquire expert opinions, to validate the results of the present study and to explain any possible inconsistencies.
- To synthesize and assess the methods used (as well as results) of various research efforts undertaken by institutions and agencies in assessing the cost-effectiveness of various treatments or strategies for pavement maintenance.

This task involved the collection and review of information about maintenance practices at INDOT and other state and provincial DOTs, local highway agencies, research institutions, and other public and private organizations. This was accomplished through the following activities:

- Review of published material on the subject.
- Direct communication with PMS Engineers and other individuals at INDOT and other state transportation agencies.

Table 3-1 presents details on the information obtained from various sources identified for the literature review, while Figure 3-1 provides an insight into the relevance of the various aspects of the literature review and questionnaire survey by showing the relationship between various aspects of these tasks and study objectives.

Table 3-1: Information Obtained from Identified Sources

Source(s)	Information Obtained
Various state, provincial and local DOTs	Types and timings of maintenance and rehabilitation treatments used for flexible and rigid pavements * Cost of treatments * Observed increases in pavement life/performance for each treatment type * Variations in strategies for different conditions* Barriers to implementation of new maintenance techniques.
Books and publications	Causes and mechanisms of pavement distress * Methods of cost-effectiveness evaluation * Pavement design procedures * INDOT Field Maintenance and Operations Manual.
NCHRP syntheses and reports on highway preventive maintenance	Evaluation of asphalt surface treatments and thin overlays * Pavement management methodologies to select and recommend preservation treatments * Role of highway maintenance in integrated management * Effective maintenance budgeting strategies * Cost-effective preventive maintenance.
Transportation Research Records	Past and current trends in performance modeling * Effectiveness evaluation of various modeling techniques* Assessment of maintenance strategies.
Research results from various institutions and organizations	Performance modeling techniques * Cost-effectiveness evaluation * Maintenance strategy formulations
Various reports on SHRP studies on maintenance effectiveness	Design of experiments for cost-effectiveness studies * Types and timings of maintenance and rehabilitation treatments used for flexible and rigid pavements * Cost of maintenance treatments * Observed increases in pavement life/performance for each maintenance treatment * Variations in strategies for different conditions.

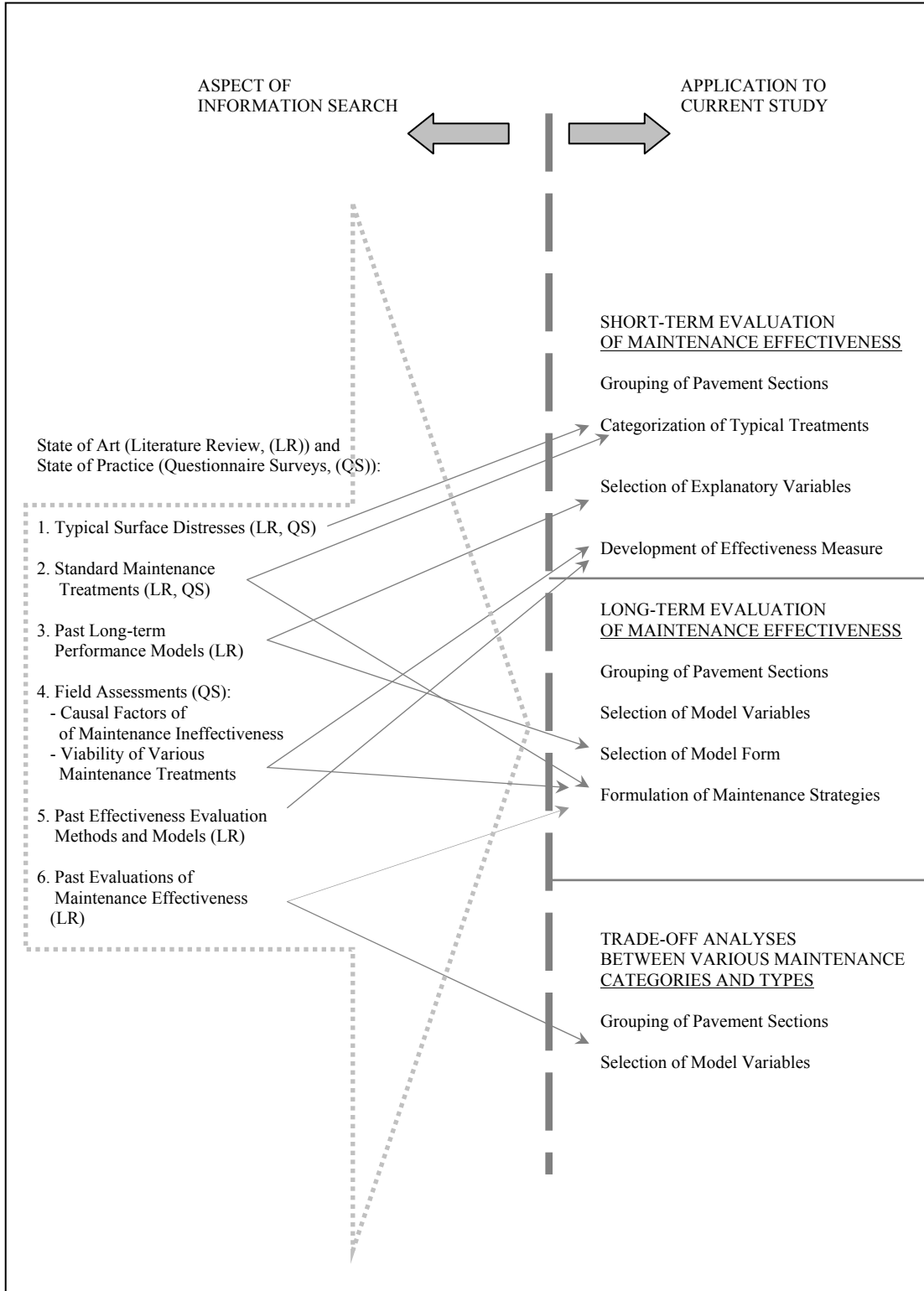


Figure 3-1: Relationships Between Aspects of Information Search and Study Objectives

3.2 Typical Pavement Distresses

There are several types of distresses on flexible and rigid pavements. Some distresses such as clogging of underdrains, joint seal deterioration, surface cracking, and raveling, are indicative of functional failure and generally require preventive maintenance treatment, such as underdrain maintenance, crack and joint (re)sealing, and seal coating, respectively. Other pavement distresses, such as alligator cracking, potholing, and bucking are associated with significant structural failure of the road structure and can only be addressed using corrective maintenance or rehabilitation. In 1994, SHRP published a manual that provides illustrated descriptions of at least 15 distress types each on flexible and rigid pavements [National Research Council, 1994]. Also, the Indiana Department of Transportation uses a distress identification manual that addresses pavement defects typically found in the state of Indiana.

A list of pavement distresses typically addressed by various preventive and corrective treatments is provided as [Tables 3-2 and 3-3](#) below [INDOT, 1997]. During each seasonal (fall and spring) road pavement inventory by INDOT's districts and sub-districts, the occurrence of any distress can be described in 4 ways: type, extent (percentage of roadway area or length covered), and severity (typically taken as the depth or width of the distress), and spread (how localized the distress is). These occurrence parameters help to decide on whether to take any action, what action to take, amount of work expected, and how to ensure efficient resource utilization for appropriate repair action (Labi, 1993).

[Table 3-2](#). Flexible Pavement Distress Types

Distress Category	Distress Type
Cracking	Alligator (Fatigue Cracking)
	Block Cracking
	Edge Cracking
	Longitudinal Cracking
	Transverse Cracking
Patching and Potholes	Pothole formation
	Patch deterioration
Surface Deformation	Rutting
	Shoving
	Bleeding
Surface Defects	Polished Aggregates
	Raveling and Weathering
	Lane-to-Shoulder Drop-off
Miscellaneous	Lane-to-Shoulder Separation
	Water Bleeding and Pumping

Table 3-3. Rigid Pavement Distress Types

Distress Category	Distress Type
Cracking	Corner Breaks (JCP only) Durability “D” Cracking Longitudinal Cracking Transverse Cracking
Joint Deficiencies (JCP only)	Transverse Joint Seal Damage (JCP only) Transverse Joint Spalling (JCP only) Longitudinal Joint Spalling Construction Joint Deterioration (CRCP only)
Surface Defects	Map Cracking and Scaling Polished Aggregate Popouts
Miscellaneous	Blowup Faulting of Transverse Joints and Cracks (JCP only) Lane-to-Shoulder Drop-off/Separation Patch/Patch Deterioration Water Bleeding and Pumping Punch-outs (CRCP only)

3.3 **Identification of Standard Maintenance Treatments**

The term “Standard Maintenance Treatments” is a misnomer, as there is currently no nationally recognized glossary of maintenance terms and activities [Geoffroy, 1996]. Nor is there any universally used document to guide the selection of appropriate preventive or corrective maintenance treatments to correct a specific distress condition. Consequently, the task of defining a standard list of maintenance treatments has been considered with caution.

3.3.1 **Current Problems with Standard Maintenance Treatment Identification**

3.3.1.1 *Lack of a Standard List or Guide to Correct a Specific Distress Condition*

Maintenance practices to correct a specific problem vary. For example, to correct the problem of moderate cracking one INDOT agency may choose to seal the cracks, while another may carry out micro-surfacing. This is neither unexpected nor undesirable. The effectiveness, and hence choice, of preventive maintenance treatments are expected to vary from one pavement to another, as pavement behavior is very

much influenced by climate, traffic, and subgrade, among other factors. Another consideration is that prevailing pavement maintenance cultures and practices in various agencies are the culmination of trial-and-error processes that evolved over several years, albeit usually without scientific documentation to justify such choices. One of the objectives of the present study is to shed light on this matter.

3.3.1.2 Lack of Standard Definitions for Maintenance Activities

3.3.1.2.1 Different Names for the Same Activity

A challenge encountered during the literature review and questionnaire surveys involves the differences in terminology used by various district and state highway agencies and pavement researchers to describe maintenance activities. For instance, “load transfer restoration”, “retrofitting” and “stitching” are typically used by different district agencies to describe the same activity. This problem was addressed by using the terminology adopted by recognized national programs (such as NCHRP and SHRP) and accepted by the INDOT Central Office.

3.3.1.2.2 Different Categories for the Same Activity

In including the experience of other states, this report took due cognizance of the fact that the categorization of maintenance terms varies from state to state. For instance, application of a two-inch thin HMAC surface may be described as a preventive maintenance by one agency, while other agency may consider that as a rehabilitation activity. In studying data from other states for their relevance to the present study, it is expected that different finance and budget requirements of other states may further exacerbate this situation. An activity that is considered a preventive maintenance in one state because it is funded from the maintenance budget may be considered a rehabilitation in another state because it is funded from the capital budget. Furthermore, an activity that is carried out on a pavement in relatively good condition may be described as preventive maintenance (such as stitching light cracks), while that same activity carried out on structurally-deficient pavement would be described as corrective maintenance [Mamlouk et al., 2000]. There are also variations in terminology based on whether the work is done by in-house forces or

by contract [Geoffroy, 1996]. In view of these issues, review of existing literature from various highway sources was done carefully to minimize any ambiguities in activity descriptions.

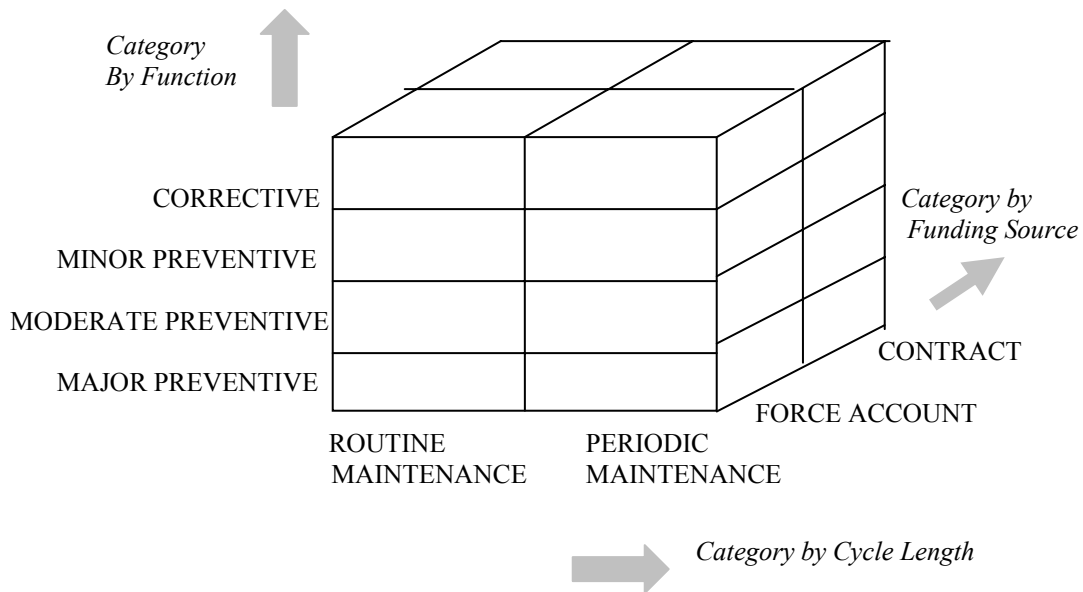
NCHRP's Report 223 offers two convenient criteria to categorize maintenance activities: urgency of the activity and effect of the activity. Using these and other criteria, the following pavement repair types are briefly described as follows [Geoffroy, 1996]:

- *Routine Maintenance*: Day-to-day activities that are scheduled, and whose timing is within the control of maintenance personnel, e.g., mowing, ditch cleaning. Generally speaking, “routine maintenance” is a broad term and is often used to describe any activity that is carried out on at relatively short intervals, such as routine preventive maintenance (such as crack sealing), routine corrective maintenance (such as patching), and non-pavement routine maintenance (such as mowing).
- *Demand Maintenance*: Urgent activities that must be done in response to an event beyond the control of the maintenance personnel, e.g., any emergency repair of a pavement.
- *Corrective Maintenance*: Planned activities to repair deficiencies, e.g., shallow patching, that aims at increasing structural capacity at a localized area only.
- *Preventive Maintenance*: Planned activities that correct minor defects, retard future deterioration, and maintain and improves the functional condition of the system, without substantially increasing the structural capacity.

For the purposes of the present study, maintenance is categorized mainly by function: either preventive or corrective, a categorization that has been used in past research [Sharaf et al., 1984; Zaniewski et al., 1999]. Also, distinction is made between “minor” preventive maintenance (e.g., joint sealing, joint/bump repair), which is localized, and “moderate” and “major” preventive maintenance (e.g., chip sealing and thin overlay, respectively), which cover the entire pavement surface.





It is important to note that thin HMAC overlay treatment, until fairly recently, was generally considered a rehabilitation activity. It is expected that current consideration of this treatment as a

preventive maintenance activity will have profound influence on the manner pavement maintenance (and its cost-effectiveness) is perceived. An attempt has been made to clarify and classify the different and sometimes conflicting terminologies found in the various literature reviewed, on the basis of maintenance function, cycle length, and funding source. This classification is shown as [Figure 3-2](#) and [Table 3-4](#). Rehabilitation is shown in [Table 3-4](#) only to show how this activity is related to other pavement repair actions. These definitions formed a basis for the categorization of work activities from descriptions provided by the various INDOT highway districts and sub-districts into the various maintenance and rehabilitation types. Those identified as preventive or corrective maintenance activities were singled out for the present study.



[Figure 3-2](#): Categorization of Pavement Maintenance

Table 3-4: Typical Treatments in Various Categories of Pavement Treatment Activities

INTERVAL AND FUNDING 			Routine Maintenance		Periodic Maintenance		Capital Investment
Function: ROLE LEVEL 	COVERAGE  		Force-Account	By Contract	Force-Account	By Contract	
	Only Affected Locations	Minor (localized)					
Preventive Treatments	Only Affected Locations	Minor (localized)	Crack Sealing Bump Repair	N/A	N/A	Undersealing Stitching	N/A
	Entire Surface (typically)	Moderate (thin coat)	N/A	N/A	Chip Sealing Sand Sealing	Chip Sealing	N/A
		Major (thin overlay)	N/A	N/A	N/A	Micro-surfacing Thin Overlay	N/A
Corrective Treatments	Only Affected Locations		Patching (Shallow and Deep)	N/A	N/A	Patching (Shallow and Deep)	N/A
	Entire Surface		N/A	N/A	N/A	N/A	Resurfacing/ Restoration Reconstruction ¹

N/A: Not Applicable.

1. For purposes of this study, capital investments such as resurfacing, restoration, and reconstruction are not considered as maintenance activities, and are shown here only for purposes of comparison.

From a preliminary review of available literature on the practices of preventive and corrective maintenance, a tentative list and description of “standard” preventive and corrective maintenance treatments in Indiana are provided below (also shown as [Figures 3-3 and 3-4](#)). For each preventive or corrective maintenance treatment or rehabilitation activity, the diagram indicates whether that activity is typically executed by in-house forces (under the force-account), or whether it is given out on contract under the capital expenditure account.

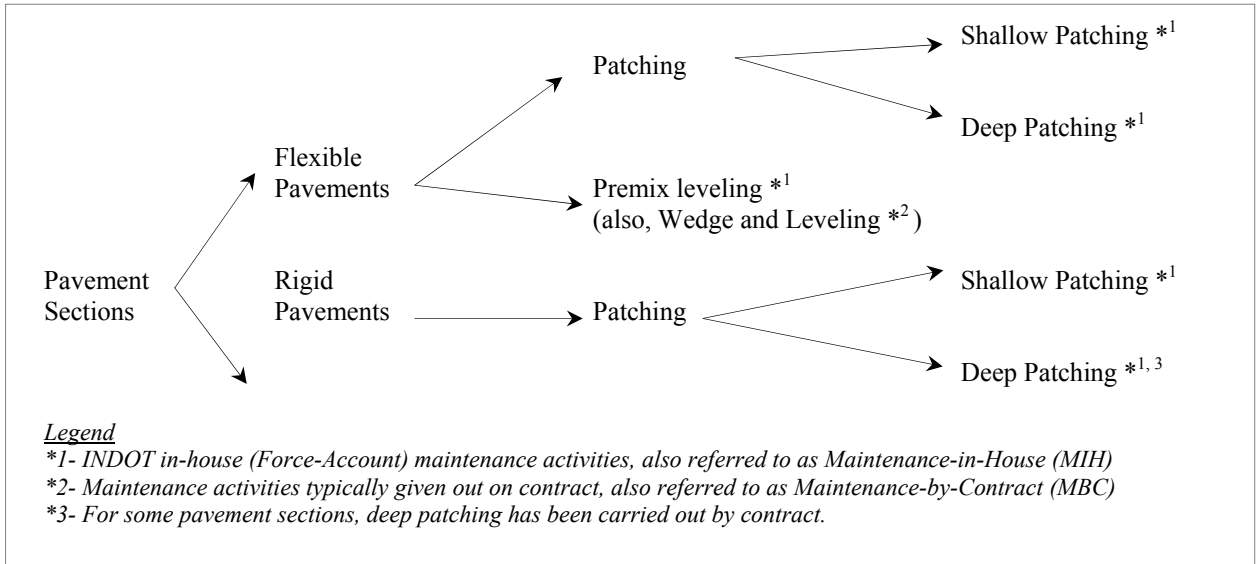


Figure 3-3: Typical Corrective Maintenance Treatment Types in Indiana

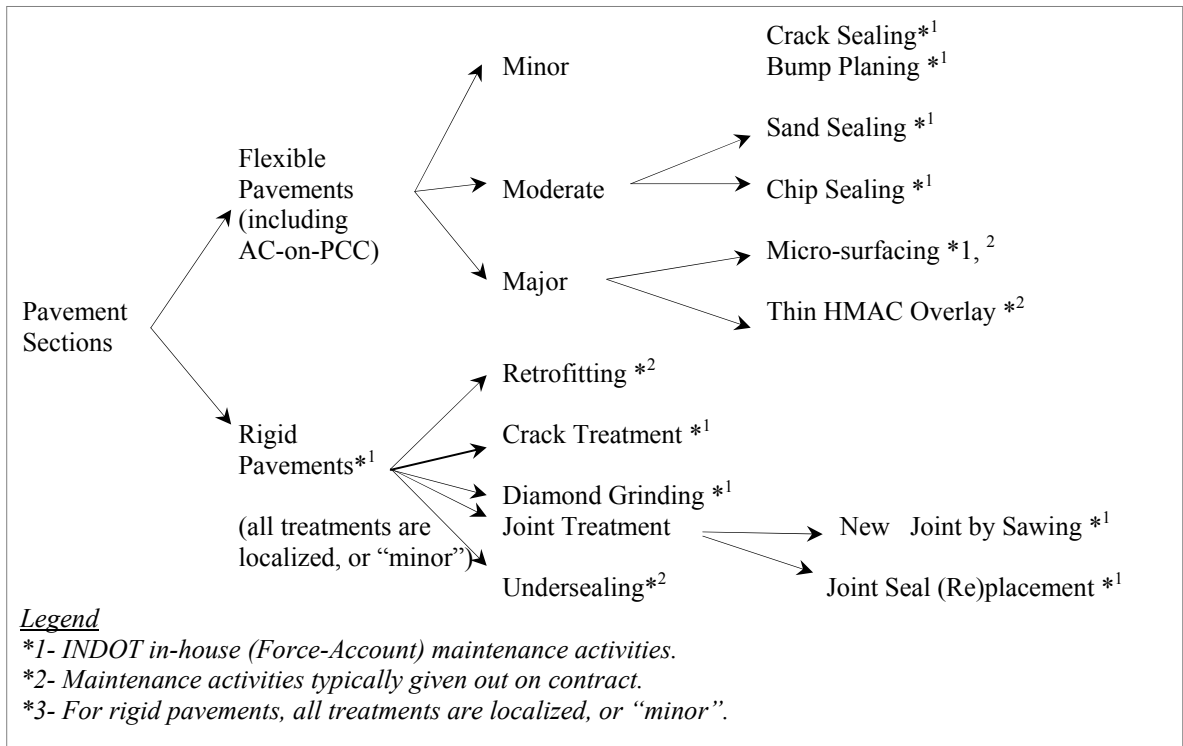


Figure 3-4: Typical Preventive Maintenance Treatment Types in Indiana

3.3.2 Non-pavement Routine Maintenance Activities (for all Pavement Types)

Non-pavement routine maintenance activities are scheduled day-to-day activities that are generally carried out to ensure efficient drainage. Such activities include vegetation control, drainage maintenance, and shoulder maintenance. The present study does not include such non-pavement routine maintenance activities as they have relatively very little direct or measurable impact on pavement performance. As mentioned previously, routine maintenance is a broad term that is often used to describe any activity that is carried out at relatively short intervals, and includes pavement and non-pavement treatments such as crack sealing, patching, and mowing.

3.3.3 Flexible Pavement Preventive Maintenance

3.3.3.1 Crack Sealing

Crack sealing, the placement of specialized materials either above or into cracks to prevent the intrusion of water and incompressibles, is commonly conducted using the simple squeegee method and the countersink methods. The squeegee method, which is simple and quick, involves cleaning out the crack using compressed air and spreading a hot asphalt mastic sealant over the crack with a squeegee. Routing and sealing cracks minimize the growth of secondary cracks and can increase service life by at least 2 years [Ponniiah and Kennepohl, 1996]. In the countersink method, the crack is routed or countersunk to form a reservoir for the sealer, and a high-quality asphalt filler is used as the sealing material. Pavements treated with this method have performed relatively well, even though this treatment is relatively expensive [Chong and Phang, 1988]. Sealants may last for only a few years and therefore require monitoring and frequent application. Studies in Indiana showed that crack sealing has a significant impact on roughness measurements [Mouaket and Sinha, 1990]. Ontario's MTC carried out a number of field investigations to assess the consequences of not sealing cracks in flexible pavements [Ponniiah and Kennepohl, 1996]. The results of that experiment indicated that not sealing cracks increases maintenance costs, decreases pavement serviceability, and increases vehicle user costs. In some cases, however, some sealant bonds failed prematurely, resulting in ingress of water into the pavement.



Figure 3-5: Crack Sealing at Sub-district Level in Indiana

3.3.3.2 *Bump Planing*

This refers to the heating and/or planing of bituminous surfaces to remove bumps, ripples and other surface irregularities [INDOT, 1998]. This activity is used to restore ride quality and pavement serviceability (See Figure 3-9).

3.3.3.3 *Surface Treatments*

Surface treatments are normally used on existing pavements to improve skid resistance and to waterproof the underlying pavement layers [Brown, 1998]. For a road section having relatively few cracks, it may be more cost-effective to fill or seal the cracks. However, when the surface of the pavement is plagued by an extensive occurrence of cracks, it may be better to apply a surface treatment to that section instead of (or in addition to) crack sealing.

3.3.3.3.1 Chip Seals:

This is a surface treatment that is carried out by spraying cold asphalt emulsion or hot bitumen on the pavement, followed by spreading a layer of small crushed stone. INDOT's Field Operations Manual [INDOT, 1998] describes chip seal coating as "... a continuous full-width application of hot bituminous material and coarse aggregates to correct extensive cracking, spalling, shallow surface failures, and to

prevent further surface deterioration”. Chip seals are usually used to fill cracks and to stop the development of further cracks. It is also used as a blanket cover on pavements suffering from loss of skid resistance, oxidation, raveling, spalling, and surface permeability. Because of its larger thickness, chip seals are generally considered as superior to sand seals, but are more expensive. For example, chip seals in Indiana cost almost twice as much as sand seals [Mouaket et al., 1992]. Generally, chip seal construction is avoided on high-traffic volume pavements because of vehicle damage due to flying chips, relatively short life expectancy, and excessive noise and roughness associated with a chip sealed surface [Shuler, 1998].

Chip seals, which are placed either in single or in multiple layers, are considered more appropriate (compared to sand seals) for cracked, spalled, or raveled pavements.

Studies have shown that it may be possible to construct a chip seal surface for pavements with traffic volumes as high as 7,500 vehicles per lane that significantly reduces such problems by adopting certain construction techniques [Shuler, 1998]. The use of smaller pre-coated chips has been recommended to reduce the hazard posed by flying chips [Mouaket et al., 1992]. In the State of New York, chip sealed sections have been found to extend pavement life by 3-4 years, depending on the level of traffic [New York DOT, 1992]. Many agencies use chip sealing as a stop-gap measure to defer capital spending, by applying this treatment to pavements approaching the end of their expected service lives. In Manitoba, chip seals are known to extend pavement life by 10-12 years [Young et al., 1986]. However, the overall success of this maintenance treatment is largely attributed to the availability of good aggregates and relatively dry weather (Mohammed-Asem et al., 1993).

3.3.3.3.2 Sand Seals:

This treatment consists of a spray application of emulsion or hot bitumen, followed by spreading a layer of fine aggregate. INDOT’s Field Operations Manual describes sand seal coating as a “continuous full-width application of hot bituminous material and fine aggregates to correct extensive cracking, and spalling, ... this preventive maintenance technique is often used to restore a weathered or oxidized surface” [INDOT, 1998]. The seal coat layer helps prevent the loss of surface material due to traffic wear and prevents the intrusion of moisture. If the sand used for sealing is clean, sharp and angular, significant

improvements to surface texture can be obtained. Sand sealing is deemed more effective than chip sealing for cases of pavement oxidation and bleeding, however some practitioners may not agree with this view.

3.3.3.4 *Micro-surfacing*

A relatively new maintenance treatment, micro-surfacing involves the laying of a mixture of polymer-modified asphalt emulsion, crushed mineral aggregate, mineral filler, water, and a hardening-controlling additive. The micro-surfacing process involves the use of a self-propelled traveling pug mill in which the components materials are mixed immediately before laying, and no rolling of the micro-surfacing layer is required [Dwight Hixon and Ooten, 1993]. Generally, micro-surfacing is used to correct rutting and to improve surface texture as it can be placed in layers of up to 50mm thickness. This preventive maintenance technique has been used on both rigid and composite pavements to improve texture and friction, and to fill ruts. The existing condition of the pavement, as well as construction quality and traffic loadings, is a critical factor for the success of this treatment type. Under favorable conditions, micro-surfacing have been found to perform well for 5-7 years [Raza, 1994]. After nine years of experimental use in the State of Oklahoma, micro-surfacing was found to correct and retard pavement rutting, improve friction, and fill alligator and depression cracks. However, its success on PCC pavements has been limited [Dwight Hixon and Ooten, 1993].

3.3.3.5 *Thin Hot Mix Asphalt Concrete (HMAC) Overlays*

When crack occurrence is extensive and traffic volumes are high, some agencies consider the use of thin HMAC overlays as more cost-effective than chip seals or other treatments. The gradation of aggregates used for HMAC overlays could be dense-, gap- or open-graded. Dense- and gap-graded mixes seal the pavement surface and improve ride quality and surface friction. Open-graded mixes enhance the ability of water to drain of the pavement surface and improve ride quality and surface friction. The service life of thin dense-graded overlays range from 2 to 10 years [Geoffrey, 1996], however, gap- and open-graded mixes tend to perform longer, partly due to their improved flexibility [Hicks et al., 1997].

3.3.4 Rigid Pavement Preventive Maintenance

The overall performance of a rigid pavement may be divided into functional and structural performance [Yu et al., 1994]. Functional performance relates to the pavement surface characteristics or profile, and how it interacts with moving vehicles. Structural performance relates specifically to the ability of the pavement to carry load. Preventive maintenance activities on rigid pavements are generally designed to address functional deficiencies. Indicators of functional deficiency of rigid pavements include:

- Decreased surface friction resulting from polished wheel paths
- Roughness due to concrete durability problems
- Inadequate cross slope, leading to poor surface drainage
- Rutted pavement surface due to the wearing effect of studded tires or chains

An agency's choice of appropriate preventive maintenance activity is typically preceded by assessment of any observed deficiency. Descriptions of typical preventive maintenance activities on rigid pavements and the experiences of some highway agencies with each activity in the State of Indiana and elsewhere are discussed below.

3.3.4.1 Joint and Crack Sealing

The critical importance of this preventive maintenance activity is emphasized in NCHRP Synthesis 211, which states that “perhaps the two most cost-effective preventive maintenance activities are cleaning and other maintenance of drainage features and resealing of joints” [McGhee, 1995]. This activity involves the sealing of transverse and longitudinal joints on the pavement, the joint between the pavement and shoulder, and any cracks on the pavement. This is carried out using any of several methods detailed in INDOT's Field Operations Manual [INDOT, 1998]. Properly sealed joints and cracks prevent the deposition of incompressible material in the joints and reduce the level of water infiltration into the pavement structure [Geoffroy, 1996]. It has been shown that the life of a PCC joint seal ranges from 2-8

years, depending on the care taken to clean and prepare the crack or joint opening, the type of joint material used, and the care taken to place the material [Belangie, 1990].

Currently there is some controversy about the effectiveness of joint sealing. Several long-term experiments carried out in Wisconsin since the 1950s have shown that pavements with unsealed joints generally yielded better performance than those with sealed joints [Shober, 1986; Shober, 1997]. Also, where thermally locked joints in dry climates and/or coarse-graded subgrades exist, the practice of using a single 3mm saw-cut joint [without sealing] may be cost-effective when evaluated in the context of long-term pavement performance [Morian and Stoffels, 1998]. These researchers further state that leaving joints unsealed may prove acceptable where positive drainage features are naturally occurring (coarse grained subgrade) or where climates are very hot and dry, resulting in minimal joint movement. But they caution that joint sealing may be appropriate for jointed pavements with fine-grained subgrades in wet climates. Ray reported that Spain and Austria build many kilometers of pavement with unsealed joints and that France and Germany have both built substantial test sections with unsealed joints [Ray, 1979].

On the other end of the argument are researchers who identified the various modes of joint failure and stressed the importance of joint maintenance [Belangie, 1998], and have demonstrated that sealing of cracks on rigid pavements has a significant effect on pavement performance, regardless of highway class, climate and loading levels [Mouaket and Sinha, 1991]. Morian and Stoffels assessed the practice of joint sealing in the United States and stated that early findings from the LTPP SPS experiments indicate that joint seal sections are generally performing better than unsealed sections [Morian and Stoffels, 1998]. There are several publications that recommend sealing of joints and cracks to increase pavement life [Chong and Phang, 1988; Chong, 1990; McGhee, 1995]. It is however generally agreed that long-term monitoring of sealed and unsealed sections at various geographic locations is necessary to enable useful conclusions to be drawn as regards the relative cost-effectiveness of these alternative activities [Shober, 1997; Geoffroy, 1996].

3.3.4.2 Joint Repair

Spalling of longitudinal and transverse joints is the cracking, chipping, or fraying of the concrete at the slab edges within 2 ft of the joint. Degradation of the concrete starts at the joint and widens on one or both sides to about 6 inches, usually deepening as it widens. Spalling is caused by a number of factors such as infiltration of incompressibles into joints and cracks, D-cracking, alkali-silica reactivity, joint lock-up, and joint inserts. Infiltration of incompressibles, which is probably the most frequent cause of spalling, occurs from the top or from the bottom of the slab. During cool weather, jointed PCC pavement contracts and unsealed/improperly sealed joints and cracks open and incompressible matter lodge in such crevices. During warm weather the pavement expands, closing the joints. However, incompressible matter in the joints prevent the joints from closing and produce high compressive stresses along joint faces, causing spalling at both the top and bottom of the slab and increased potential of slab blow-up because of decreased contact area of adjacent slab surfaces (Figure 3-6). Slab-bottom spalling is not visible from above, and is typical of reinforced concrete pavements with relatively long joint spacings.

Corrosion of mechanical load-transfer devices and reinforcement, especially when they are placed too near the surface often leads to development or widening of transverse cracks and subsequent spalling. Depending on the cause, spalling can initiate from any point through the thickness of the slab: the top, middle, or bottom [Yu et al., 1994].

According to NCHRP 211, joint distress is related to reactive aggregates: coarse aggregate particles in the mix expand and exert disruptive forces in the matrix. The aggregate expands because the aggregate silica and cement alkali reaction results in products that occupy greater volume than the original aggregate structure.

The level of maintenance required to address spalling depends on the severity of the problem. Partial depth repairs are used when deterioration is located primarily in the upper third area of the slab while the load transfer device are still functional, and when spalling is caused by corrosion of metal inserts and misplaced reinforcing steel. However, full-depth repairs are considered more appropriate for spalls caused by mis-aligned dowel bars, D-cracking, or alkali-silica reaction as the extent of damage caused by these factors often occur all along the depth of the slab [Yu et al., 1994].

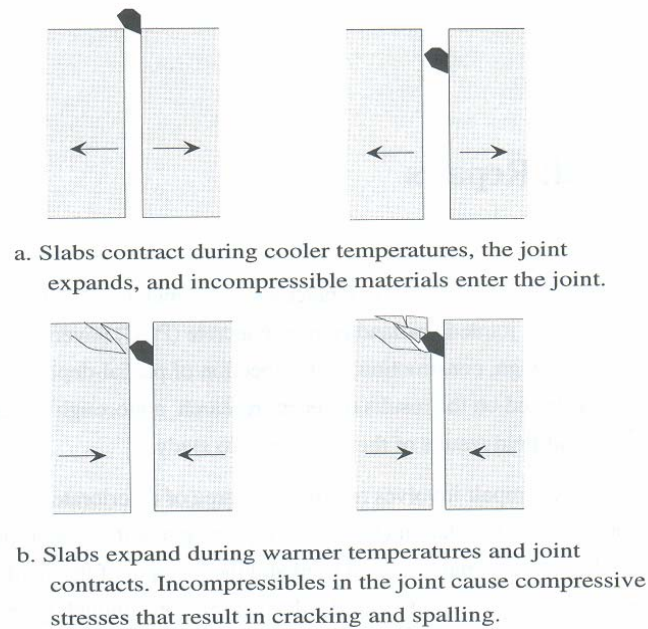


Figure 3-6: Mechanism of Spalling at Joints or Cracks [Yu et al., 1994].

3.3.4.3 *Stitching (Load Transfer Restoration)*

Many jointed concrete pavements in the United States were originally constructed without mechanical load transfer devices across their joints, and significant faulting has occurred on some of these pavements as a result of poor transfer of wheel loads from the upstream slab to the downstream slab. Many other jointed concrete pavements were constructed with dowels, but under heavy traffic such dowels became loose. Such failures of load transfer devices across transverse joints lead to pumping and slab failure. Development of cracks around the joints or at other sections of the slab is often indicative of imminent failure of the load transfer device. Highway agencies have used various devices such as retrofitted dowel bars, double-V shear devices, figure-8 devices, and miniature I-beam devices to stitch such cracks [Hall et al., 1993].

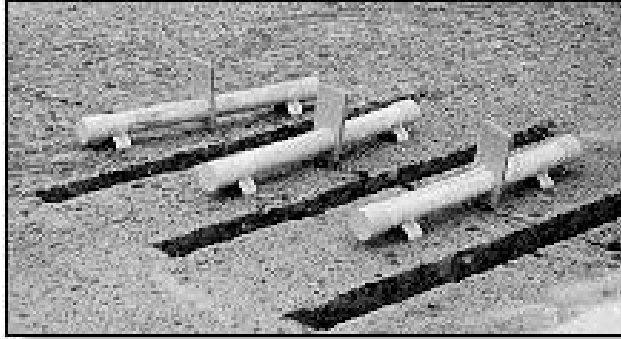


Figure 3-7: Dowel Bars Ready for Installation in Sawed Grooves in Jointed Concrete Pavement for Load Transfer Restoration [RTT, 1997]

Load transfer is typically done in conjunction with diamond grinding to remove existing faults at joints and cracks. This restores the integrity of load transfer across the joint. This preventive maintenance treatment has been successfully used to extend the lives of several rigid pavement sections in Puerto Rico by over 10 years [Ferragut and Papet, 1994]. Its use in the United States was generally hampered by the lack of inexpensive means of carrying out that treatment, until a special FHWA study, “SP-204-Retrofit Load Transfer”, identified means of reducing the rather high unit cost associated with that operation. This maintenance activity has been carried out at certain locations of the Interstate 70 near Indianapolis. This maintenance treatment is not directly considered in this study, as available data does not indicate the precise location of such treatments.

3.3.4.4 *Relief Joint Provision*

This activity refers to the sawing of the concrete slab to provide new provision of relief joints at certain locations, especially at locations near the end of bridge decks, to allow for expansion of the slab. INDOT’s Field Operations Manual describes this activity as “installation of relief joints in the pavement surface near the ends of bridge decks, where excessive blow-up are [imminent] to allow for expansion of the pavement and structure” [INDOT, 1998].

3.3.4.5 Under-sealing (Slab-jacking)

Pavement pumping is closely related to joint faulting. This phenomenon involves expulsion of water from beneath the concrete slab through joints and cracks under the action of repetitive wheel-loads [National Research Council, 1994]. In severe cases, pavement material (especially fines) from underneath the slab go into suspension, and at each pass of a wheel load, a water/fines slurry is pumped out through the crack or joint. Cyclical pavement deflections gradually produce small voids under the slab. Under-sealing is a type of maintenance that fills any existing voids that exists under pavement slabs (Figure 3-8). The operation involves pressure pumping of material such as cement, bitumen, or other pozzolanic slurries into the void, in an effort to restore support for the concrete pavement.

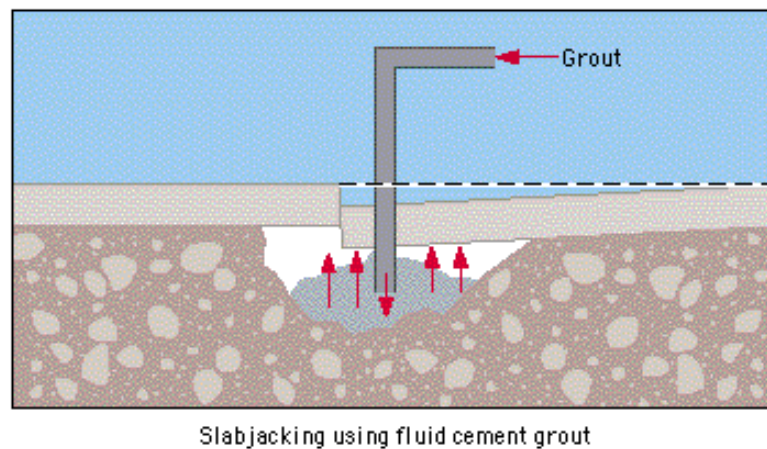


Figure 3-8: Under-sealing (slab-jacking)
Source : <http://www.atlasrestoration.com/100.htm>

3.3.4.6 Diamond Grinding

In the past, many jointed concrete pavements were constructed without mechanical load transfer devices across joints, and have often suffered from faulting as a result of poor load transfer from one slab to the next, ultimately leading to development of transverse cracks and faulting [Hall et al., 1993]. Faulting, which is the vertical displacement of abutting slabs at joints or cracks [National Research Council, 1994], is caused by repetitive wheel loading [Geoffroy, 1996] and leads to a difference in elevation across a joint or

crack on a rigid pavement. Increasing severity of faulting means increasing difference in elevation. As a wheel approaches the joint, water and suspended solids beneath the approach slab are forced into the area beneath the departure slab. When the wheel crosses the joint onto the leave slab, the slurry is forced back to the area underneath the approach slab with high velocity. This action causes pumping and erosion of underlying material, leading to void development under the leave slab and a build up of material under the approach slab, resulting in the lifting of the approach slab [Geoffroy, 1996].

Faulting can be considered as a symptom of a distress, rather than being a distress itself. It is symptomatic of failed load transfer across slabs, which is generally caused by non-existent dowels, inadequate number or size of dowels in place, or corrosion of existing devices. As faulting is a progressive type of distress, many states use diamond grinding as a preventive maintenance technique to correct less severe occurrences of faulting and to retard the further development of this distress (Figure 3-9). In its severe form, faulting has been repaired by corrective maintenance activities such as slab replacement and/or dowel provision [Yu et al., 1994]. NCHRP Report 211 states that the philosophy of joint fault repair has undergone a major transition over the past two decades: over twenty years ago, almost all corrective effort involved slab jacking, and in severe cases, total removal and replacement of the affected slab. However, almost all agencies now use selective grinding of joint areas to eliminate the level differential across adjacent slabs. In some cases, grinding is accompanied by undersealing to fill any voids under the pavement and delay future faulting. Grinding is generally considered feasible when joints are faulted no more than 6 mm and if the pavement has not been previously ground.

NCHRP's Report 211 further states that joints that have been ground typically perform well for several years before the faulting again gradually develops to a stage where further corrective action is necessary. According to Peterson [1989], full-depth removal and replacement, if justified by economic analysis, may be carried out where grinding is not feasible.



Figure 3-9: Diamond Grinding Operation [Correa and Wong, 1997]

3.3.4.7 *Underdrain Maintenance*

Many premature pavement failures have been attributed to inadequate subsurface drainage. It has generally been recognized that water in the pavement is undesirable, and attempts to reduce this hazard include sealing the surface joints and cracks, constructing permeable base courses, and providing underdrains during construction or for existing pavements. It has been determined that the maintenance of subsurface drainage systems to ensure its efficient functioning is vital to the long-term effectiveness of such systems and that the use of video cameras (Figure 3-10) for inspection is recommended for effective maintenance [Christopher and McGuffey, 1997].

Using case studies, it has been shown that pavement service life increases by at least 33% and 50% for asphalt and PCC pavements, respectively [Forsyth et al., 1987], and it has been found that pavement life can be extended significantly if adequate subsurface drainage systems are installed and maintained properly [Christopher and McGuffey, 1997]. Compared with drained sections, the service life of undrained pavement sections in France have been known to suffer a 70% reduction in service life [Ray, 1981].



Figure 3-10: Silted Under-drain Pipes Identified by Video Inspection [Christopher And McGuffey, 1997]

3.3.5 Preventive Maintenance on AC-over-PCC Composite Pavements

3.3.5.1 *Sawing and Sealing*

It is generally believed that cracking of bituminous surfacing constructed over semi-rigid and rigid road bases results from reflection cracking generated in the road base. This is because reflection cracking has been observed to be very prevalent on AC-on-PCC composite pavements and on flexible pavements with cement-stabilized bases. Pavement researchers generally agree that horizontal and differential vertical movements at joints and cracks in the existing pavement cause reflection cracks in AC-on-PCC overlay pavements [Kilareski and Bionda, 1997].

Horizontal movements, which are considered more damaging, are caused by three factors: daily temperature cycle, seasonal temperature changes, and traffic loadings. Seasonal temperature changes and daily temperature cycles cause expansion, contraction, and curling stresses in the existing base slab and the overlay as illustrated in [Figure 3-11](#). A change in moisture content causes the base slabs to warp, creating stress concentrations in the overlay that lead to cracking. The extent of cracking depends on the temperature change, thermal properties of the top and base materials, joint and crack spacing, and interlayer friction. Quin-Lin states that the problem of reflection cracking is especially prevalent in new flexible pavements with semi-rigid bases where insufficient time was provided for shrinkage of the road-base prior to laying of the bituminous surface layer [Quin-Lin, 1988]. In the case of old PCC pavements overlain with

an AC surface layer, the problem of reflection cracking occurs due to temperature-induced stresses and strains in the underlying PCC slab [Yu et al., 1994].

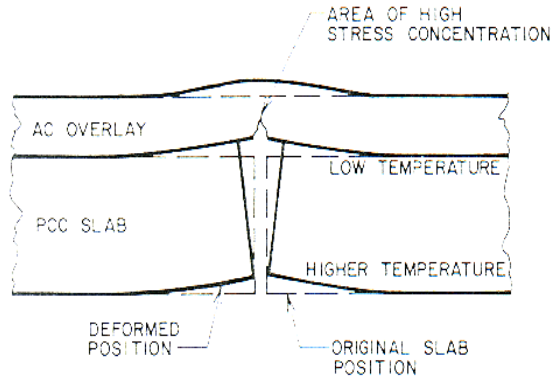


Figure 3-11: Thermal Curling of Underlying Slab in Overlays [Kilareski and Bionda, 1997].

Sawing and sealing is a preventive maintenance activity that involves sawing a joint in the AC overlay above the existing joint and sealing the joint. The performance of AC overlays with this maintenance technique was a subject of a national study in which pavements with up to 10 years of service life were evaluated through condition surveys, roughness measurements, and deflection measurements [Kilareski and Bionda, 1997]. Both saw-and-seal and control sections were evaluated. The analysis indicated that sawing-and-sealing improves the rideability of the AC overlay and significantly reduces the amount of transverse reflection cracking.

3.3.6 Flexible Pavement Corrective Maintenance

Highway agencies typically prescribe corrective maintenance treatments for pavements that are found to be structurally deficient. Such treatments are discussed in subsequent sections.

3.3.6.1 Partial Depth Repairs (Shallow Patching)

This is described as a project of limited scope where failures, holes and other defects are patched to a partial depth using bituminous material. The performance standards of INDOT's Division of Operations Support describes this activity as "minor patching small areas of bituminous roadway with hot

or cold bituminous mixtures to correct potholes, edge failures and other potential hazards". This activity also includes temporary patching of bituminous and concrete surfaces and the use of hot bituminous material and aggregates for patching bituminous surfaces or crack and joint spalling of concrete surfaces [INDOT, 1998].

3.3.6.2 Full-depth Repairs (Deep Patching)

According to the performance standards in INDOT'S Field Operations Manual, deep patching includes the full depth removal of surface and base material and replacement with compacted bituminous mixture [INDOT, 1998]. Full-depth repair is described as a project of limited scope where failures, holes and other defects are patched to full depth using bituminous material.

3.3.6.3 Premix Leveling

The performance standard of the Division of Operations Support describes this activity as minor machine or hand leveling and wedging of small isolated areas of bituminous or concrete roadway and shoulder surfaces with hot or cold bituminous mixtures to correct depressions at bridge ends, surface failures and depressions caused by pipe replacements and deep patches [INDOT, 1998].

3.3.7 Rigid Pavement Corrective Maintenance

Structurally deficient pavements are candidates for corrective maintenance. Visible signs of structural deficiency in rigid pavements include corners breaks, transverse working cracks, shattered slabs or failed repairs of these distresses in jointed concrete pavement [Yu et al., 1994]. In CRCP pavements, structural distress is often manifest by punch-outs. Corrective maintenance activities are designed to provide sufficient strength at a localized location of distress and involve removal and replacement of part (partial-depth) or whole (full-depth) pavement structure at that location.

On the other hand, rehabilitation, in the context of this study, generally involves leaving the original pavement layer intact (albeit sometimes repairing minor surface defects), and placing a new layer (overlay) over the entire stretch. Concrete pavement restoration (CPR) is a large scale effort involving

various treatments to repair extensive and severe defects on a rigid pavement. CPR constituent treatments include partial and full-depth patching, under-sealing, grinding, and retrofitting. In the present study, CPR as a rehabilitation activity is not considered.

3.3.7.1 Full-Depth Repairs

Full-depth repairs are used to restore the structural integrity and rideability of concrete pavements at spots where certain structural deficiencies distress types have been observed. Distresses that may warrant full depth pavement repair include faulting or spalling where over 1/3 of the pavement surface is affected, joint lock-up, and slab-breakup [Yu et al., 1994]. Full-depth repair involves sawing the pavement to its full depth, carefully removing the distressed slab without damaging the adjacent slabs, removing and replacing subbase material and providing drainage, if necessary, and placing the concrete. Dowels are anchored in adjacent slabs to enhance load transfer to and from the new slab.

3.3.7.2 Partial-Depth Repairs

Partial-depth repairs for concrete pavements are used where concrete deterioration is confined to the top 1/3 of the slab exhibits certain distress types. Repairing surface spalls and popouts this way can improve the rideability of JCP pavements and reduce moisture infiltration and intrusion of incompressibles into the joints. SHRP's Users Manual for concrete pavement rehabilitation recommends that partial depth repair on existing PCC pavements be considered prior to AC or bonded PCC overlay [Yu et al., 1994]. This corrective maintenance activity involves saw cutting the pavement to an appropriate depth, and removing and replacing the deteriorated concrete.

3.3.7.3 Repair of Deteriorated Construction Joint (CRCP only)

This is the repair of a series of closely spaced transverse cracks or several interconnecting cracks near the construction joint. This distress progresses from a condition with only light cracks to one with moderate or high degree of spalling or faulting, and leads to eventual breakup of the material within 10 ft of the construction joint [National Research Council, 1994].

3.3.7.4 *Punch-outs Repair (CRCP only)*

The development of two closely-spaced cracks near the pavement edge, and a short longitudinal crack between the transverse cracks, results in the “carving” of a rectangular area on the concrete surface [National Research Council, 1994]. With time and traffic loading, these cracks widen and deepen, and the steel reinforcement is ruptured, leading to loosening and downward punching of the concrete block formed within the cracks (Figure 3-12). The repair of this defect is referred to as punch-out repair. This repair activity may have been given a “shallow patching” description in INDOT Operations Support Division’s maintenance records. Therefore, it was not possible to determine the effectiveness or unit cost of this treatment, but this may be possible, assuming that shallow patching at CRCP sections are actually punch-out repairs.



Figure 3-12: Punch-out on CRC Pavement [SHRP, 1993]

3.4 Past Studies on Short-Term Maintenance Effectiveness and Expenditure Modeling

The literature review for short-term effectiveness of maintenance activities included a review of published information on two other areas that are closely related to effectiveness: maintenance decision and annual pavement maintenance expenditure. Maintenance decision models help predict whether a certain type of maintenance will be carried out at a future year, given the pavement attributes for that year, while annual pavement maintenance expenditure models enable the estimation of the expected level of expenditure at a future year for maintenance of a given pavement section. A review of available literature on each of the above categories of short-term modeling was carried out in order to obtain an insight into the approaches used and problems encountered in past studies.

3.4.1 Maintenance Decision and Expenditure Models

3.4.1.1 Maintenance Decision Models

Short-term maintenance choice models are probability-based discrete choice functions that estimate the likelihood of carrying out maintenance (or maintenance of a certain type) versus the probability of not carrying out maintenance, given an array of explanatory variables (pavement attributes, and sometimes, attributes of the maintenance treatment). Such models are often resorted to in the absence of pavement maintenance expenditure data for each pavement section. The response variable takes on a value of 1 if maintenance is carried out, and 0 if maintenance is not carried out. Even though maintenance decision modeling was not carried out in this report, it was useful to extend the literature review to previous maintenance effectiveness studies that have involved decision modeling in order to highlight certain features of those studies that are relevant to the present study.

Maintenance decision models are based on the theory of probabilistic choice (Ben Akiva and Lerman, 1985; Pindyck and Rubinfeld, 1991]. The development of such theories arose from the need to explain inconsistent preferences of individuals that were observed in an experiment at that time. In choice experiments, individuals were observed to select alternatives at different times even when faced with the same choice set. In the context of pavement maintenance, the “individual” is the field inventory team or

pavement manager, and the choice set is the array of alternative maintenance treatments from which the individual chooses to apply to a given pavement section. Each alternative (treatment) is associated with a certain utility, such as the highest effectiveness (e.g., reduction in roughness). Because effectiveness is a function of pavement attributes (type, location, loading, subgrade, etc.) and sometimes also a function of treatment attributes (e.g., cost), utility can be expressed as a direct function of these attributes. This consideration has led to two possible errors in past studies: both effectiveness and attributes have been included in the utility function.

Discrete choice models for pavement maintenance decisions can be carried out in one of 2 ways: the Constant Utility approach and the Random Utility Approach. The constant utility approach, which takes its roots in mathematical psychology, hypothesizes that the utility of each alternative is fixed, and that the individual's (pavement manager) choice of any alternative is a function that includes these utilities as parameters [Ben Akiva and Lerman, 1985]. In the random utility approach, which is more in line with neoclassical economic behavior, utilities are not constant but are random variables. The non-constancy of utilities is due to unobserved variables, unobserved preferences, measurement errors, and the effect of instrumental variables. In any case, both approaches assume that the decision-maker (pavement manager) makes a choice that is associated with maximum utility.

The most popular model forms used for estimating decision are the probit and logit models. Unlike the linear probability model, such models have error terms that are not uniformly distributed, and therefore obviate problems that renders linear probability models prone to heteroscedasticity (non-constancy of error variance). Model forms with heteroscedastic error terms generally yield coefficients that that may be unintuitive and predicted values that may lie outside the range of the domain of response variables [Greene, 1999]. Because the error term of linear probability models is heteroscedastic, it cannot be guaranteed that the predictions from this model will fall between 0 and 1, or that the variances will be non-negative. However, researchers have evidently not given up on the linear probability model, and research on the use of this model form is very much in progress. As regards other model forms, the binary probit model has been described as "intuitively reasonable" and as having some theoretical grounds for its assumptions about the error term [Ben Akiva and Lerman, 1985]. However, that model type has been

associated with analytical difficulty of expressing choice probability as an integral. The use of the binary logit model obviates this problem. An example of recent work in Indiana that used a logit model to estimate the probability of carrying out maintenance on Interstate pavements in Indiana, is shown as follows [Mohammad et al., 1997]:

$$\text{Probability of maintenance} = e^B$$

$$\text{Where } B = -4.68 + 1.08 * RN_{t,n} + 0.003 * ESAL_{t,n} - 0.21 * THICK_{t,n} - 0.94 * RG_{t,n}$$

RN = log of roughness number, n = number of observations, t = year of study

AGE = number of years since last rehabilitation

$ESAL$ = equivalent single-axle load repetitions

$THICK$ = pavement thickness in inches

RG = dummy variable: 0 for pavements in northern Indiana, 1 for southern Indiana.

This model showed that the likelihood of maintenance increases with lower pavement condition, higher annual loading, thinner pavements, and northern location of pavement section. It was not indicated whether interaction terms were investigated, as a strong interaction is expected to exist between ESALS and pavement thickness [George et al., 1993; Paterson et al., 1993]. The model results were generally intuitive. For instance, the finding that pavements located in Northern Indiana were associated with a higher likelihood of maintenance was in accord with past research [Mouaket et al., 1991; Fwa and Sinha, 1992]. A unique aspect of that study was that the researchers identified the restrictiveness of the assumptions made in previous modeling efforts (that past maintenance has a unilateral and exogenous effect on pavement performance) and therefore made efforts to avoid resulting simultaneity bias in such formulations. Therefore the above equation was actually part of a 2-stage model that was estimated simultaneously (the other part was a deterioration prediction model), and econometric methods were used to arrive at more intuitive model coefficients.

Multinomial logit models were developed to estimate the probability of maintenance and the probability of rehabilitation on pavement sections [Madanat and Mishilani, 1995]. The researchers argued that because the sections that received maintenance were not randomly selected among all pavement sections (in other words they were chosen because their need for maintenance was perceived), the sample

could be described as self-selected. They therefore included a correction term to account for the presence of selectivity bias. However, the coefficient of the correction term was not consistently significant in the models they developed, implying that the problem of selectivity bias may not be significant in all cases.

3.4.1.2 *Maintenance Expenditure Models*

Short-term maintenance expenditure models estimate the annual maintenance that a pavement receives in any point in time, either in terms of age or accumulated loading. These models are considered superior to maintenance decision models as they estimate not the probability of maintenance, but the level of maintenance in monetary terms. This is typically done in one of three ways:

- i) Using the average of annual maintenance expenditure values that all pavements in a certain category and of a certain usage level (age group) receive,
- ii) Calculating the average annual maintenance expenditure incurred by each pavement section over a period, and modeling these average values as a function of pavement attributes, to yield average annual maintenance expenditure models (AAMEX) [Mamlouk et al., 1996; Li and Sinha, 2000], or
- iii) Using the individual annual maintenance expenditures of each pavement section at each year, (rather than the average of such values over time) as the response variable in a model to estimate such expenditure as a function of pavement attributes (this yields annual maintenance expenditure models (AMEX)). This is similar to the data “pooling” approach described in some statistics literature. The present study utilizes this approach.

A further detailed discussion of these approaches is presented in Chapter 5.

Maintenance expenditure models may also be categorized by the type of explanatory variables used. That is, the level of maintenance that a pavement receives may be expressed as a function of any one of the following sets of variables:

- Condition of the pavement at a previous year [Al-Mansour and Sinha, 1994],
- Change in condition of the pavement up to the previous year,
- Factors that influence pavement condition levels or change thereof, or

- A combination of pavement condition (or change thereof) and factors that influence pavement condition levels or change thereof [Li and Sinha, 2000]. This may render the maintenance expenditure model prone to endogeneity bias.

A 1984 study carried out in the state of Indiana estimated average pavement maintenance cost values for a variety of pavement categories [Sharaf et al., 1984]. No equations were developed, but a table of maintenance expenditure for each type of pavement, each road functional class (Interstate/non-Interstate) and geographical region (North/South) was established. In that study, the nuances of the meanings of the terms “cost” and “expenditure” were not specifically addressed, and as a consequence, the word “costs” was used where “expenditure” might have been more appropriate. The study appropriately identified that maintenance cost consists of a fixed cost of the treatment itself and a variable cost that was comprised of surface treatment cost prior to the main treatment. It is obvious that the fixed cost relates to attributes of the maintenance treatment (unit prices of labor, aggregate, etc.), while the variable cost relates to the attributes of the pavement (surface condition, etc.). The present study makes an attempt to provide unequivocal descriptions of these two types of costs: unit “costs” for each maintenance treatment are determined for each treatment type, regardless, as much as possible, of other externalities, while unit “expenditures” are established for each pavement category as a function of age and other pavement attributes. It is worthy to note that maintenance treatment costs vary only with prices of labor, equipment and material, unless the units of the treatment cost inadvertently reflects pavement condition. For example, crack sealing is measured in lane-miles, so the more the unit costs of crack sealing depend on the condition of the pavement, unlike the case of shallow patching which is measured in tons of material used. It is also significant to note that pavement expenditure in a given year may consist of the costs of none, one or several maintenance treatments.

With implicit assumption that increased maintenance expenditure compensates for increased pavement damage due to traffic loading and weather, average annual maintenance expenditure models using 1995-1997 Indiana data were developed as part of a cost allocation study [Li and Sinha, 2000]. Using

the averaging approach, that study developed models to estimate the level of annual routine maintenance, for each major pavement type, as a function of change in previous year's roughness and functional class.

For purposes of life cycle costing, routine maintenance was modeled as a linear function of maintenance policy as follows [Markow, 1994]:

$$C(t) = [c * L(t) * A * R(t)]/5$$

Where $C(t)$ = pavement routine maintenance cost in year t

c = unit cost of maintenance activity, in constant dollar, per lane-mile per year corresponding to the level of maintenance $L(t)$

$L(t)$ = relative level of maintenance expenditure performed in year t

$R(t)$ = ratio of actual adjustment in deterioration curve due to routine maintenance and total theoretical adjustment in year t , found as follows:

$$R(t) = \Delta P_t / [E * L(t) / 10]$$

Where ΔP_t = adjustment in PCI due to maintenance in year t

E = effectiveness of routine maintenance at current value of PCI, P_t

In that study, maintenance expenditure was examined more in the context of individual treatment cost than of overall maintenance expenditure for a given pavement. The present study addresses the issues of maintenance treatment costs and annual pavement maintenance expenditure by treating these concepts in a very separate manner.

As part of a study that sought to assess marginal maintenance costs due to traffic increments, a maintenance cost function was developed [Small et al., 1990]. The study yielded annualized routine maintenance costs as a function of annual traffic, pavement width, and pavement thickness. The marginal annual maintenance cost ($MAMC$), was then found by partially differentiating the annualized maintenance costs function with respect to annual traffic, as follows:

$$MAMC = r * \frac{\partial M}{\partial Q} = r * \frac{\partial M}{\partial T} * \frac{dT}{dQ} = - \left\{ \frac{r^2 e^{rT} C(w)}{(e^{rT} - 1)^2} \right\} * \left(\frac{dT}{dQ} \right)$$

Where $MAMC$ = marginal annual cost of maintenance

r = discount rate

T = overlay interval

$C(w)$ = cost of last overlay

dT/dQ = rate of change of overlay interval with respect to annual traffic loading.

A study similar in purpose to the Small et al. study assessed the additional pavement infrastructure expenditure due to increased axle mass limits in Australia [Martin, 2000]. The researcher presented an approach to estimate increments in maintenance expenditure needed to counter the potential loss of pavement capacity due to load and weather effects. Additional levels of pavement maintenance were estimated as a function of pavement loading and weather effects. However, because of limitations that include lack of empirical quantification of the influence of maintenance in reducing pavement deterioration, the researcher stated that the approach is not recommended for practical use at that time.

In Indiana, annual routine maintenance expenditure models were developed as a function of pavement condition, for flexible pavements [Al-Mansour and Sinha, 1994]. The models, which were developed as part of an overall routine maintenance study, were of the following form:

$$\text{Log } AME = A - B * PSI$$

Where AME = annual maintenance expenditure in dollars

PSI = pavement condition (present serviceability index) in year before maintenance

A and B are constants whose values depend on the class of road (high volume vs. low volume)

Even though regional effects were considered in the overall project, such effects were not considered in the annual maintenance expenditure model shown. Therefore, it is possible that variations in flexible pavement expenditure due to regional difference (subgrade and climate) were missed during the modeling process. Also, these models were developed using only basic routine maintenance data, and therefore excluded all works done on contract, which can be quite substantial, especially in recent years where policy changes have resulted in significant amounts of pavement maintenance work being let out on

contract. Because the models in that study utilized the pavement condition during the previous year, rather than change in condition from one year to the next, only one data point of condition data was needed. Therefore data requirements were kept low.

3.4.2 Short-Term Maintenance Effectiveness Modeling

A literature review on existing short-term maintenance effectiveness showed that such models are indeed useful because they provide an insight into the immediate benefits of maintenance in general, and the effectiveness of individual maintenance treatments in particular. The efficacy (or lack thereof) of individual treatments in slowing down deterioration may be masked if effectiveness is evaluated only in the long-term. This is particularly important if several maintenance treatments are applied over a long-term period, making it difficult to isolate the impact of individual treatments. A few past studies have developed short-term effectiveness models to determine the incremental change in pavement condition in response to past maintenance in a general sense [Ramaswamy and Ben-Akiva, 1991], or a maintenance treatment in particular [Mouaket and Sinha, 1991]. Some of such models have been useful in the development of zero-maintenance curves [Fwa and Sinha, 1987].

The concept of DRL (deterioration reduction level) which is the decrease in deterioration from one year to the next, has been used to determine the change in roughness over a 1-year period in response to various types of routine maintenance treatments [Fwa et al., 1987]. These researchers developed models that predict the change in pavement condition (PSI) as a function of maintenance and other pavement attributes. Also, a routine maintenance study in Indiana [Sinha et al., 1988] expressed maintenance effectiveness as the change in pavement roughness, RRN, as follows:

$$RRN = a + b \cdot \log_{10} RM + c \cdot R + d \cdot (\log_{10} RM \cdot R)$$

where $R = 0$ for Northern pavements, and 1 for Southern,

RM = unit routine maintenance expenditure.

The response variable for maintenance effectiveness was computed as

$$RRN = (RN_{85} - RN_{84}) / RN_{84}$$

where RN = is the roughness of a pavement section in a given year, in counts per mile.

In that study, investigation of maintenance effectiveness over a 1-year period led to the conclusion that “for most treatments, roughness increases after treatment, regardless of maintenance expenditure level”. That finding was obviously due to non-consideration of the relative timing between maintenance application and the condition surveys. The study found that maintenance effectiveness is lower in the north compared to the south, attributing this finding to the extended cold period in the north.

The change in roughness number, also a DRL concept (see Chapter 5), has been used as a response variable in models that estimated the effectiveness of general maintenance and rehabilitation [Madanat and Mishilani, 1995]. Using recent data from Indiana, DRL models that predict change in IRI as a function of pavement attributes have been developed [Li and Sinha, 2000]. That study made mention of the effectiveness-expenditure simultaneous relationship and made attempts to address this issue by utilizing a 2-stage model. The concept of performance jump has been used to develop equations that estimate the instantaneous reduction in roughness due to overlays of varying thickness, which include thin overlays [Colluci-Rios et al., 1984]. The literature review did not reveal any past studies that investigated maintenance effectiveness from the standpoint of deterioration reduction rate (DRR), which is the reduction on the slope of the deterioration curve due to maintenance (Chapter 5). However, this concept has often been mentioned in literature [Darter, 1980; Lytton, 1987].

It is seen from the above literature review that relatively few studies have been carried out to investigate maintenance effectiveness, and where this has been done, deterioration reduction level (DRL), and to a lesser extent, performance jump (PJ) have been used as the measure of effectiveness. Also, it is obvious that previous studies did not implicitly consider the relative timing between maintenance occurrence and the time of deterioration measurement, an oversight that could be costly in estimating maintenance effectiveness (as demonstrated in Chapter 7). Also, past studies did not provide a relation between the various measures of deterioration (DRL, DRR, and PJ). Therefore an agency that might be interested in a particular measure has been unable to convert the available measure into the measure of interest. The present study (Chapter 5) addresses these issues.

3.5 Past Studies on Long-Term Maintenance Effectiveness Evaluation

A review of available literature on long-term maintenance effectiveness covered two main areas that are vital for such evaluation:

- Pavement performance modeling
- Methods used in effectiveness evaluation

Also the literature review covered results of past studies on maintenance effectiveness. For each of these areas, relevant sections of various literature reviewed are briefly presented below.

3.5.1 Pavement Performance Models

Pavement performance models are essential elements in long-term maintenance effectiveness evaluation because they provide a means by which the benefits of maintenance can be “measured”, i.e., the incremental area under the performance curve. Pavement performance modeling has often been described as the most essential part of any pavement management system [Darter, 1980], and allows highway agencies such as INDOT, to predict pavement condition/performance based on past trends, to determine optimal times to carry out preventive maintenance or rehabilitation, to predict the impact of M&R actions on pavement condition, and to determine pavement remaining service life.

Against this background, it is important that pavement performance model should, as much as possible, reflect actual trends. Poorly designed models and mistakes in prediction can lead to inappropriate cost-allocation policies and costly mistakes in the selection of M&R type and timings. Such considerations have led to the establishment of certain criteria for effective performance models [Darter, 1980; Lytton, 1987]. These are (i) an adequate PMS database (condition, materials, loading, environmental, design, etc.), (ii) selection of an appropriate functional model to represent the real-world situation, (iii) consideration of all significant variables that affect deterioration, and (iv) criteria to assess the precision of the model.

It was necessary to carry out a review of existing literature on pavement performance modeling because it is a key aspect of the present study. The section below discusses pavement performance models developed by highway agencies and research and educational institutions over the past couple of decades.

3.5.1.1 Empirical Pavement Deterioration Models

Pavement performance curves were developed using each of three performance indices selected for the State of North Dakota [Johnson and Cation, 1994]. The three performance indicators were a structural index, a roughness index, and an overall distress index. The equations developed as a result of the study took the form of the following constrained least-squares equation:

$$Y = P_o + P_1 * (age) + P_2 * (age)^2 + P_3 * (age)^3 + P_4 * (age)^4 \dots\dots\dots (1)$$

Where Y = structural index, roughness index, or overall distress index

These models had the drawback of lack of detailed climate, loading and maintenance data. Therefore pavement age was used as the sole independent variable. A similar study used pavement rutting data collected by the Transport Research Laboratory (TRRL) over a 20-year period in the United Kingdom, to develop a regression-based rutting model that took into account material properties, layer thickness, and aggregate types [Kerali et al., 1996]. The results of the analysis indicated that quadratic and cubic model forms, an example of which is shown below, appeared to adequately predict rutting.

$$\text{Rutting} = a_1 * T + a_2 * BT * T + a_3 * BT * T^2 + a_4 * BT * T^3 \dots\dots\dots (2)$$

Where T = traffic loading, BT = base thickness, a_i are model coefficients.

The study also confirmed that material properties, layer thickness, and their combined effects influence rutting, but in ways that vary greatly. The researchers conceded that more scientifically reliable ways of measuring pavement distress and better understanding of the causes of performance variability are needed.

Several curves have been developed to predict pavement deterioration solely as a function of age, either as a polynomial or as a power function. As part of an effort to develop a methodology to quantify the life cycle effect of delaying M&R actions, pavement performance models for various pavements grouped on the basis of pavement structure and traffic use were developed [Sharaf et al., 1988]. A large number of models were tested and the best model obtained was of the following form:

$$C = 100 - bx^m \dots\dots\dots (3)$$

Where

C = pavement condition expressed in terms of PCI, B = slope coefficient

X = pavement age in months, m = a parameter for the degree of curvature of the curve

The best fit was determined by the highest R^2 value (coefficient of determination), using the least squares method. Curves of similar functional form utilizing age as the sole explanatory variable have been developed by several researchers [Jackson et al., 1996; Pierce and Mahoney, 1996; Chan et al., 1997].

Other studies that have largely used age as the sole or one of the very few independent variables include an Illinois study that sought to determine the life span of the pavement using the initial pavement roughness and age only [Smith et al., 1996]. That study suggested that rougher pavements increase the dynamic loading effects of truck traffic on the pavement and argued that pavement roughness at the time of construction or rehabilitation greatly influences roughness at any future time, and ultimately determines the life span of the pavement. Data from over 200 pavement projects from 10 states were analyzed. Pavement sections adjacent to those under study were used as control sections. Therefore the effect of traffic, loadings, age, design features and other variables on the performance were constant for both experimental and control sections, and this enabled the effect of initial roughness to be isolated. The study found that a 50% increase in smoothness from specified target levels increased pavement life by at least 15% in many cases. Observations of time-series performance data showed that the following multiple non-linear regression model of the exponential form was appropriate for most sections:

$$S_t = a_0 + a_1 S_i^{b_1} + a_2 t^{b_2} + a_3 S_i^{b_3} t^{b_4} \dots \dots \dots (4)$$

Where

S_t = pavement smoothness at time t , a_0, a_1, a_2, a_3 are regression coefficients

b_1, b_2, b_3, b_4 = exponential coefficients for initial smoothness, time, and initial smoothness-time interaction variables, S_i = initial pavement smoothness, and

t = age of the pavement (number of years since construction or overlay to time of smoothness S_t).

Although long-term smoothness was related to initial smoothness for many of the projects studied, many extenuating factors could mask this relation, as was observed for some of the sections studied.

An aggregate damage model for highway pavement performance analysis in Indiana resulted in the introduction of the concept of PSI-ESAL loss as an indicator of pavement deterioration and loss of serviceability [Fwa and Sinha, 1986]. In contrast to the traditional PSI-Age parameter, PSI-ESAL offers a more representative and quantitative measure of historical performance. In that study, the concept of zero-

maintenance was also introduced as a reference level for quantifying the impacts of various routine maintenance effort levels. The concepts of zero maintenance and PSI–ESAL loss in the evaluation of long-term maintenance effectiveness were considered in the present study.

The development of distress prediction models for rigid non-overlaid PCC pavements for Texas DOT's Pavement Management Information System also made use of age as a very influential variable [Robinson et al., 1996]. A sigmoidal regression equation was used for all distress types considered. The regression models predict distress level versus pavement age, but for CRCP pavements, modifying factors which were intended to capture the effects of structural, environmental, and traffic loading variables, were included. According to the researchers, the shape and modifying coefficients described in the above equation are used to modify the general equation to fit a specific pavement section.

Livneh, [1996] introduced a universal pavement deterioration model that predicts performance as a function of age for unalid pavements, and as a function of age, traffic and structural number for overlaid pavements. The general model for unalid (no overlay) pavements was of the form:

$$OPI = 100 [1 - a(A/A_0)^r - 3(1 - a) (A/A_0)^{2r} + 2(1 - a)(A/A_0)^{3r}] \dots\dots\dots(5)$$

Where

OPI = pavement overall condition index

A = pavement age in years

r and A_0 are functions evaluated from observed data.

The general model for overlaid pavements was:

$$OPI = 100 [1 - F*a(A/A_0)^r - F*3(1 - a) (A/A_0)^{2r} + F*2(1 - a)(A/A_0)^{3r}] \dots\dots\dots (6)$$

Where

$$F = (SN_0/SN_F)^{1.872} * (1 + i)^{0.338At}$$

SN_0 = original structural number of the pavement

SN_F = target structural number upon rehabilitation

i = yearly geometric growth rate of ESALs

A_t = age of pavement at time t.

According to the researcher, this new deterioration model encompasses all of the possible shapes of the deterioration curve, including (a) sigmoidal mode with a slow rate in the early life of the pavement,

(b) sigmoidal mode with a rapid rate in the early life of the pavement, (c) the regular convex-up mode, (d) the regular convex-down mode. Another interesting feature of this “universal” model is that it can be reduced to some other well-known models such as the Washington State PMS model (where $a = 1.0$), and the old linear model (where both a and r are $= 1$). Sufficient data are required for development of this model. If data are insufficient, different models could be obtained from a given data set, and the results could be misleading.

3.5.1.2 Mechanistic Pavement Deterioration Models

A study for the US Army Cold Regions Research and Engineering Laboratory (CRREL) used a series of computer programs to develop flexible pavement deterioration models in areas susceptible to freeze-thaw cycles [Allen et al., 1991]. The study was partly based on the premise that seasonal variation in pavement strength results from the seasonal temperature variations because (a) base and subgrade strength increases when frozen, (b) base and subgrade lose strength upon thawing in spring, and (c) asphaltic concrete strength and modulus changes with temperature. The model used a series of programs that compute frost heave and thaw settlement of the pavement structure and soil conditions at a given time increment. The programs also reduced the pavement structure to a layered system with distinct material properties, calculated stress, strain and deflection at given points in the pavement profile, and computed the incremental and cumulative damage to the pavement structure. An important result from the study was a mechanistic explanation for the widely observed phenomenon of significant failures at spring periods, indicating that the thaw period is crucial in the life of a pavement. However, other deterioration factors such as vehicle loads were not considered. These models are therefore probably most appropriate for road types and geographical areas where environmental effects (particularly freeze-thaw) account for a far greater portion of pavement damage compared to load effects.

In response to the ever-increasing diversity of heavy truck design and use (axle configuration, suspension, tire type, inflation pressure), and their consequences on pavement loading, NCHRP 363 reported on the use of computer-based methods to assess the influence of major vehicle and pavement variables that affect road damage [Gillespie et al., 1993]. The study related the characteristics and properties of trucks to pavement damage, identified the most critical truck properties, and provided insights

into the mechanism of damage to aid in pavement management. Fatigue-induced cracking and pavement deformation (rutting) were used as the primary indicators of pavement damage.

Most of the findings reinforced existing understanding of pavement behavior in response to truck loads, but the study also provided a systematic overview of the interactions between pavement, environment, truck loads and truck type characteristics. The study made the following observations, among others: axle spacing is generally not an important truck characteristic affecting pavement damage, however speed is an important factor that influences pavement damage (higher speeds cause more damage due to vehicle dynamics in response to road surface irregularities.) A favorable effect of higher speed, is the reduction of the time duration of wheel load at a given point (and consequently, reduced fatigue damage). However, the authors state that this benefit of high speeds is unique to visco-elastic materials such as asphaltic concrete. It was found that road surface roughness excites truck dynamic axle loads, thus increases fatigue damage: rough pavements (2.5 PSI) were found to experience 1.5–3 times more rate of damage than smooth pavements (over 4.5 PSI).

3.5.1.3 Empirical-Mechanistic Pavement Deterioration Models

After reviewing various types of prediction models, a study concluded that empirical-mechanistic models best explains flexible pavement performance [George et al., 1991]. Pavements with AC surface were grouped into three categories: AC Pavements with AC overlay, AC pavements with no overlay, and AC-on-PCC composite pavements. Prediction equations were developed for each of these categories. Pavement Condition Rating (PCR) was used as the measure of pavement performance. The equations were validated by comparing them with several existing empirical and mechanistic models. An interesting result obtained was that age was, by far, the most significant predictor of serviceability. According to the study results, traffic volume and weight expressed in terms of ESALs, and the structural make-up of the pavement (described by the composite structural number) play only a secondary role in forecasting pavement performance.

Using pavement distress functions in the Highway Design and Maintenance Standards (HDM) model, two generalized equations were developed to predict roughness progression in flexible pavements [Paterson and Attoh-Okine, 1993]. The first model was described as having a close fit to the original

incremental model, and predicts roughness using factors such as age, traffic, pavement strength, environment and distress occurrences such as rutting, patching and cracking at a specified time. The second model is simpler as it omits surface distress parameters and compensates for this through the primary structural, traffic, age and environmental factors. The authors stated that the search for a mechanistic pavement performance model has been elusive because of the complex interaction between the various causes of deterioration and between maintenance and deterioration. However, they averred that the model used in their study (the Road Deterioration and Maintenance sub-model of the World Bank's HDM-III) comes close to this goal as it quantifies these interactions and predicts all modes of distress and maintenance impacts. The existing HDM-III sub-model quantifies all the primary effects, including the concurrent effect of traffic and aging through an incremental recursive approach, calculating the change in each mode of distress sequentially for each year of the analysis period. According to the researchers, that simulation approach requires a substantial amount of computing capacity and speed. The objective of that study therefore was to develop simpler and more efficient algorithms that approximate the primary effects captured by the full recursive models and permit rapid prediction of roughness from a small number of primary parameters. Also, the summary model was designed to predict absolute rather than incremental pavement roughness. The study found that for applications where data or predictions of rutting, cracking, and patching are available, the recommended model is as follows:

$$RI_t = 0.98 e^{mt} [RI_0 + 135SNCK_4^{-5} NE_t + 143 RDS_t + 0.0068CRX_t + 0.056 PAT_t \dots \dots \dots] \quad (7)$$

Where

$$SNCK_4 = 1 + SNC - 0.00004 HS CRX \quad \text{for } HS CRX < 10,000.$$

t = pavement age since last construction or rehabilitation

RI_t = roughness at pavement age t , RI_0 = initial pavement roughness

NE_t = cumulative ESALs at age t , SNC = structural number

HS = thickness of bound layers, CRX_t = cracking index at time t

RDS_t = standard deviation of rut depths at time t , PHV = pothole volume/lane km

PAT_t = area of patching (S), m (average values) = 0.010 for dry non-freeze areas
 = 0.020 for dry freeze areas
 = 0.023 for wet non-freeze areas
 = 0.070 for wet freeze areas.

A more general model that does not incorporate the effect of surface distress was developed as follows:

$$RI_t = 1.04 e^{mt} [RI_0 + 263(1 + SNC)^{-5} NE_t], \dots\dots\dots (8)$$

Where symbol meanings are as shown above.

Another comprehensive effort to develop an empirical-mechanistic pavement deterioration model was undertaken under the auspices of AASHTO [Daleiden et al., 1994]. Studies were conducted to evaluate the impact of numerous pavement properties on the prediction of typical distresses. For flexible pavements, the models were generally of the form:

$$\text{Distress} = N^B 10^C \dots\dots\dots (10)$$

Where

N = the number of cumulative ESALs in 1000's

$B = b_0 + b_1X_1 + b_2X_2 + \dots b_NX_N$

$C = c_0 + c_1X_1 + c_2X_2 + \dots c_NX_N$.

For example, using data from 35 GPS sites with HMAC on granular bases in the wet-freeze regions, change in IRI is predicted as follows:

$$\Delta IRI = N^B 10^C$$

Where

N = the number of cumulative ESALs in 1000's

$B = 0.25$

$C = 0.0403 + (0.00014 * \text{asphalt viscosity}) + (0.0704 * \text{HMAC air voids})$
 $+ (0.314 * \log \text{HMAC thickness}) - (0.00162 * \text{base thickness})$
 $- (0.00165 * \text{annual days} > 90 \text{deg F}) + (1.628 * 10^{-5} * \text{freeze index} * \text{HMAC air voids}).$

Predictive equations for the various pavement and environmental configurations were selected after hundreds of trials, and the best models were selected on the basis of collinearity-minimization. The researchers argued against lumping of all distresses into an overall combined performance index for purposes of distress prediction, as that would mask the contribution of each specific layer to a specified level of pavement performance at any given time. The researchers also contended that it is not possible to develop a single effective model to predict distress across a broad range of environmental conditions such as that of the United States.

3.5.1.4 Performance Models that Included Maintenance Effects

Relatively very few studies on pavement deterioration prediction have utilized the occurrence of pavement maintenance, implicitly or explicitly, as an explanatory variable. Performance data from the State of Nevada were used to develop nine flexible pavement performance models that relate PSI to material properties, traffic loading and environmental conditions for each selected maintenance technique [Sebaaly et al., 1995]. This research demonstrated that the type of maintenance treatment a flexible pavement receives influences pavement performance over time. Models were built for each maintenance technique and for each district. In all models, the age variable featured prominently as a significant variable.

A similar study carried out in Indiana utilized maintenance occurrence as an explanatory variable that took on discrete values of 1 or 0 depending on whether maintenance was carried out [Mohammad et al., 1997]. Another interesting aspect of that study was that restrictiveness of assumptions made in previous modeling efforts (that past maintenance has a unilateral and exogenous effect on pavement performance) was identified and duly addressed. In other words, past performance response models that included maintenance occurrence as an explanatory variable did so without recognizing that the maintenance variable was, in turn, a response variable in another model that predicts or estimates maintenance occurrence and uses performance change as an independent variable. A continuous model was developed to predict performance change in response to maintenance, traffic, age and other variables, and then the study proceeded further to a second stage to develop a model to predict maintenance occurrence (in the form of a decision) as a function of performance, traffic, age and other variables, while making the necessary correction for simultaneity. The study made use of data from 126 randomly chosen Interstate pavement sections in Indiana. Both the single-equation and the two-stage procedures were carried out, and it was revealed that the single-equation estimation method did not yield an acceptable model for pavement performance prediction as some of the signs of some critical variables were found to be counter-intuitive. On the other hand, the two-stage approach yielded results that were not only intuitive, but also offered a closer fit with observed data. The models obtained, which explicitly considers the interaction of performance and maintenance as shown in Equations (11) and (12):

Performance Model:

$$RN_{t,n} = 6.63 - 0.85 * M_{t,n} + 0.14 * AGE_{t,n} + 0.001 * ESAL_{t,n} - 0.16 * THICK_{t,n} \quad \dots \quad (11)$$

Maintenance Decision Model:

$$\log_e [P(M_{t,n} = 1 | RN_{t,n}) / P(M_{t,n} = 0 | RN_{t,n})] = -4.68 + 1.08 * RN_{t,n} + 0.003 * ESAL_{t,n} - 0.21 * THICK_{t,n} - 0.9 * 4RG_{t,n} \quad \dots \quad (12)$$

Where

$M = 1$ for pavement section with past maintenance, 0 otherwise

$P(M_{t,n} = 1)$ = probability of pavement section having maintenance at time t

RN = log of roughness number, n = number of observations, t = year of study

AGE = number of years since last rehabilitation

$ESAL$ = equivalent single-axle load repetitions

$THICK$ = pavement thickness in inches

RG = dummy variable: 0 for pavements in northern Indiana, 1 for southern Indiana.

3.6 Methods Used to Evaluate Cost-effectiveness

Cost-effectiveness evaluation is an economic evaluation technique for comparing that which is sacrificed (cost) to that which is gained (effectiveness) for the purpose of evaluating alternatives. It generally includes those procedures and concepts that involve comparing input costs to outcomes, whether such outcomes are priced or not. Cost-effectiveness can be measured in the short-term (i.e., for one or more treatments administered at a given time), or in the long-term (i.e., for several treatments carried out over an extended period of time, such as the service life of the pavement). Cost-effectiveness evaluation may be considered more appropriate for long-term studies, and not for the short-term: because of the typical multiplicity of alternatives in the long-term (each alternative having different costs and benefits). In the short-term, however, cost-effectiveness may be appropriate in only a few cases, e.g., where it is sought to compare two alternative treatments to address a given pavement distress, such as crack sealing with traditional sealant or with crumb rubber.

Outcomes of each strategy could be benefits, returns, satisfaction, or progress towards stated objectives. Some cost-effectiveness analyses proceed on the basis that, although the cost can be presented in dollars, the effectiveness of these costs in producing desirable goals and results can be described only in qualitative terms because not all the benefits and adverse consequences can be presented on a dollar basis [Mouaket and Sinha, 1991]. The cost-effectiveness of a maintenance treatment depends on the following [Chong, 1991]:

- How the treatment changes the existing condition: i.e., how effectively it corrects existing distress,
- How well the treatment effectively delays the distress deterioration process, thereby extending pavement life,
- Whether there is a particular condition or time during the progression of the cracking distress when appropriate maintenance can be most effective.

The first consideration is suggestive of short-term effectiveness, where it is sought to determine the level of reduction of deterioration or increase in condition either instantaneously or after a 1-year gap, and is generally considered more appropriate for maintenance effectiveness evaluation in the short-term. The second consideration is in line with long-term effectiveness, where it is sought to determine the extension in service life due to a treatment. Because there are several treatments a pavement can have over its life, it is difficult to isolate the extension in service life offered by any one treatment, even though pavement managers in Indiana, in providing their perceptions on pavement maintenance effectiveness, have made earnest attempts to do so through the questionnaire survey (see Chapter 4). Rather, the second consideration is more appropriate if thought in terms in maintenance strategy (a series of treatments spaced out over a period of time) rather than just treatments. It is useful to note that pavement managers in Indiana indicated both short-term effectiveness (change in condition) and long-term effectiveness (extension in service life) of their maintenance practices. In line with the philosophy of this approach, Chong [1991] argued that information needed to establish cost-effectiveness must quantify the effectiveness of treatment, extension of pavement service life, and the influence of treatment time.

From an economist's viewpoint, effectiveness evaluation could be carried out in two ways: the first approach is based on seeking the maximum benefits for a given level of investment (the maximum benefit approach); the second approach seeks the least cost for effective treatment of problems (least cost approach). The first approach is often used in capital investment decisions while the second is considered more appropriate for evaluation of maintenance investments.

3.6.1 Maximum Benefit Approach

This approach is often used for evaluation of capital investment projects as such activities typically involve a single large investment that is associated with significant elements of uncertainty and where the cost of each alternative is the same. Consequently, the assessment of exact benefits is very difficult. Furthermore, the measures of effectiveness for such projects are often difficult to identify and complex to define due to the long duration of such activities and spillover effects [Mouaket and Sinha, 1991]. Over the past two decades, much research has been carried out to define measures for evaluating benefits of capital improvements and the idea has been further extended for some maintenance activities.

These benefits include reduced travel times, reduced tort liability, reduced vehicle operating and maintenance costs, increased motorist comfort and safety, reduced rate of pavement deterioration, and reduced or deferred capital expenditures through preservation of capital [Geoffroy, 1996].

In the context of pavement management, most of the research efforts utilize the performance curve concept. All the fore-mentioned benefits could be represented by the area under the performance time curve. The rationale for this approach is simple: a consistently well maintained pavement (a gently sloping performance curve, yielding a large area under that curve) provides the user greater benefits than a bad pavement (a steep performance curve having a small underlying area). Because the benefits of a well-maintained pavement are numerous and difficult to quantify in monetary terms, the area under the performance curve could be used as a surrogate for user benefits. Another way of measuring benefit is to estimate the extended remaining service life by carrying out that improvement, i.e., time taken for the pavement to deteriorate to a certain threshold level.

3.6.2 Least Life Cycle Cost Approach

Maintenance investments are often smaller in value and take a relatively short period for completion compared to capital improvements. Also, their impacts are experienced immediately after completion. In the short-term evaluation of corrective maintenance “investments” the least cost approach is considered most appropriate, as all the alternatives are considered to provide the same benefit. For example, faced with occurrence of severe cracking on a localized section of road, a field engineer considers the possible options (all of which have the same “benefit” of reinstating that section to the original condition), such as crack filling and partial depth patching. He then selects the most cost-effective alternative as that which has the least cost. This methodology assumes that all the corrective maintenance strategies being compared provide the same level of service, and that the preferred option is one that minimizes life cycle costs. However, for the evaluation of long-term maintenance effectiveness, both benefits and costs have to be considered.

3.6.3 Combination of Cost and Benefit Approaches

The evaluation of corrective maintenance requires the Least Cost approach, and for evaluating rehabilitation activities, the Maximum Benefit approach is used. However, evaluating preventive maintenance is not so straightforward a task. The nature of preventive maintenance activities and the objectives they are intended to achieve place such activities somewhere in between corrective maintenance and rehabilitation. Through preventive maintenance, minor defects are corrected. But then, the performance of the road is somewhat renewed, providing the road with an upward jump in performance albeit of a magnitude less than that for rehabilitation. For this reason, it is more appropriate to use both approaches for evaluating preventive maintenance activities. NCHRP Synthesis 223 [Geoffroy, 1996] suggests that both benefits accrued to the users and the cost incurred to provide those benefits, be considered. That study states that when the benefits and costs can be quantified in monetary terms, a benefit-cost analysis can be made.

Life cycle cost and benefit analysis, which requires the conversion of all factors into economically measurable units, is one of the most powerful tools available for measuring effectiveness of various maintenance activities [Peterson, 1989]. For the purposes of this study, a life cycle is defined as the period between one rehabilitation activity and the next. To perform life cycle costs analysis for this study, it was essential to identify the various agency and user cost components and to predict the amount of such costs. Cost models developed in previous studies were used to generate unit costs for life cycle cost analysis.

Life cycle cost and benefit analysis in maintenance management has been used in one of two ways: first, as the least present-worth of the life cycle cost and benefit [Chong and Phang, 1988], and second, as the least annualized life cycle return, calculated in perpetuity [Sharaf et al., 1988]. A basic life cycle cost and benefit analysis procedure was used to determine the cost-effectiveness of network level maintenance and rehabilitation treatments [Darter et al., 1987]. The selected strategy was one that yielded the least equivalent annual cost per unit area of pavement. Also, life cycle costing was used to quantify the effect of deferring maintenance and rehabilitation of pavements based on data obtained from U.S. military installations [Sharaf et al., 1988]. Another application of life cycle costing was in Ontario, where it was used to evaluate the cost-effectiveness of crack sealing [Joseph, 1992]. Other studies in Indiana included

one in which this technique was used to evaluate the cost-effectiveness of chip and sand sealing activities [Mouaket et al., 1991].

In a study that evaluated the effectiveness of each preventive maintenance strategy, effectiveness was measured on the basis of equivalent annual cost of the strategy and the extra service life as a benefit [Hicks et al., 1997]. A decision model was developed that allows users to assign weights not only to material costs and service life benefits, but also to other cost and benefit factors that suit the needs of the decision-maker. For each set of traffic and distress conditions, the alternative with the highest weighted score was selected as the best preventive maintenance treatment under those conditions. Decision trees were developed for various levels of distress types and traffic loading.

In developing budget optimization techniques for PAVER (the U.S. Army Corps of Engineers Pavement Management System), the area under the condition-time curve was used as a measure of performance [Shahin et al., 1985]. Also, Kher et al. [1985] used the area under the performance curve as a surrogate for user benefits, for the Ontario Ministry of Transportation and Communication's Program Analysis of Rehabilitation System. Joseph [1992] used the area under the performance curve combined with the average annual daily traffic (AADT) and road section length to compare the cost-effectiveness of preventive maintenance strategies. With the concept of PSI-ESAL loss (where the performance measure was PSI, and the "time" scale was represented by cumulative loadings applied to the pavement) benefits were represented by the area under the PSI-load curve [Fwa and Sinha, 1987]. The area under performance-time curve concept was used to establish a funding allocation procedure for the San Francisco Bay Area Metropolitan Transportation Commission [Smith et al., 1987]. The New York State Department of Transportation has used the area under the pavement performance curve to compare the cost-effectiveness of alternative preventive maintenance strategies [Geoffroy, 1992]. It is clear that the concept of using the area under the performance curve to represent the benefit of pavement repair is well established within this field.

3.7 Past Studies on Maintenance Effectiveness

Brief descriptions of recent studies that assessed the effectiveness of maintenance, particularly preventive maintenance, are presented below. Features of these studies that are relevant to the present study are also identified.

3.7.1 The SHRP-LTPP Rigid Pavement Maintenance Effectiveness Experiment (SPS-4)

The LTPP and other SHRP-related research programs were started in 1984 with the objective of providing the tools to better understand pavement behavior with a goal of better management of highway infrastructure without major increases in financial resource [Smith et al., 1993, Hadley, 1994; Hanna, 1994]. This effort sought to answer fundamental questions about climatic effects, maintenance practices, long-term load effects, material variations and construction practices by carrying out an intensive long-term study of a large number of actual pavement and field conditions. The Specific Pavement Studies #4 (SPS-4) experiment, which is part of the overall LTPP study, was specifically designed to investigate the effectiveness of the following common preventive maintenance treatments on rigid pavements: undersealing, joint sealing, and crack sealing. It is expected that analysis of pavement performance data obtained from these sites (Figure 3-13) will help quantify the ability of different maintenance treatments to extend service life or reduce distress rates [Hadley, 1994]. This experiment also sought to examine the effects of various environmental regions, subgrade type (fine-grained or course-grained), traffic rate, base type (dense granular or stabilized), and pavement type (plain or reinforced) on preventive maintenance of rigid pavements.

The 500 ft-long test sites used in this experiment were constructed in 1990 and 1991, adjacent to General Pavement Studies (GPS) sites so that traffic and other data collection carried out under the GPS program could be utilized. The SPS experiments consist of individual sites composed of multiple test sections, with each site having similar details and materials according to the various experiment requirements. These sites are distributed among climatic regions as well as subgrade soil types. The configuration of each site is shown in Figure 3-14.

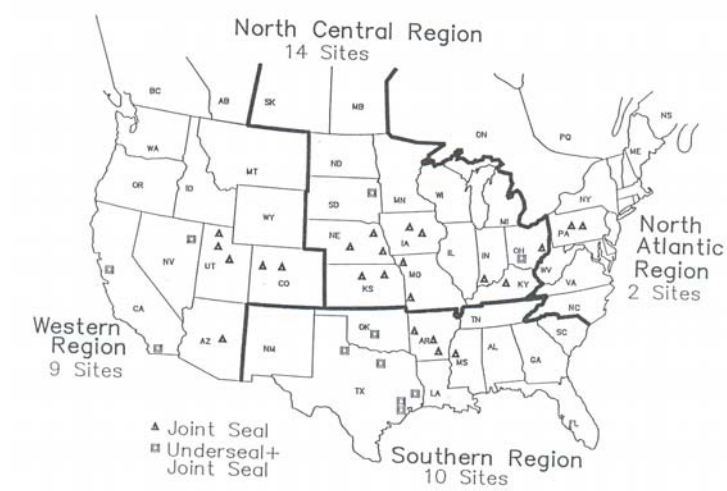


Figure 3-13. Distribution of LTPP SPS Test Sites for Monitoring Effectiveness of Preventive Maintenance Treatments on Rigid Pavements

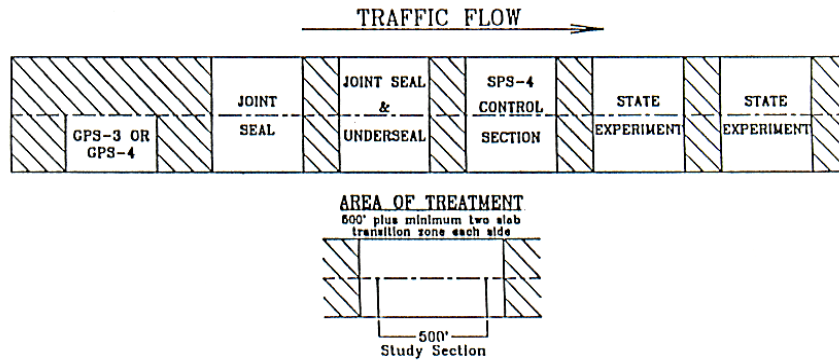


Figure 3-14. Typical Layout for SPS-4 Rigid Pavement Test Sections

Mathematical equations that could be used to evaluate the effectiveness of rigid pavement maintenance treatments were developed in a related SHRP study [Smith et al., 1993].

In 1992-93, and again in 1995, task groups consisting of experts from academia, industry and highway agencies conducted reviews of the SPS-4 sites to evaluate the performance of preventive maintenance treatments on those rigid pavement sections after a 5-year period. Based on the limited number of sites sampled, the findings were as follows [Morian et al., 1998]:

- Unsealed joints contain significantly more debris than sealed joints.
- Unsealed joint sections experienced significantly more spalling than the sealed joints.
- Minor amounts of debris lodged in the sealed joint sections had little or no effect on pavement performance.
- No conclusions were evident regarding the performance of underseal sections. However, after 5 years, those sections were performing consistently well.
- At the Arizona site, all supplemental SPS-4 sections with joint reservoirs of varying widths (3mm to 9mm) were performing well to date.
- At the South Dakota site, it was observed that supplemental SPS-4 sections (which had had the following preventive maintenance treatments: diamond grinding, dowel insertion, and maintained edge drain) had experienced reduced pavement pumping at their transverse joints.

3.7.2 The SHRP-LTPP Flexible Pavement Maintenance Effectiveness Experiment (SPS-3)

This experiment sought to investigate effectiveness of common preventive maintenance treatments on flexible pavements. Constructed in 1990/1991 at locations adjacent to GPS sites, the 500 ft-long SPS-3 test sites each consist of individual sites composed of multiple test sections, with each site having similar details and materials according to the various experiment requirements (Figure 3-15).

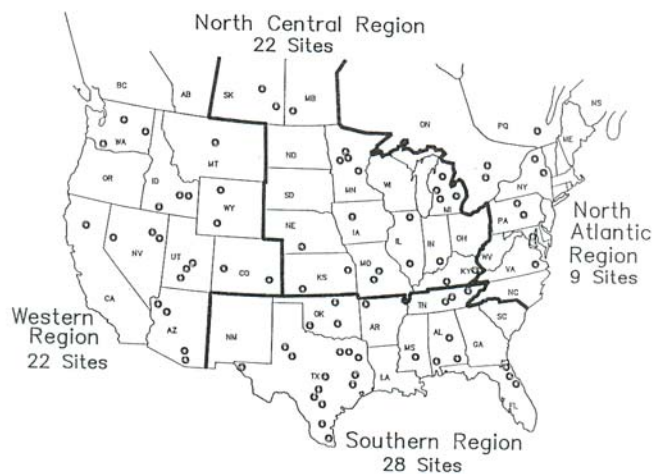


Figure 3-15: Distribution of LTPP SPS-3 Sites for Monitoring Effectiveness of Preventive Maintenance Treatments on Flexible Pavements

These sites are distributed among climatic regions as well as subgrade soil types. At each site, one section (the control section) received no experimental maintenance treatment, while the remaining four sections were treated by chip seal, slurry seal, crack seal, or thin overlay (Figure 3-16). The evaluation of these treatments, which is a long-term effort, also aims at examining the effects of various environmental regions, subgrade type (fine-grained or coarse-grained), traffic rate, ratio of structural capacity, and condition of pavement at time of application, on performance of the selected preventive maintenance treatments.

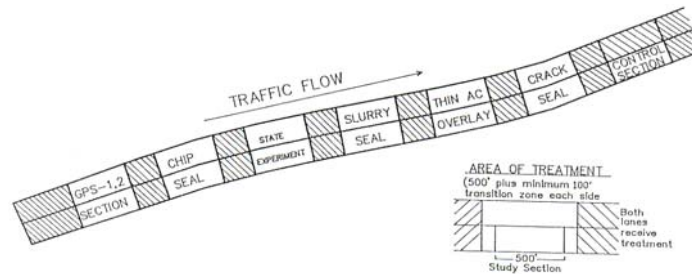


Figure 3-16: Typical Layout for SPS-3 Flexible Pavement Test Sections

Most of the SPS-3 sections were on asphaltic concrete pavements that had a granular base. SPS-3 maintenance treatments were applied to existing pavements that were in good, fair or poor condition. Smith et al. [1993] developed a number of equations that could be used to evaluate pavement maintenance effectiveness based on data generated from the SHRP sites.

A preliminary review of initial data from the SPS-3 study sections was carried out under SHRP Project H-101, which reported the following observations:

- It is more cost-effective to apply pavement preventive maintenance treatments throughout the life of the pavement rather than allow the pavement to deteriorate to a point where major rehabilitation is needed.
- If modest-cost surface treatments are applied at the right time in the decay cycle [service life] can be extended over a much longer time. This way the need for major rehabilitation is delayed, and the extra cost, hazards, and inconvenience associated with work zones due to frequent rehabilitation, are avoided.

- Pavement life would be further extended if maintenance was carried out before the initiation of significant pavement deterioration, rather than waiting until the pavement deterioration has reached an advanced stage.

These observations were very important to the present study, as they helped shape the design of the experiment and formulation of preventive maintenance strategies. Various contacts associated with the SHRP LTPP projects were made for performance reviews and general information relating to the GPS/SPS experiments.

3.7.3 Routine Maintenance and Pavement Characteristics Study

Two separate studies in Indiana investigated the relationship between the level of maintenance and pavement characteristics, and the Cost-effectiveness of maintenance activities, using data from the state highway network in Indiana [Sharaf and Sinha, 1984; Mouaket and Sinha, 1990]. Models were developed to estimate the total annual maintenance costs per lane-mile as a function of age and accumulated traffic for rigid and flexible pavements. Separate models were built to estimate future patching and crack sealing costs. The results of the study revealed that crack sealing costs had a strong relationship with climate and traffic levels. A by-product of the study was a set of average cost matrices for eight corrective and preventive maintenance treatment types detailed by climatic region, highway class, and pavement type. The study provided an insight on the performance of some preventive maintenance treatments in the mid-western part of the country. Various aspects of that study help facilitate the development of cost models for the present study.

3.7.4 The Supplemental Maintenance Effectiveness Research Program (SMERP)

A research effort carried out to closely monitor the effectiveness of selected maintenance treatments typically used in Texas involved asphalt rubber chip seal, polymer-modified emulsion chip seal, latex-modified asphalt chip seal, conventional asphalt chip seal, and a micro-surfacing treatment [Syed et al., 1998]. All treatments were placed on test sections that were 213.3 m long. Both lanes and shoulders were treated. The goal was to establish the cost-effectiveness of these treatments. The data collected were

converted into Pavement Condition Indices, and performance curves were developed. The initial condition, i.e., the pavement surface condition at the time of applying the maintenance treatment, was noted. Some general observations include the following:

- For chip seals, performance of pavements in good initial condition was approximately the same as the performance of pavements in fair initial condition.
- For asphalt rubber modified chip seals, performance of pavements in good initial condition was significantly greater than the performance of pavements in fair initial condition.
- Pavements in good and fair condition at time of treatment application generally out-performed those in poor initial condition.
- Treatments on the fog seal section showed little or no impact. To be effective, fog seal should have been applied on a routine basis.

The researchers of the study implied that a major motivation for the study was the need to investigate the relative cost-effectiveness of each treatment applied to the pavement in fair, good and poor initial conditions. The findings of the study demonstrated that both treatment type and treatment timing (as regards pavement condition at time of treatment) were critical in the effectiveness of maintenance treatment applications. In the present study, strategies were formulated with treatment types and timings as the two major variants.

3.7.5 Comprehensive Study on Preventive Maintenance

A study carried out for the state of New York evaluated two alternative maintenance strategies for managing a mile of newly constructed flexible pavement over a period of 24 years [Geoffroy, 1992]. The first strategy was to fill cracks every fourth year, and apply a thin HMAC overlay every twelfth year. The second alternative was to do no preventive maintenance during the 24-year period, and carry out complete reconstruction at the end of the 24th year (Table 3-5). The cost-effectiveness of each strategy was assessed in terms of the life cycle benefits (measured in terms of condition-years) and life cycle cost. It was

determined that the strategy with preventive maintenance was 3.65 times more cost-effective than that without preventive maintenance.

In a separate aspect of the study, the cost-effectiveness of filling cracks in a thin HMA overlay over a 24-year period was assessed. When the cracks were filled every 4 years, it was observed that the pavement life was extended by four years. The lessons from that study provided motivation for the present research as it cogently demonstrated the overall cost-effectiveness of preventive maintenance.

Table 3-5: Maintenance Strategy for Comprehensive Study on Preventive Maintenance

Year	Preventive Maintenance Strategy		
	Strategy 1		Strategy 2
	Seal Cracks	Thin HMA Overlay	Do Nothing
2			
4	X		
6			
8	X		
10			
12	X	X	
14			
16	X		
18			
20	X		
22			
24	X		Reconstruction

3.7.6 Maintenance Cost-effectiveness Evaluation at Network Level

The Oakland research effort evaluated the cost-effectiveness of maintenance and rehabilitation treatments at network level [Darter et al., 1987]. Life cycle cost analysis was used to determine the most cost-effective treatment, and a decision-tree network level assignment procedure was developed. Five major cost components were included: initial costs, future maintenance costs, salvage values, traffic delay costs due to rehabilitation work zones, and extra user costs incurred (vehicle operation, time, accidents and discomfort). Although the procedure allowed for inclusion of all these costs, user costs and salvage values were specifically excluded in the initial application due to the difficulty of reliably estimating such costs.

The treatments that were considered included rejuvenating seal, slurry seal, single chip seal, double chip seal, and thin asphaltic concrete overlay, and thick overlay. Streets were divided into two groups: Residential and arterial streets. Four different pavement condition levels at time of application were considered. Life expectancies and impacts on service life were estimated using a survey of expert opinions. Some of the major findings in the study included:

- The average long-term annual cost is much higher when the pavement is allowed to deteriorate
- For any given pavement condition (of the four condition types studied) there is a considerable difference in annual average costs for different maintenance strategies.
- The most cost-effective maintenance strategy depends on both the pavement condition at the time of treatment and traffic
- Complete reconstruction and thick overlays appeared to be poor choices, from a general point of view.

The results of that study are relevant to the present study, as they provide evidence to the comparative benefits of alternative preventive maintenance applications, the importance and ways of selecting an optimal type and timing for preventive maintenance for different conditions pavement surface, traffic, etc., and the incorporation of expert opinion in such analysis.

3.7.7 The Pavement Design and Rehabilitation Strategies Study

A FHWA/State of Utah study was carried out to investigate the cost-effectiveness of pavement rehabilitation design strategies [Anderson et al., 1979]. The model framework used in the study had four phases: Phase 1 was a pavement condition and analysis module that considered data pertinent to the various highway sections and identified deficient sections that needed further analysis in the next phase. In Phase 2, appropriate maintenance and rehabilitation strategies were selected for candidate sections identified at Phase 1. Phase 3 calculated the benefits and costs of each strategy for each section and ranked the strategies in relative order. In Phase 4, the strategies were selected on a network basis. The study utilized relationships that tie user cost to PSI and maintenance costs to PSI, by road class. According to the study

report, the model was primarily designed for rehabilitation strategy analysis, but could be modified to handle preventive maintenance practices. That study provided useful hints in the formulation and evaluation of maintenance strategies for the present study.

3.7.8 The Road Improvement and Maintenance (RIM) Study

A study was conducted for the Jamaican Ministry of Public Works to evaluate the effectiveness of four maintenance strategies: basic “routine” maintenance, resealing, overlay, and “rip-up and reseal” [Weatherell and Ebrahim, 1988]. Use was made of data collected all over the world including Jamaica. The results of the study included a set of threshold curves that defined the decision boundaries between the strategies. The curves, which were plotted against traffic and roughness, provided a simple visual display of the decision space for the choice of each strategy (Figure 3-17). The study assumed that all accrued benefits are due to savings in vehicle operating costs and increased agricultural production for the main roads and feeder roads respectively. Interpreting the behavior of pavements and appropriate preventive maintenance practices for pavements under similar environmental regimes, as done in that study, was an important aspect of the present study.

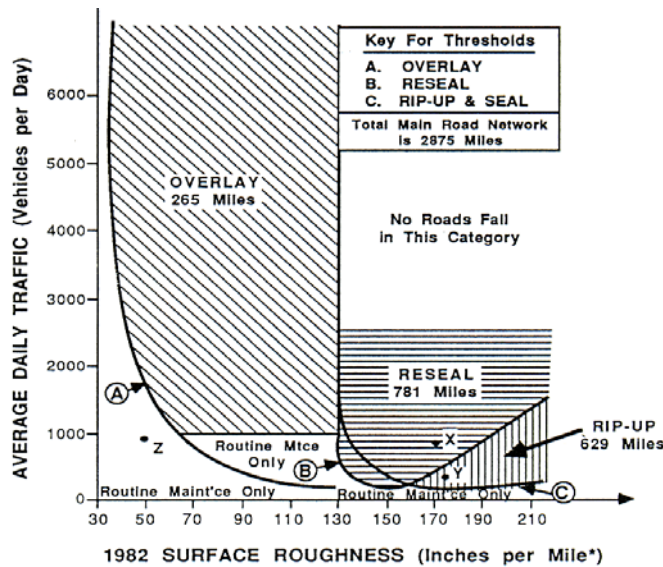


Figure 3-17: Threshold Curves Developed in the RIM Study

3.7.9 Flexible Pavement Preventive Maintenance Study

A study in Canada found that rout-and-seal of transverse cracks could extend the serviceability of a flexible pavement by 4 years [Joseph, 1992]. Three alternatives were considered, as shown in Figure 3-18. The first was a rehabilitation activity (50-mm HMA overlay) every 10-11 years, with no preventive maintenance. The second was rehabilitation every 12-13 years with preventive maintenance (routing and sealing) on the third or fourth year after each rehabilitation activity, and the third alternative was rehabilitation every 13-14 years with preventive maintenance (routing and sealing) on the third and eighth year after each rehabilitation activity. The concept of performance usage (similar to PSI-ESAL Loss concept), rather than performance only, was utilized in the calculation of benefits: the product of the area under the performance curve and the amount of travel (vehicle-distance traveled) yielded the benefit of each strategy. The third option was found to be the most cost-effective strategy. The study demonstrated that maintenance of a preventive nature are indeed cost-effective for flexible pavements in that environmental domain.

Years	3	4	8	11	13	14	17	21	22	25	27	29	30
<u>Activity</u>													
Rout & seal cracks 50mm HMA				X				X					
<u>Strategy 1</u>													
Years	3	4	8	11	13	14	17	21	22	25	27	29	30
<u>Activity</u>													
Rout & seal cracks (50mm HMA		X			X		X			X		X	
<u>Strategy 2</u>													
Years	3	4	8	11	13	14	17	21	22	25	27	29	30
<u>Activity</u>													
Rout & seal cracks 50mm HMA	X		X			X	X		X		X		X
<u>Strategy 3</u>													

Figure 3-18: Preventive Maintenance Strategies [Joseph, 1992]

3.7.10 The Wisconsin Study on Cost-Effectiveness of PCC Joint Sealing

A long-term research project in Wisconsin reports that PCC pavements with unsealed joints performed better than pavements with sealed joints, and that contraction joint sealing costs are not cost-effective [Shober, 1986; Shober, 1997]. This finding is contrary to the observational experience of most pavement and maintenance engineers which indicates that sealing of pavement joints and cracks is beneficial because it reduces the amount of water infiltrating through the crack.

Shober argues that the need to seal PCC pavement joints is so ingrained in the US pavement culture and is so apparently sound from a theoretical perspective that it has been considered an unchallengeable truth. He states that those who have challenged it have been viewed as having conducted poor research. Shober explains that the “truth” of keeping water and incompressibles out of joints may have had a basis when PCC pavements were built directly on the subgrade, but since the advent of base courses the need to seal joints has not been proven.

The Wisconsin observations started with a fortuitous accident in 1953, which suggested the benefits of unsealed joints. In 1958, and again in 1966, test sections were established purposely to study the issue in greater detail. The outcome of these experiments was that pavement with unsealed joints actually exhibited superior performance to those with sealed joints. The Wisconsin researchers now state that the burden of proof now lies with the supporters of sealing and have challenged them to prove their case.

The dichotomy of opinions in this regard suggests that certain specific conditions probably account for the observed difference in effectiveness of PCC joint sealing. Such conditions may include climate, subgrade and base type. For example, sealing a PCC joint may be uneconomical if that pavement is in a dry region, has a granular crushed rock base, an efficient sub-drainage, system, and a gravelly subgrade devoid of clay or silt (and is therefore not susceptible to strength loss or pumping upon wetting). Local construction features and maintenance cultures may also account for the relatively good performance of PCC pavements with unsealed joints in some regions. The Wisconsin conditions generally consisted of a permeable base, short joint spacing, and dowel joints.

3.7.11 To Seal or not to Seal: A Field Experiment to Resolve an Age-old Dispute

In a bid to shed more light on the PCC joint sealing controversy, a field experiment was specifically designed to study the performance of 15 different material-joint configuration combinations [Hawkins et al., 2000]. The pavement consisted of a 250 mm PCC slab overlying a 100mm free-draining base, a 150mm aggregate subbase, and a silty-clay subgrade. The design life was 20 years, with design year AADT of 10,950. Evaluations included profile surveys and visual inspections. The results indicated that sections with unsealed joints, as well as those with joints sealed with preformed compression, were performing satisfactorily. The study also found that sections with narrow joint widths were in worst condition.

3.7.12 Life Expectancy of Routine Maintenance Activities

Life expectancies of various corrective and preventive maintenance activities were estimated through a stratified random sampling survey of maintenance personnel at the sub-district level in Indiana [Feighan et al., 1986]. The study documented estimates of daily accomplishments of the maintenance crews, unit costs of various maintenance types applied to the pavement at different condition levels (good, average, poor), and observed service lives of pavements that had received various types of maintenance treatment applications. The results of that study was useful in the present study because it was shown that pavement condition at the time of treatment affects the level of subsequent impact (effectiveness) of that maintenance treatment.

3.7.13 Study on Pavement Maintenance Effectiveness

Rajagopal and George [1991] employed time-series pavement performance data to develop mechanistic empirical models to predict the immediate jump in pavement condition after treatment and the rate of pavement deterioration after treatment. Pavement condition rating (PCR) an aggregate statistic of both roughness and distress was used as a measure of serviceability. Using these performance jump and performance trend models, the study further evaluated the effect of timing on the effectiveness of various levels of treatment, such as surface treatment, thin overlays, and thick overlays. Life cycle analysis of each

of the three treatments applied at various condition levels indicated that if repairs are performed while the pavement is in the “slow rate” phase of pavement deterioration, the condition after repair is greater and also life cycles are greatly increased, as shown in Figures 3-14 and 3-15.

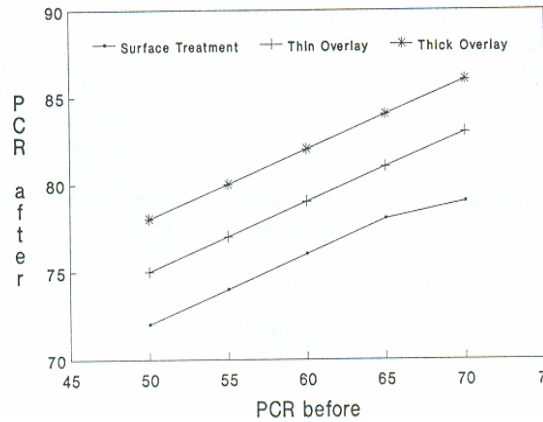


Figure 3-19: Performance Jump after Various M&R Activities

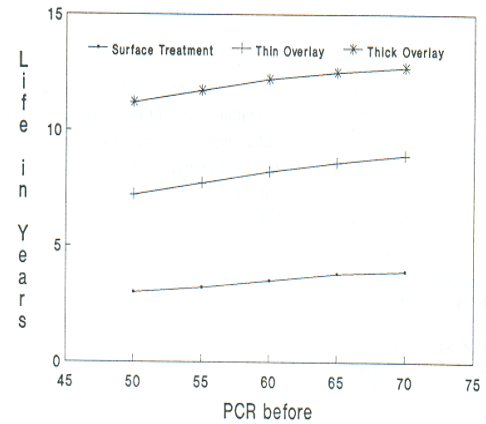


Figure 3-20: Effect of Initial Condition on Service Life

3.7.14 Study to Monitor the Performance of Retrofitted (Restored Load Transfer) Pavements

Hall et al. [1993] reported on the most comprehensive load-transfer restoration experiment in the United States. Fourteen different treatments for load transfer restoration were carried out at various locations on Interstate 10 in Florida, and the performance of each of these treatments was monitored continuously over a six-year period (1986-1992). Performance was measured using results of condition surveys, deflection measurements and faulting surveys. The study found that all retrofitted pavements extended the life of the pavements by over 5 years. However, the study indicated that a longer life could be achieved if the incidence of cracking (where retrofit dowel had been installed) could be addressed. The study also found that joints with retrofit dowels have higher load transfer efficiencies and lower corner deflections than joints with shear devices. However, both devices were about equally effective in controlling faulting.

3.7.15 Study on Sawing and Sealing of Joints in Asphaltic Concrete Overlays

Sawing and sealing, which involves sawing of a joint in the AC overlay directly above the existing joint in the PCC pavement and sealing the joint with joint sealant material, was evaluated as part of a national study [Kilareski and Bionda, 1997]. This treatment is carried out to control reflection cracking. In that study, pavements with up to 10 years of service were evaluated through condition surveys, roughness measurements, and deflection measurements. Both saw-and-seal sections and control sections were evaluated. The analysis indicated that sawing and sealing improves the rideability of the AC overlay and significantly reduces the amount of transverse reflection cracking. On the average, saw-and-seal overlays exhibited about 20% less roughness than control sections. As recent studies have shown that roughness is one of the primary indicators of pavement performance and pavement life, sawing and sealing can be said to not only provide a better level of performance, but also to help extend the life of an AC-on-PCC overlay.

3.7.16 Other Field and Desk Studies on Preventive Maintenance

Available evidence from Puerto Rico suggests that the service lives of jointed concrete pavements can be extended by 10-20 years by retrofitting distressed joints, depending on the condition of the existing pavement and estimated number of heavy trucks using the pavement [Ferragut and Papet, 1994]. In this respect, the FHWA encourages highway agencies to consider such load-transfer restoration as a cost-effective maintenance or rehabilitation technique to extend the service life of concrete pavements in good or fair condition.

The City of Mesa, Arizona has a preventive maintenance strategy that uses the following treatment types and timings [Mamlouk and Zaniewski, 1999]:

- a fog seal 3 and 6 years after construction or rehabilitation,
- a crack seal using a rubber-asphalt seal material every eighth year, and
- a chip or slurry seal every ninth year after construction or rehabilitation activity.

Given the aridness and hotness of Mesa's environment, and high traffic levels, this sequence of treatments was considered most cost-effective for that city's hot-mix asphalt pavements.

3.8 Trade-off Relationships Involving Maintenance

There have been relatively very few studies that have investigated trade-off relationships involving maintenance. In a study that investigated the level of energy savings, rather than service life, from increased preventive maintenance, it was shown that a higher level of spending on crack sealing in the fall season translated to less patching needs after the winter season [Sharaf and Sinha, 1986]. The level of patching effort was measured in terms of the amount of fuel used for that activity after the winter season. The study concluded that there was indeed a trade-off between the amount of sealing (preventive maintenance) done in a past period and the amount of corrective or demand maintenance required at a subsequent period. The present study draws on the realization that preventive maintenance not only affects rehabilitation, but also reduces the need for corrective or demand maintenance, and hence that such impacts could be considered in the overall assessment of long-term maintenance effectiveness.

3.9 Chapter Summary

3.9.1 Typical Distresses and Preventive Maintenance Treatments

On flexible pavements, cracking, pothole development and raveling can be prevented or retarded using crack sealing, patching, and seal coating respectively. Thin hot-mix asphaltic concrete overlay is used to address cases of rutting and cracking before these distresses become severe. On jointed concrete pavements, most typical surface distresses were found to be not only interrelated, but also strongly associated with drainage (or lack thereof) of the underlying pavement layers. Pumping, faulting and corner breaks are symptomatic of poor subdrainage, and are typically prevented by installation of sub-surface drains, or treated using undersealing, grinding, or/and dowel installation. Other jointed concrete pavement distresses such as blow-ups and joint spalling are related to insufficient space for slab expansion, and are therefore addressed by ensuring adequate room for expansion, either by sawing the concrete or by prompt maintenance of existing joints suffering from deposited incompressible matter or damaged sealant. For

continuously-reinforced concrete pavements, preventive maintenance does not appear to play a crucial role as most typical distresses, such as punch-outs, appear to be structural in nature and therefore require substantial levels of corrective maintenance. For asphaltic-concrete composite pavements, reflection cracking, the most prevalent distress, is prevented by sawing and sealing joints in the overlying flexible layer at locations directly above the joints in the underlying rigid pavement.

3.9.2 Pavement Performance/Condition Modeling

3.9.2.1 Regional and Jurisdictional Application

The literature search included a review of performance models that have been developed for pavements in various regions and under various hierarchical jurisdictions of government. Models for performance of pavements in diverse geographical locations, as well as areas close to Indiana, were reviewed. Most of these models were developed for use by pavement management systems of state DOTs, while a few were developed on a regional basis. There were yet others developed with a more parochial intent, and were therefore only applicable to certain parts of a state, or for local city or county jurisdictions.

3.9.2.2 Modeling Details

Most of the performance models reviewed utilized cross-sectional data to predict performance at a given time, while very few utilized time-series data. As regards model shape, the predominant shape of curves obtained, for a given traffic level and other explanatory variables, were the power and sigmoidal curves. A few researchers obtained curves that were quadratic or cubic in nature. Most models were empirical in nature. Only one model used purely mechanistic variables such as measured stresses and strains in the pavement layers. Several models utilized a combination of empirical and mechanistic variables.

Most of the performance models reviewed utilized a response variable that is expressed as a condition index that is an aggregation of the indices or rating of several distress types. Examples of such aggregated indices were Present Serviceability Index (PSI), Pavement Condition Rating (PCR), Overall Pavement Index (OPI), Pavement Structural Condition (PSC), and Distress Maintenance Rating (DMR). A few models used indices associated with a single distress, such as fatigue and rutting. Three models used a

dis-aggregate response variable such as roughness, expressed either as a number (Root-Mean-Square Vertical Acceleration) or an index (International Roughness Index).

With regard to independent variables, the literature review revealed that factors that influence pavement performance have been incorporated in performance models in one of two ways: (i) using a specific factor as an independent variable in the performance model, (ii) using that factor as a criterion to group pavement sections, and then building performance models for each group. Most performance models typically utilized a combination of variables representing stress (traffic, environment, age, etc.) on one hand, and strength (thickness, structural number, CBR, etc) on the other hand. A review of models developed recently indicate an increasing trend towards the inclusion of non-traditional independent variables such as occurrence of maintenance in the preceding year(s), and condition of the pavement at the time of last rehabilitation. Of the models reviewed, the least common variables were asphalt and steel content (for flexible and CRC pavements respectively), previous year's condition, lane status (inner versus outer), and average speed of heavy vehicles. Age was found to be the most dominant variable (Figure 3-21). The preponderance of the use of this variable, as well as its consistent significance whenever it is used, suggests that age alone (or at least with very few other explanatory variables) might be sufficient to explain pavement performance. The literature review showed that use of the age variable has gained popularity especially with counties, cities, and other jurisdictions that are severely handicapped with lack of resources to collect data on pavement condition, loading, material composition, etc.

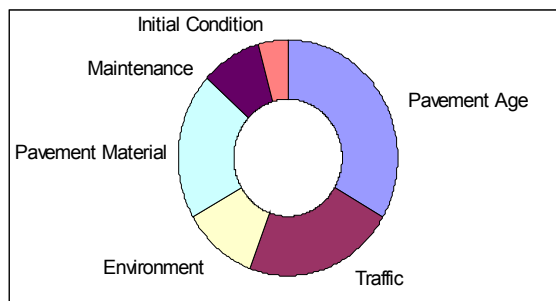


Figure 3-21: Frequency of Use of Independent Variables in Reviewed Performance Models

Another explanation for the dominance of the age variable is that age is a common factor in the estimation of the accumulated effects of both traffic loading and vagaries of the environment over the entire life of a pavement. In this respect, it can be argued that age acts as a surrogate for the cumulative impacts of all these factors over time. Indeed, several researchers have consistently argued that age is the single most important variable influencing pavement performance.

Many researchers utilized traffic loading and pavement thickness or strength as distinct entities in their models, but a few realized that the use of these two variables in the same model gives rise to problems of statistical collinearity, and have therefore used these two variables as a ratio. Some researchers used “ESAL-pavement thickness” as an interaction term to capture the role played by traffic loads and pavement strength. Others used the “ESAL-structural number” instead, as this is obviously a more representative parameter due to the fact that the structural number computation process includes both the thickness as well as the mechanical properties of the various pavement layers. Rather than include environmental factors as a distinct variable for performance models, many researchers resorted to grouping pavements on the basis of environmental characteristics, and then building models for each group. A few others, however, directly used environmental variables, such as freeze index, and average temperature, in their models.

3.9.2.3 Pavement Type

As seen in Figure 3-22, a rather large fraction (50%) of models reviewed was developed specifically for flexible pavements without overlay. On the other extreme, only 2 were developed specifically for continuously reinforced concrete pavements.

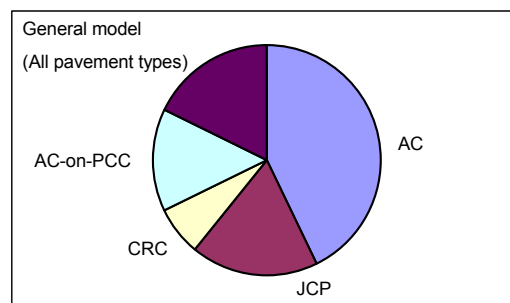


Figure 3-22: Pavement Types Encountered in the Reviewed Performance Models

3.9.3 Methods of Cost-effectiveness Evaluation

The literature review indicated a widespread and long-established use of the area under the performance curve as a surrogate for benefits of pavement repair. Over the years, this concept has been refined to incorporate performance usage, rather than merely performance, by using cumulative loading (instead of time). This is done either by using cumulative load as the abscissa on the performance graph, or by multiplying the area under the performance curve by traffic volume (in terms of AADT) or traffic loading (ESALs) or in some cases, by travel (VMT). Most researchers have carried out costing of pavement repair on the basis of life cycle, rather than just the initial costs. Traditionally, costing has covered the material acquisition and placement costs, but there is a growing trend of pavement researchers to include the costs associated with lane closure due to maintenance. In current practice, cost-effectiveness evaluation of a maintenance activity is carried out by comparing both benefits and costs associated with that activity to that of a base case (typically the zero-maintenance scenario).

3.9.4 Maintenance Cost-effectiveness Studies

[Table 3-6](#) shows a summary of selected published information on the general performance of preventive maintenance treatments, while [Table 3-7](#) synthesizes the performance of such treatments specifically in terms of service life.

**Table 3-6: Summary of Selected Published Information
Preventive Maintenance Effectiveness on Pavement Condition:**

Agency	Treatment	Performance	Comments, Source and Reference
SHRP (SPS-4 Rigid Pavement Test Sections)	Sealing of joints in rigid pavements	Unsealed joints experienced more spalling than sealed joints	[Morian et al., 1998] Treatments rather than strategies are being evaluated.
	Undersealing, and Sealing of joints in rigid pavements	No conclusions yet	
	Other preventive maintenance treatments on rigid pavements	Diamond grinding, dowel installation and consistently-maintained edges resulted in significantly reduced pumping	
SHRP (SPS-3 Flexible Pavement Test Sections)	Sealing of cracks	<ul style="list-style-type: none"> • No treatment-specific observations yet • More cost-effective to carry out PM throughout life of pavement • Service life extension can be maximized if PM is carried out on pavement in good to fair condition 	[Hanna, 1994] Most test sections have a granular base. Treatments rather than strategies are being evaluated.
	Chip sealing		
	Slurry Sealing		
	Thin Overlay		
SMERP Program (Texas)	Traditional Chip Seals	Performance of chip-sealed pavements same for those in good initial condition as those in fair or initial condition	[Syed et al., 1998] Treatments rather than strategies were evaluated.
	ARM chip seals	ARM chip-sealed pavements in good initial condition outperformed those in fair or bad initial condition	
	Fog Seal	Fog seals had little or no impact	
	All treatments	PM treatments on pavements in good initial condition generally outperformed those in fair or bad initial condition	

PM- Preventive Maintenance

Table 3-2 (continued): Preventive Maintenance Effectiveness on Pavement Condition: Summary of Selected Published Information

Agency	Treatment	Performance	Comments, Source and Reference
The Oakland MTC Study	Rejuvenating Seal	<ul style="list-style-type: none"> • Strategies that did not involve PM were found to be poor choices • Pavement condition at time of PM is a vital factor in cost-effectiveness • Average annual maintenance cost is higher in long-term if pavement is allowed to deteriorate 	[Darter et al., 1987]
	Slurry Seal		
	Single Chip Seal		
	Double Chip Seal		
	Thin HMAC overlay		
	No preventive maintenance		
Purdue University	Crack Sealing	Increased levels of crack sealing in the Fall season results in significantly decreased resources expended on corrective maintenance (patching) in the following Spring Season.	[Sharaf and Sinha, 1986]
The Ontario MTC Study	Various combinations of crack sealing and HMA overlay application intervals, including a “do-nothing” strategy	In the long run, strategy involving crack sealing every 4 years and thin HMA overlay every 8 years was found to be most cost-effective	Strategies, rather just treatments, were evaluated. [Chong et al., 1988]
Wisconsin DOT	Sealing of Joints	<ul style="list-style-type: none"> • Pavements with unsealed joints performed better than those with sealed joints • Pavements with wide joints outperformed those with narrow joints 	[Shober, 1994]
	Non-sealing of joints		
The Mississippi Study	Surface Treatment	Pavement condition at time of maintenance has a profound effect on service life	[Rajagopal and George, 1991]
	Thin HMAC overlay		
The Florida I-10 Study	Stitching of cracks in rigid pavements	<ul style="list-style-type: none"> • Over 5-year life extension observed & faulting reduced. • Retrofit dowels yield higher load transfer than shear devices 	[Darter et al., 1994]

**Table 3-7: Summary of Selected Published Information
Preventive Maintenance Effectiveness on Service Life**

Agency	Treatment	Service Life (approx.)	Comments, Source and Reference
Indiana DOT	Chip Seal	4 years average	For pavement in good condition. [Feighan, et al., 1986]
	AC crack seal	2.2 years average	
Ontario MTC	AC Rout and Seal	2-5 years	[Joseph et al., 1992]
New York State DOT	PCC Joint & Crack Filling	2 years	[New York State DOT, 1992]
	PCC Joint & Crack Sealing	8 years	
	AC Rout & Crack seal	5 years	
	AC Crack filling	2 years	
	Thin Overlay	8 years	
	Surface Treatment	3 years median	
NCHRP	Chip Seal	1-6 years	[Shuler, 1984]
	Slurry Seal	1-6 years	
	Micro-surfacing	4-6 years	
	Thin Overlay	> 6 years	
FHWA	Micro-surfacing	5-7 years	[Raza, 1994]
	Slurry Seal	3-5 years	
	Thin Overlay	8-11 years	
	Chip Seal	4-7 years	
Oregon DOT	Chip Seal	3-6 years	[Parker, 1993]
U.S. Corps of Engineers	Slurry Seal	3-6 years	[Brown, 1988]
	Surface Treatment	3-6 years	
	Crack seal	3-5 years	

CHAPTER 4 SYNTHESIS OF QUESTIONNAIRE SURVEY RESULTS

4.1 Introduction

As part of present study, a survey was carried out to determine the state-of-practice of highway pavement maintenance and to document the experiences of Indiana's district and sub-district pavement managers. Districts are responsible for supervising maintenance and other pavement work given out on contract, while sub-districts directly carry out maintenance on a force-account budget. Maintenance treatments carried out under each of these administrative units are varied with respect to work cycle (routine and periodic), and type of work (corrective and preventive) over the years. District and sub-district pavement managers have acquired an intimate and first-hand knowledge about the behavior of pavement systems in response to maintenance treatments and other factors. By including their perceptions in this study, advantage is taken of their immense field experience in this area.

The survey questionnaire, designed along the guidelines suggested by NHCRP 223 [Geoffroy, 1996], was mailed to each of INDOT's 38 sub-districts and 7 districts (1 of which is the Toll Road district). A list of the sources of responding districts and sub-districts is provided in [Table 4-1](#). The questionnaire begun with an introduction to the terminology used for describing various levels and types of such maintenance, as recommended by NCHRP 223, and approved by INDOT Central Office. The various "standard" types of maintenance treatments indicated in the questionnaire are discussed in Chapter 3 (Literature Review). Responses to the questionnaire provided a useful insight into maintenance application criteria as well as the effectiveness of various maintenance treatments in the short-term (change in condition), the long-term (extension in service life), and trade-offs between various maintenance treatments and rehabilitation cycles. A discussion of the responses to each part of the questionnaire is presented in the subsequent sections of this chapter.

Table 4-1: List of Responding Districts and Sub-districts

Respondent Name	Abbreviation	Respondent Name	Abbreviation
1. Fort Wayne District	FTW	9. Evansville Sub-district	EVL
2. Greenfield District	GRF	10. Frankfort Sub-district	FRT
3. LaPorte District	LAP	11. Goshen Sub-district	GOS
4. Vincennes District	VIN	12. Madison Sub-district	MAD
5. Toll Road District	TOL	13. Petersburg Sub-district	PET
6. Angola Sub-district	ANG	14. Wabash Sub-district	WAB
7. Centerville Sub-district	CEN	15. Warsaw Sub-district	WAR
8. Columbus Sub-district	COL	16. Winamac Sub-district	WIN

4.2 Usage of Specific Preventive Maintenance Treatments

Questions were asked regarding specific treatments typically used to address distresses on each of the three major pavement types, and details of their responses are discussed in the subsequent section.

4.2.1 Preventive Maintenance on PCC pavements

Figure 4-1 shows the distribution of respondents who indicated the use of specified preventive maintenance treatment types for PCC pavements.

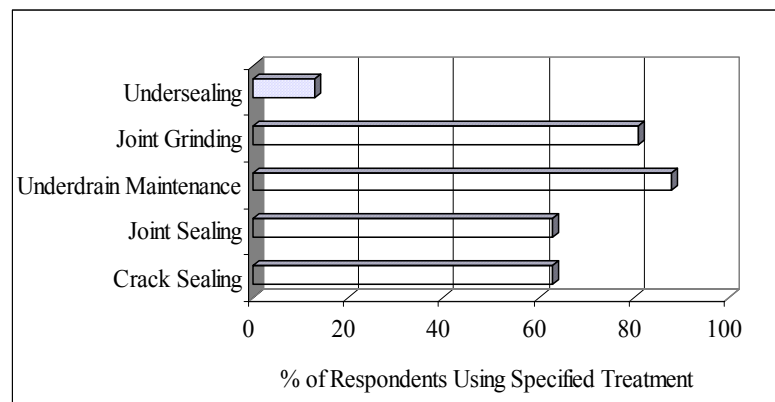


Figure 4-1: Usage of Specified Preventive Maintenance Treatments, PCC pavements

It is seen from [Figure 4-1](#) that most respondents indicated the use of some form of preventive maintenance treatment to protect rigid pavements from accelerated deterioration. Inspection and cleaning out underdrains is the most prevalent activity. Underdrains are deep subsurface drains located at a sufficient depth to intercept and lower groundwater to an acceptable level, and maintenance of such systems are carried out by video-inspection of the pipes and flushing any debris found within. Underdrain maintenance is therefore carried out to ensure that the pipes in such drains function effectively and keep the pavement layers free of unwanted water.

Joint grinding, which is the use of mechanical equipment to level faulted joints, is also a very common activity for PCC pavements. Many of the state's jointed PCC pavements are near the end of their design lives, with a mean age exceeding 20 years. Many of such pavements were constructed at a time when dowel and underdrain technologies were relatively young or non-existent. Therefore these pavements typically suffer from distresses that ultimately lead to joint faulting. Districts and sub-districts carry out joint grinding to retard the faulting spiral and to arrest the development of secondary distresses that are associated with this defect.

Most respondents indicated that they carry out crack and joint sealing on their PCC pavements. However, two sub-districts, Warsaw and Wabash, indicated that they do not typically carry out joint or crack sealing on their PCC pavements. While this may seem contrary to conventional wisdom, it may probably be a prudent policy when one considers the nature of surficial soils in those areas. Most of Wabash and Warsaw lie in the Upper Wabash River Basin, which is characteristically flat and poorly drained, and whose soils largely consist of stratified sand and gravel (kames), deposited as outwash by glacial meltwaters during the ice age [Fenelon, 1994]. The subgrades in those areas are therefore relatively granular in nature, with low plasticity indices and relatively high CBRs. Such mechanical properties of soils typically guarantee them significant stability even under inundated conditions, and therefore sealing their joints and surface cracks to prevent the ingress of surface run-off may have relatively little utility and may not be cost-effective.

4.2.2 Preventive Maintenance on Full-Depth Asphaltic Concrete Pavements

Figure 4-2 shows the distribution of respondents indicating the use of specific treatment types on full-depth asphaltic concrete pavements.

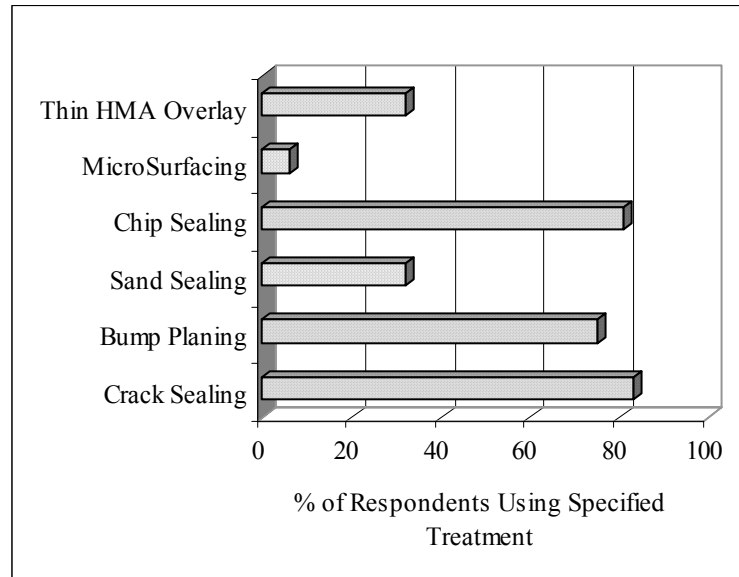


Figure 4-2: Distribution of Usage of Specific Preventive Maintenance Treatments on Full-Depth Asphaltic Concrete (FDA) Pavements

Crack sealing is the most dominant pavement preventive activity on full-depth asphaltic concrete pavements. This is done to keep the pavement free from surface moisture, especially in areas with poor subgrade quality. The use of chip sealing is prevalent among the districts and subdistricts as a stopgap measure to defer the need for higher levels of maintenance or rehabilitation. However, there is a general policy not to apply such surface treatments to Interstate highways due to the hazards posed by airborne chips. A third of all respondents indicated that they use sand sealing and thin overlay HMA treatments, while only a few indicated that they use micro-surfacing. Most respondents indicate that they use bump planning to level irregularities on flexible pavement surfaces. Bump planing may be considered a preventive maintenance activity because if uncorrected, bump severity may worsen due to mechanical vibration of passing traffic. The dominance of crack sealing, chip sealing and joint bump repair treatments on state highway pavements of this type can be explained by the nature of the pavement material as well as the ambient environment. Asphaltic cement undergoes plastic deformation under load and warm weather,

while it experiences brittleness and breakup in cold weather, therefore necessitating frequent bump planing and crack sealing, respectively. The nature of climatic conditions in the state of Indiana (cold winters and warm summers) fosters the development of such distresses. Bumps are also caused by deformation of pavement layers beneath the surface. Also, the hardening of bitumen with age causes loss of binder/aggregate bonding and results in raveling of the pavement surfaces especially in their mid-lives. For such pavements, districts and sub-districts typically apply a chip or sand coat to fill cracks, to rejuvenate the pavement surface, and to preserve the pavement in satisfactory condition until the next resurfacing activity.

A few respondents indicated that they apply thin HMA overlays when the pavement reaches a relatively advanced age. This treatment is typically carried out under contract and supervised at the district level.

4.2.3 Preventive Treatments on Overlay Pavements

Figure 4-3 shows the distribution of respondents indicating the use of specific treatment types on AC-over-PCC overlay pavements.

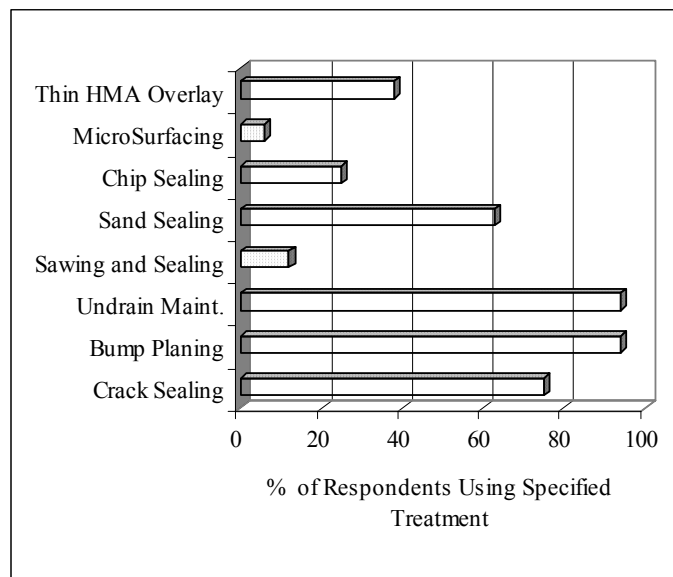


Figure 4-3: Distribution of Usage of Specific Preventive Maintenance Treatments on AC-over-PCC Overlay Pavements

The distribution of types and usage of preventive maintenance treatments on overlay pavements is similar to that for flexible pavements (Figure 4-2). However, it is seen that slightly more respondents use each type of treatment for OVR pavements than they do for flexible pavements. It is also seen that overlay pavements receive higher levels of maintenance (e.g., thin HMAC overlay) compared to full-depth asphaltic concrete pavements. This is probably because the AC layer that comprises the topmost layer of overlay pavements does not always effectively cover defects that existed on the underlying PCC slab at time of overlay.

It is worth noting that some respondents refrain from carrying out certain maintenance treatments not because such treatments are not effective, but due to restrictions imposed by institutional policy. For instance, work expenditure ceilings in sub-districts' force account budget dictate that some treatments, by virtue of their high costs, should be carried out by contract under supervision by the districts.

4.3 Application Criteria and Benefits of Individual Preventive Maintenance Treatments

Tables 4-2 to 4-4 present syntheses of the responses of pavement managers when asked about the timing and benefits of each individual preventive maintenance treatment for rigid (PCC) pavements, full-depth asphaltic concrete pavements (FDA), and AC-over-PCC overlays (OVR). In particular, the respondents answered questions about the age of pavement at time of first application of the treatment, average frequency of application thereafter, and the perceived increase in pavement life due to the application of the treatment.

4.3.1 Treatments on PCC Pavements

Table 4-2 presents application criteria and benefits of individual preventive maintenance treatments for PCC Pavements. Six respondents reported that they reseal their joints 3-10 years after construction and every 3-10 years thereafter. All respondents who use joint sealing reported that they had perceived a 5-20 year increase in pavement service life due to that maintenance treatment, a benefit which seems to be much higher than that expected of this treatment.

Table 4-2: Timing and Benefits of Individual Preventive Maintenance Treatments, PCC Pavements

Treatment	Average Age at 1 st Application (Years)	Average Frequency of Application (Years)	Average Perceived Increase in Pavement Life (Years)	Respondent
Joint (Re) sealing	3	1	“Significant increase”	Greenfield District
	5-10	5 approx.	15-20 years	Madison Sub-district
	10	“Varies”	“Varies”	Fort Wayne District
	10	10	6-7	Vincennes District
	5	3	12+	Angola Sub-district
	10	10	5	Petersburg Sub-district
Underdrain Maintenance	1	1	Not indicated	Angola Sub-district
	3	3	1 year	Petersburg Sub-district.
	3	3	1 year	Vincennes District
	1	1	“Some benefits”	Greenfield District
	1	1	Not indicated	Madison Sub-district
	1	1	10% increase	Wabash Sub-district
	2	2	“Some benefits”	Fort Wayne District
	1	2	“Some benefits”	Vincennes District
Crack Sealing	5+	1	Not indicated	Greenfield District
	1	“As needed”	3-5	Toll Road District
	3	5	10% increase	Wabash Sub-district
	15+	“Varies”	“Varies”	Fort Wayne District
	10	3-5	2-5	Vincennes District
	5	3	12+	Angola Sub-district
	5	5	Not indicated	Warsaw District
Crumb Rubber Sealing	1-2	As needed	“Significant”	Laporte District

Also, crack sealing, from the perception of the respondents, imparts 2-12 years of extra service life to the pavement. Two sub-districts, Wabash and Warsaw reported that they did not seal cracks on their PCC pavements. The Toll Road District reported that for the very few PCC pavements in its jurisdiction, crack sealing typically yielded 1-5 years extension in service life depending on the quality of concrete originally used for the construction. Most respondents indicated that they carry out underdrain maintenance a year after construction and at frequent (1-3 years) intervals thereafter. Angola sub-district specifically touted the benefits of underdrain maintenance, stating that “PCC pavement life is longer where underdrain maintenance was carried out”.

4.3.2 Treatments on Full-Depth Asphaltic Concrete (FDA) Pavements

Table 4-3 presents a synthesis of the answers of the respondent pavement managers when asked about the timing and benefits of each individual preventive maintenance treatment for full-depth asphaltic concrete pavements (FDA).

Virtually all respondents indicated that they carry out crack sealing of their FDA pavements, but at starting times that vary considerably (1-8 years after rehabilitation), and at frequencies of 2-5 years after first application. All respondents perceived significant increase in pavement life (2-5 years) due to crack sealing. Angola sub-district stated that crack sealing, when carried out in a timely fashion, has extended pavement life considerably.

In some cases, crack sealing has been carried out using crumb rubber; a treatment that as started relatively recently but has won over many pavement managers because of the superior effectiveness it offers compared the traditional material used for this treatment. Frankfort sub-district reported that crumb rubber sealing for large cracks is very effective in improving pavement condition and in extending service life. Laporte district reported that “the application of crumb rubber sealing in the first and second years after rehabilitation has greatly reduced the incidence of cracking and overall maintenance costs in subsequent years”.

Table 4-3: Timing and Benefits of Individual Preventive Maintenance Treatments, FDA Pavements

Treatment	Average Age at 1 st Application (Years)	Average Frequency of Application (Years)	Average Perceived Increase in Pavement Life (Years)	Respondent
Crack Sealing	3	4-5	4-5	Fort Wayne District
	2	5	20% increase	Wabash Sub-district
	2	3	5	Madison Sub-district
	1	2	2	Vincennes District
	3	“As needed”	“Very significant”	Greenfield District
	8	“Varies”	2	Angola Sub-district
	3	3	3	Petersburg Sub-district
Chip Sealing	7	5	5	Fort Wayne District
	8	5	20%	Wabash Sub-district
	4	6	Not indicated	Winamac Sub-district
	6	6	5	Madison Sub-district
	10	5	5	Vincennes District
	8	5-6	10+	Angola Sub-district
	15	3	Not indicated	Petersburg Sub-district
Sand Sealing	10	5	5	Vincennes District
	15	3	Not indicated	Petersburg Sub-district
Thin HMAC Overlay	15	10-15	10-15	Vincennes District
	20	Not indicated	Not indicated	Angola Sub-district
	15	10	10	Petersburg Sub-district
Micro-surfacing	15	Not indicated	3	Vincennes District
Crumb Rubber Sealing	1-2	Not indicated	Not indicated	Winamac Sub-district

Like crack sealing, chip sealing is also very common for full-depth asphaltic concrete pavements. From [Table 4-4](#), it is seen that districts and sub-districts typically apply chip seals to their pavements 4-10 years after reconstruction or rehabilitation, and approximately every six years thereafter.

The average perceived extension in pavement life due to chip sealing is 6 years. Frankfort sub-district provided a specific recent example of chip sealing effectiveness. According to this sub-district, crack sealing on State Road 47 from Thorntown to US Road 52 Junction) has “done a good job” of extending the life of the pavement section. Columbus sub-district also reported that application of chip seals has greatly enhanced pavement condition of their full-depth asphaltic concrete roads.

Respondents indicated that thin HMAC overlays are typically applied 15-20 years after rehabilitation, and every 10-15 years thereafter. The perceived increase in pavement life due to thin HMAC overlay and microsurfacing are 10 and 3 years respectively. Respondents indicated that both these treatments, which involve the application of overlay of a finite thickness not exceeding 1.5 inches, are associated with the highest levels of effectiveness in terms of increasing pavement condition and extending service life. Moreover, it is apparent that such treatments also offer the greatest value per dollar, especially if applied to the pavement at mid-to-old age. However, as indicated by respondents, the high cost of such major preventive maintenance treatments inhibits their widespread use.

4.3.3 Treatments on AC-over-PCC Overlay (OVR) Pavements

[Table 4-4](#) presents a synthesis of the answers of the respondent pavement managers when asked about the timing and benefits of each individual preventive maintenance treatment for AC-over-PCC overlay pavements (OVR).

The application timing criteria and perceived benefits of preventive maintenance treatments for AC-over-PCC pavements were generally similar to those of full-depth AC pavements. Respondents indicated that crack sealing is typically carried out about 2 years after rehabilitation and at approximately 3-year intervals thereafter, or as needed.

Table 4-4: Timing and Benefits of Individual Preventive Maintenance Treatments, Overlay Pavements

Treatment	Average Age at 1st Application (Years)	Average Frequency of Application (Years)	Average Perceived Increase in Pavement Life (Years)	Respondent
Crack Sealing	2	3	5	Madison Sub-district
	1	1	3-5	Toll Road District
	3-5	“As Needed”	5-6	Greenfield District
	2	3	3	Petersburg Sub-district
	1	2-3	2	Vincennes District
	2	4-5	4-5	Fort Wayne District
	2	5	20%	Wabash Sub-district
	2	3	10	Angola Sub-district
	2	3	3	Petersburg Sub-district
Chip Sealing	8	5	5	Vincennes District
	15	3	Not indicated	Petersburg Sub-district
	7	5	5	Fort Wayne District
	8	5	20%	Wabash Sub-district
	10-12	5	10	Angola Sub-district
Sand Sealing	8	5	5	Vincennes District
	15	3	Not indicated	Petersburg Sub-district

Table 4-4: Timing and Benefits of Individual Preventive Maintenance Treatments, Overlay Pavements (continued)

Treatment	Average Age at 1st Application (Years)	Average Frequency of Application (Years)	Average Perceived Increase in Pavement Life (Years)	Respondent
Thin HMAC Overlay	18-25	7-13	5-6	Toll Road District
	“Varies”	10-15	10-15	Vincennes District
	18-25	10	10	Angola Sub-district
	15	10	10	Petersburg Sub-district
Micro-surfacing	15	Not indicated	3	Vincennes District
Crumb Rubber Sealing	3-5	“As needed”	“Very Significant”	Greenfield District
	1-2	Not indicated	“Great increase”	Laporte District
Underdrain Maintenance	1	1	20%	Wabash Sub-district
	1	1	Not indicated	Angola Sub-district

The perceived benefits of crack sealing on overlay pavements range from 2-10 years extension in service life, a rather wide variation that is probably due to differences in subgrade vulnerability, material used, climatic differences, and institutional practices. Respondents who use chip seals on overlay pavements indicated that they first do so when the pavement is 7-15 years of age, and repeat this treatment at 5-year intervals. Two sub-districts that indicated the use of sand seals use timing criteria similar to that used for chip seals.

Major preventive maintenance activities such as thin HMAC overlays and microsurfacing are carried out by a number of districts and sub-districts when the overlay pavement reaches a relatively advanced age (15 years, on the average). However, the perceived increase in service life for thin HMAC overlays (5-15 years) is far greater than that offered by micro-surfacing (3 years). The use of crumb rubber for sealing cracks on overlay pavements was reported by 2 respondents albeit with very different

application timing criteria. However, both sub-districts reported that this treatment had resulted in very substantial increases in the service life of overlay pavements.

Subsurface drainage systems were constructed for existing PCC slabs on most rigid pavements in the 1980's, a policy geared toward increasing the life of such pavements. In later years, some of these pavements were resurfaced with asphaltic concrete, but there is still a need to maintain the subsurface drains of such pavements. Inspection and cleaning of such underdrains is especially crucial for such formerly rigid pavements that were initially constructed directly on fine-grained and/or plastic subgrades, which are vulnerable to strength loss upon wetting. Most respondents indicated maintenance of underdrains on overlay pavements every year. One respondent reported that this preventive maintenance treatment has extended the service life of overlay pavements by 20%, while another respondent stated that the benefits of this treatment are "significant but are hard to measure".

4.3.4 Overall Discussion for Treatments on All Pavements

Average values of application timing criteria and perceived benefits (service life extension) of each treatment on each pavement type, for all responding districts and sub-districts, are provided as [Table 4-5](#). It is significant to note that for crumb rubber sealing, respondents did not indicate the frequency of application or perceived increase in service life. This is obviously because this treatment is relatively new in Indiana (use begun in 1995). It is also worth noting that for almost all treatments, the extension in pavement life is approximately equal to the frequency of application. This is consistent with expectation because a treatment is typically repeated just after expiration of the extension it offers to pavement service life.

Table 4-5: Timing and Benefits of Individual Preventive Maintenance Treatments, All Pavements

Pavement Type	Treatment	Average Age at 1st Application (Years)	Average Frequency of Application (Interval in Years)	Average Perceived Increase in Pavement Life (Years)
PCC	Joint Sealing	8	6	10
	Crack Sealing	6	4	6
	Underdrain Maintenance	1	2	2
ACP	Crack Sealing	3	4	3
	Chip Sealing	7	5	6
	Sand Sealing	12	4	5
	Crumb Rubber Sealing	2	NI	NI
	Microsurfacing	15	NI	3
	Thin HMAC Overlay	17	11	11
OVR	Underdrain Maintenance	1	1	2
	Crack Sealing	2	3	4
	Chip Sealing	10	5	5
	Sand Sealing	12	4	5
	Crumb Rubber Sealing	1.5	NI	NI
	Micro-surfacing	15	NI	3
	Thin HMAC Overlay	20	11	9

Note: 1) NI- not indicated. 2) All values rounded-off to the nearest integer.

4.4 Details and Benefits of Preventive Maintenance Strategies

4.4.1 Uses and Variation of Preventive Maintenance Strategies

Figures 4-4 (a) and (b) provide details of any strategies that are used by sub-districts. The questionnaire sought to determine the extent to which preventive maintenance strategies (rather than treatments) are used by the various sub-districts and districts. A strategy consists of a series of treatments spread out over specific intervals in time. For sub-districts that use preventive maintenance strategies, the reasons for use are shown in Figure 4-4 (a), while Figure 4-4 (b) indicates the number of sub-districts for using such strategies by functional class, and traffic volume.

More than half of the respondents indicated that they use some form of preventive maintenance strategy. The remaining respondents do not use any strategy, but apply individual treatments as and when necessary. For sub-districts that indicated affirmative responses, details of the preventive maintenance strategy adopted for each pavement type are provided in subsequent sections of this chapter. For sub-districts that reported non-use of any such strategies, lack of funding was indicated as a primary reason for that situation. Others reported that they had a form of “strategy” that was characterized by treatments applied at irregular intervals.

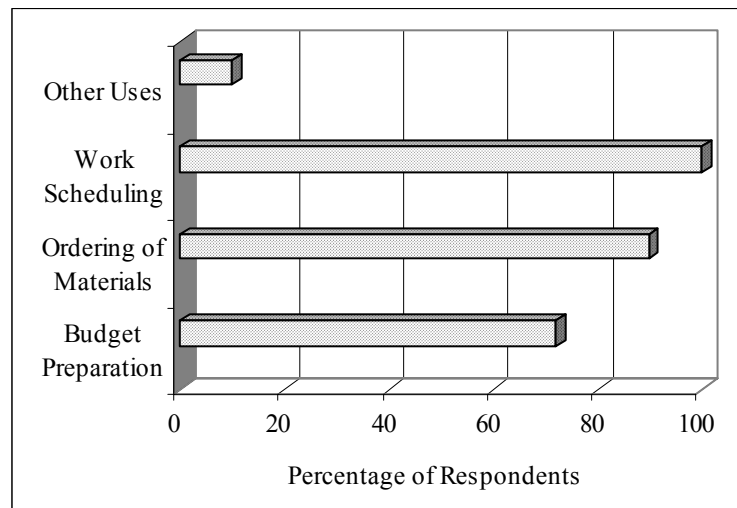


Figure 4-4(a): Uses of Preventive Maintenance Strategies

A majority of respondents from sub-districts with a strategy in place for preventive maintenance indicated that such a strategy is used to schedule pavement maintenance activities for the following maintenance period. As such the strategies help them to order materials and to prepare their budgets for the next year.

Figure 4-4 (b) indicates the distribution of respondents for whom preventive maintenance strategies differ by pavement attributes. For a given pavement type, most respondents indicated that their strategies vary by traffic volume and road functional class. For example, chip and sand sealing are not carried out on Interstate pavements.

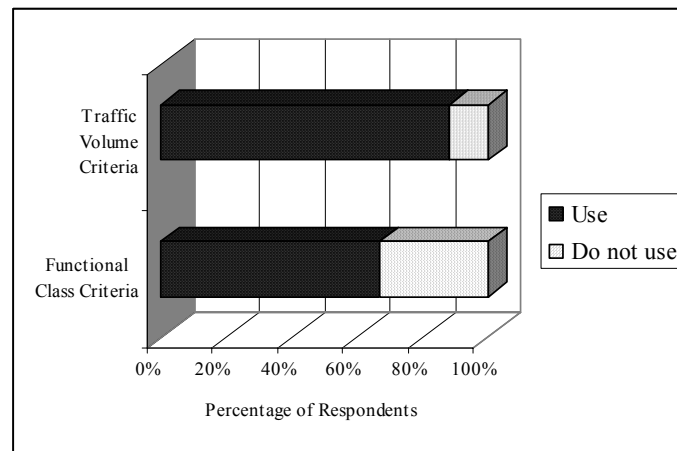


Figure 4-4(b): Criteria for Preventive Maintenance Strategy Use

4.4.2 Application Criteria and Benefits of Preventive Maintenance Strategies

Respondents who currently use some form of preventive maintenance strategy for their pavements were asked to provide details on the application criteria of each maintenance treatment type in each strategy. Such criteria included age of the pavement at time of first application and the frequency of application of each treatment type. Respondents were asked for indications of the effectiveness (perceived benefits) of each strategy with regard to service life extension, reduction in levels of corrective maintenance in the subsequent year, and increase in pavement condition in general. This was done for each pavement type. Table 4-6 presents the details (application timing criteria and effectiveness) of preventive maintenance strategies currently used by some districts and sub-districts in the state.

Table 4-6: Details and Effectiveness of Preventive Maintenance Strategies

Pavement Type	District/ sub-district	Perceived Effectiveness of Preventive Maintenance (PM) Strategies				Application Criteria for PM Strategy
		R/R cycle (years) if PM is NOT used	R/R cycle extension (years) if PM is used	Reduction in level of CM if PM is used	Increase in pavement condition if PM is used	
PCC	GRF	16-20	3-4	16-20%	11-15%	<ul style="list-style-type: none"> - Underdrain maint. after 1st year and as needed - Joint Sealing after 3rd year and as needed - Crumb rubber sealing every 3-5 years - Crack sealing after 5+ years
	WAB	13-15	3-4	16-20%	5-10%	<ul style="list-style-type: none"> - Underdrain maint. after 3rd year and every year thereafter - Crack Sealing after 3rd year and every 5 years thereafter
	MAD	16-20	7-8	16-20%	21-25%	<ul style="list-style-type: none"> - Underdrain maint. every year - Joint Sealing after 5-10 years and every 5 years thereafter - Thin HMAC Overlay after 10-15 years
	Average	17	5	18	15	

Note: Perceived effectiveness of preventive maintenance strategies are rounded-off to the nearest integer.

PM- Preventive Maintenance

CM- Corrective Maintenance

Table 4-6: Details and Effectiveness of Preventive Maintenance Strategies (continued)

Pavement Type	District/ sub-district	Perceived Effectiveness of Preventive Maintenance (PM) Strategies				Details of PM Strategy
		R/R cycle (years) if PM is NOT used	R/R cycle extension (years) if PM is used	Reduction in level of CM if PM is used	Increase in pavement condition if PM is used	
FDA	GRF	10-12	3-4	21-25%	21-25%	- Crack Sealing after 3rd year and every 5 years thereafter
	MAD	10 max	3-4	11-15%	16-20%	- Crack Sealing 3 years - Chip Sealing after 6th year and every 6 years thereafter
	WAB	10-13	5-6	5-10%	5-10%	- Crack Sealing after 2 years and every 3 years thereafter - Chip Sealing after 8th year and every 5 years thereafter
	ANG	10-12	5-6	16-20%	16-20%	- Crack Sealing after 5 years, variable frequency thereafter - Chip Sealing after 8th year and every 5-6 years thereafter - Thin overlay after 20 yrs
	Average	11	5	15	17	
OVR	TRD	10 max	5-6	16-20%	21-25%	- Crack Sealing as and when needed - Thin overlay after 10 yrs
	GRF	10-15	5-6	16-20%	21-25%	- Crack Sealing after 3rd year and every 5 years thereafter
	MAD	10 max	3-4	11-15%	16-20%	- Crack Sealing every 3 years
	WAB	13-15	5-6	5-10%	16-20%	- Underdrain maintenance every year - Crack Sealing after 2 years and every 5 years thereafter - Chip Sealing after 8th year and every 5 years thereafter
	ANG	10-12	5-6	11-15%	16-20%	- Underdrain maintenance every year - Crack Sealing after 2 years and every 3 years thereafter - Chip Sealing after 10-12 years and every 5 years thereafter - Thin overlay after 18-20 years and every 10 years thereafter
	Average	12	5	14	20	

Note: 1) Perceived effectiveness of preventive maintenance strategies are rounded-off to the nearest integer.
2) PM- Preventive Maintenance
3) CM- Corrective Maintenance
4) R/R is rehabilitation or reconstruction

4.4.2.1 Discussion on Application Criteria and Benefits of Preventive Maintenance Strategies

As indicated in Section 4.4.1, 11 respondents reported that they had in place some form of preventive maintenance strategy. Of these however, only 6 respondents provided details and benefits of their strategies. The discussion below is for each pavement type.

4.4.2.1.1 Preventive Maintenance Strategies for PCC Pavements

For PCC pavements, Greenfield District (GRF) has a strategy that includes underdrain maintenance after the first year of rehabilitation, and as needed in the future. Secondly, this district carries out resealing of PCC joints at the third year following reconstruction, and at variable intervals after that. Also, cracks on PCC pavements are sealed every 3-5 years. According to the respondent, the use of this strategy has resulted in perceived increases in PCC pavement life by 3-4 years, a 16-20% reduction in the level of corrective maintenance, and a 11-15% increase in pavement condition. Wabash sub-district (WAB) cleans out its underdrains every 3rd year after reconstruction and every year thereafter, and seals cracks on PCC pavements 3 years after reconstruction and at 5-year interval after that. As has been noted in Section 4.2, Wabash does not typically carry out joint sealing of its PCC pavements and seals cracks at longer intervals than Greenfield District does. The respondent from Wabash reported that PCC pavements that have received this preventive maintenance strategy for PCC pavements were afforded an extra 3-4 years in pavement life. Also, the level of corrective maintenance activities, such as shallow patching, was perceived to have reduced by 16-20%, and pavement condition increased by 5-10%. For its PCC pavements, Madison sub-district (MAD) carries out underdrain maintenance every year, and joints are resealed 5-10 years after reconstruction, and at 5-year interval thereafter. This respondent reported that the use of this strategy was very beneficial in the long-term; 15-20 years extension in average service life. Fort Wayne District (FTW) reseals joints and cracks on its PCC pavements 10 and 15 years respectively after reconstruction, and when needed thereafter. Also, the respondent stated that PCC underdrains are inspected and cleaned 2 years after reconstruction at 2-year intervals thereafter. The respondent did not indicate its perception of the benefits of this overall strategy, but rather provided perceived benefits of individual treatments (See Section 4.2.1).

Evansville sub-district carries out preventive maintenance activities every 3 years, but did not provide details on the treatment types, application criteria and perceived benefits.

4.4.2.1.2 Preventive Maintenance Strategies for Full-depth Asphaltic Concrete Pavements

As regards full-depth asphalt concrete pavements, Greenfield district seals cracks after 3 years and every 5 years thereafter. Flexible pavements in Greenfield district that received this strategy benefited from 3-4 years extension in service life, and a 21-25% decrease in both corrective maintenance levels and deterioration levels. Madison sub-district indicated that only a maximum of 10 years of service life was perceived for full-depth asphaltic concrete pavements that did not receive its preventive maintenance strategy for this pavement type. This strategy involved the sealing of cracks every 3 years and application of a chip seal every 6 years. The use of this strategy was perceived to extend service life of AC pavements by a maximum of 4 years, while reducing the level of subsequent corrective maintenance by 11-15% and increasing general pavement condition by 16-20% on such pavements. Compared to Madison and Greenfield, Wabash sub-district has a relatively conservative crack sealing policy (first application after 2 years of rehabilitation, and at 3 years intervals thereafter), but a relatively liberal chip sealing policy (first application after 8 years of rehabilitation, and at 5-6 years intervals thereafter). It was perceived that full-depth AC pavements that received this strategy had about 5-6 years extension in their service lives. Also, such pavements had 5-10% reduction in the levels of their corrective maintenance activities such as shallow patching, deep patching, and premix leveling. Angola sub-district's preventive maintenance strategy for its full-depth AC pavements consists of crack sealing after 5 years of rehabilitation, and at variable frequency thereafter. Chip seals are applied to the pavement in their mid-life (after 8th year, and every 5 years thereafter). A thin overlay is applied at the 20th year. The effectiveness of this strategy appear to be higher than that for other responding sub-districts (5-6 year increase in overall service life), 16-20% decrease in both corrective maintenance and pavement deterioration). Warsaw sub-district in northern Indiana seals cracks on their AC pavements every 5 years, and applies a chip seal every 5 years for sections with relatively low traffic levels (less than 2,500 vpd). The respondent from this sub-district did not provide details on the benefits of the strategy.

4.4.2.1.3 Preventive Maintenance Strategies for Overlay Pavements

The Toll Road District (TOL), which oversees the upkeep of pavements on the busy I-90 Corridor (which links northern Ohio to the Gary-Chicago-Milwaukee Corridor) reported that most pavements in its jurisdiction are AC-over-PCC overlays. Their preventive maintenance strategy consists of crack sealing at variable frequencies and thin HMAC overlay after 10 years. No chip or sand sealing is done. The perceived benefits of this strategy were considered to be profound, as it has made possible for the Toll Road District to extend pavement rehabilitation cycle by 5-6 years and has reduced the level of pavement corrective maintenance up to 20%. Furthermore, the respondent for this district perceived that road users benefited directly from a 21-25 % increase in pavement condition arising from the use of this strategy.

Overlay pavements in the Greenfield District receive crack sealing treatment 3 years after rehabilitation and every 5 years after that. The effectiveness of this strategy is reflected in perceived significant increase in pavement life and pavement condition (See [Table 14](#)). Madison sub-district reported that overlay pavements that do not receive its preventive maintenance strategy for such pavements typically do not last for more than 10 years. However, where such strategy was used, significant benefits (shown in [Table 14](#)) were reaped both in the short and long-term. Wabash and Angola sub-districts use preventive maintenance strategies that consist of 3 or more treatment types, for their overlay pavements. Wabash carries out underdrain maintenance every year, crack sealing 2 years after rehabilitation, and then at intervals of 5 years. Chip sealing is done 8 years after rehabilitation and every 5 years thereafter. With this strategy, 5-6 years of extension in pavement service life were perceived. Also, a 5-10% increase in general pavement condition was perceived for overlay pavements that receive this strategy. In Angola, where the preventive maintenance strategy is more liberal, application of a thin overlay after 18-20 years, is added to the crack sealing and chip sealing regimen as described above, for such pavement types.

4.5 Factors Affecting Sustained Pavement Performance after Maintenance

Respondents were asked about their perceptions on the various factors that militate against sustained maintenance effectiveness. Their responses are presented as [Figure 4-5](#) for rigid pavements, and [Figure 4-6](#) for flexible pavements. For both pavement types, respondents indicated that adequate drainage

was most crucial for sustained pavement condition after maintenance. This was followed by the quality of subgrade. Also, pavement age was found to be very significant. This is intuitive because all else being equal, a mid-aged pavement is expected to sustain the benefits of maintenance for a longer time than older pavements, although older pavements are typically associated with higher short-term benefits (increase in pavement condition). Greater pavement thickness and lower traffic level were found influential for better performance after maintenance. Temperature and precipitation were also perceived to be important. The figure also shows that for flexible pavements, the respondents consider traffic loading to be less detrimental to sustained performance than for rigid pavements. These perceptions appear consistent with expectation, as several cost allocation studies in the past have ascribed relatively lower load shares (relative to non-load factors) to flexible pavement deterioration, while rigid pavements have relatively higher non-load fractions [Fwa and Sinha, 1987; Martin, 1996; Li and Sinha, 2000]. Also the thickness of the rigid pavement slab was found to be more influential than the thickness of the flexible pavement, a finding that is considered intuitive because of the fact that a rigid pavement's slab bears a proportionately higher portion of load compared to that borne by flexible pavements' asphaltic concrete surface layer, for the same level of traffic loading. As in all opinion surveys of contributory factors, the results of the survey need to be interpreted with caution. A factor may not be perceived as significant because there is little variation in its values within the jurisdiction of a respondent.

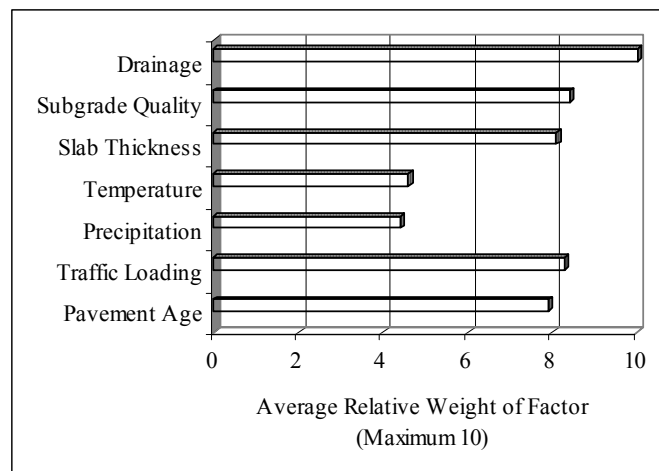


Figure 4-5: Factors Affecting Sustained Performance after Maintenance, Rigid (PCC) Pavements

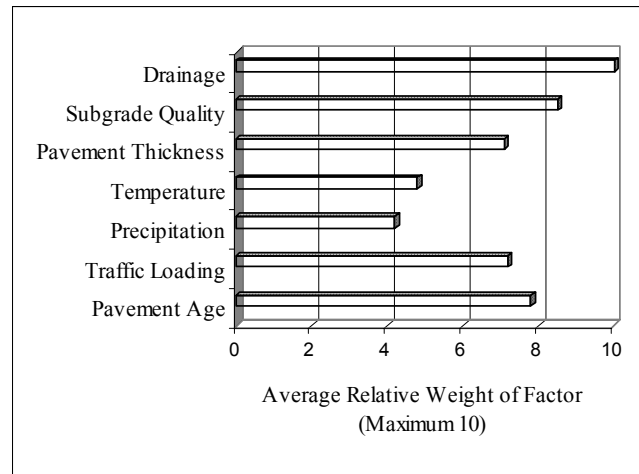


Figure 4-6: Factors Affecting Sustained Performance after Maintenance, Flexible (FDA and OVR) Pavements

4.6 Chapter Summary

The questionnaire survey was designed primarily to identify current state of practice of preventive maintenance for the state highway network in Indiana, and to determine the benefits of such treatments in the short and long-term, and the trade off between preventive maintenance and corrective maintenance from the perspective and experience of pavement managers in INDOT's districts and sub-districts.

For PCC pavements, common preventive maintenance treatments were underdrain maintenance, joint grinding, joint (re)sealing, and crack sealing. Only a few respondents indicated that the use of slab undersealing, obviously because detection of voids under the slab is a difficult undertaking. Two sub-districts located in areas with granular and moist surficial soils do not carry out crack sealing on such pavements. Common preventive maintenance treatments on flexible (full-depth AC and AC-over-PCC overlay pavements) include bump planing, crack sealing, seal coating, and thin HMAC overlay.

Many respondents provided application criteria (age at first application and frequency of application) of individual preventive maintenance treatments and indicated the benefits offered by such treatments, for each pavement type. Also, some respondents provided details about their preventive

maintenance strategies if any. Such respondents provided details on application criteria and benefits (extension in service life, increase in pavement condition, and preventive/corrective maintenance trade-offs) associated with each strategy, for each pavement type. The responses provided overwhelming evidence to support the benefits of preventive maintenance, regardless of pavement type and regional location. Districts and sub-districts with a preventive maintenance strategy in place stated that such a program helped them to schedule work, order materials, and prepare budgets. Finally, respondents stated that drainage, design and construction features, loading, and weather factors are influential factors if the benefits of pavement maintenance is to be sustained.

A comparison of perceptions of maintenance effectiveness from this study and that of an earlier Indiana study [Sinha et al., 1988] showed that service lives in the current study were much higher for the current study, generally between 5-10 times that of the earlier study. Further investigations revealed that the definition of service lives were different for each study. In the questionnaire for the earlier study, extended service life pertained to maintenance treatments, and was defined as “the time that elapses until more work of any kind is necessitated at the location where the treatment was carried out”. In the questionnaire for the present study however, the meaning of extended service life was the time the pavement needed rehabilitation, and pertained to the pavement, and not the treatment.

Summing up, the questionnaire survey provided a vital insight into the effectiveness of various maintenance treatments in the short-term (change in condition) and of strategies in the long-term (extension in service life), as well as trade-offs between various maintenance treatments and rehabilitation cycles. Such information on the state-of-practice therefore offered a practical perspective to concepts and findings from the literature review, and helped build a basis for the design of the short-term and long-term maintenance effectiveness modeling process.

CHAPTER 5: STUDY FRAMEWORK, METHODS AND THEORY

5.1 Introduction

This chapter explains the overall framework (Figure 5-1) methods utilized, and underlying theories for each of the three aspects of the present study, namely short-term effectiveness evaluation, long-term effectiveness evaluation, and trade-off analyses. A detailed discussion of each aspect of the framework is also presented.

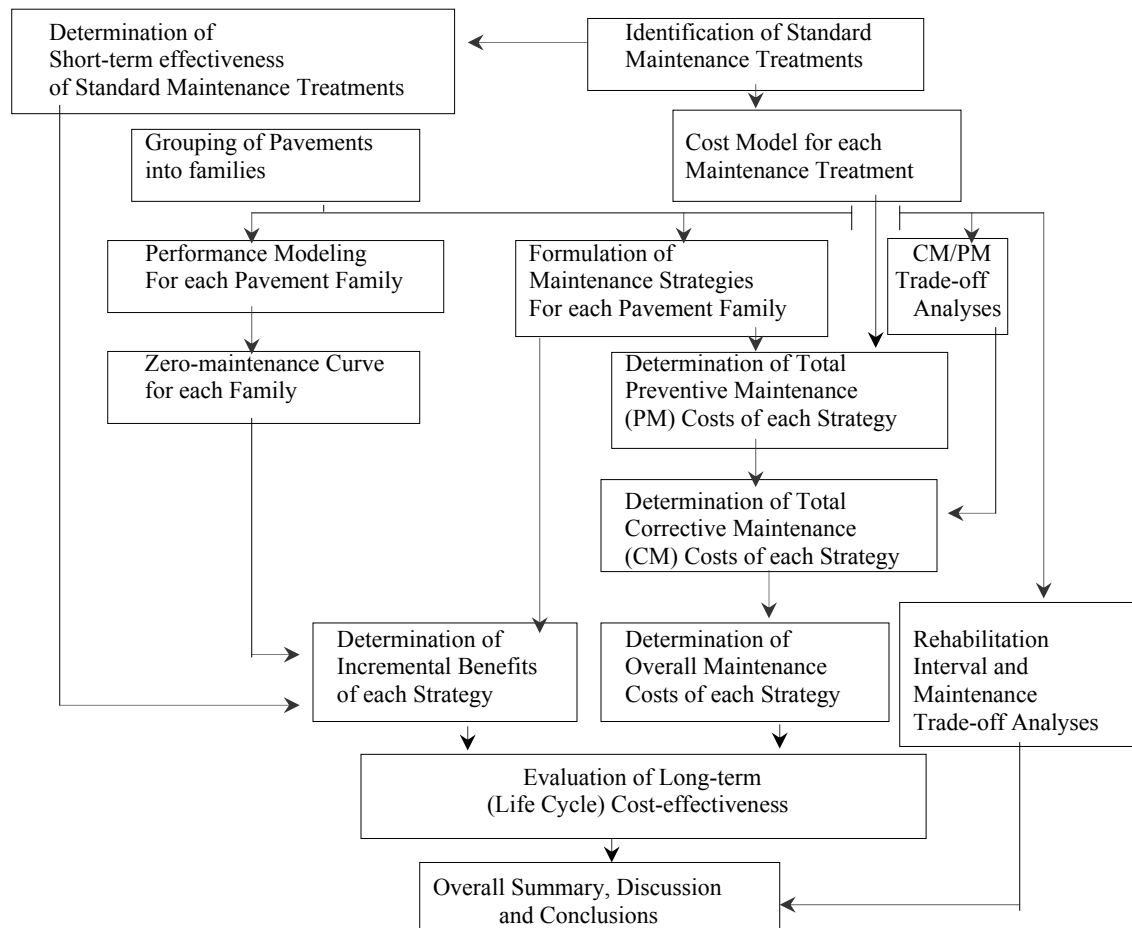


Figure 5-1: Study Framework

5.2 Short-term Effectiveness and Expenditure Modeling

Generally, the effectiveness of pavement maintenance activities is best assessed over an extended period of time, because of the gradual effect pavement performance factors have in influencing pavement condition in general and maintenance effectiveness in particular. Furthermore, it is more useful, from a holistic viewpoint and from the perspective of pavement management systems (PMSs), to assess maintenance strategies (time-based sequences of treatments), rather than just treatments. Evaluations of long-term effectiveness typically involve the use of pavement performance trend models that enable the estimation or prediction of the deterioration in pavement condition over time.

However, short-term effectiveness models, which provide a means of assessing the impact of treatments, immediately after or within one or two years, are very useful in certain cases. Example of such instances include when the necessary data types for long-term evaluation are not available, when data available spans only a short period of time, or most importantly, when organizational units involved with pavement maintenance, such as INDOT's Operations Support Division or Program Development Division, wish to know how effective a specific treatment (or combination of treatments) are in addressing pavement distress within the maintenance cycle period. In these regards short-term maintenance effectiveness models are very useful to operators of maintenance management systems (MMSs). Finally, short-term effectiveness provides a useful input for long-term effectiveness analysis (the performance jump or rate of deterioration reduction for each treatment enables the computation of incremental benefits due to application of an individual treatment that is part of a long-term maintenance strategy).

This chapter starts by discussing certain issues related to short-term maintenance effectiveness modeling, such as the time-lag between maintenance application and effectiveness, endogeneity, simultaneity. Next, the chapter presents the annual pavement maintenance expenditure models that were developed to estimate the level (amount of money expended) on maintenance for a given pavement given a set of explanatory factors. Unlike maintenance treatment accomplishment cost models, pavement maintenance expenditure models are specific to pavements, not treatments. Thirdly and most importantly, this chapter presents models that estimate the short-term effectiveness of maintenance. Such effectiveness

is measured in terms of improvement in pavement condition or performance, i.e., “instantaneous” vertical jump in condition, “subsequent” (after 1 year) change in condition, or reduction in the rate of deterioration as evidenced by a “slowing” of the deterioration curve.

5.2.1 Background to Short-term Effectiveness Evaluation

The methodology employed in evaluating the effectiveness of short-term maintenance was developed after a careful study of the process by which INDOT’s sub-districts make decisions to carry out specified type and levels of maintenance on highway pavements in their jurisdiction. A brief description of this process is provided as follows:

1. Once every fiscal year, a field team from the sub-district travels along each road within the sub-district’s jurisdiction. The location, frequency and severity of all distresses (defects, deformations, etc) are measured and recorded. During this trip, the team typically makes recommendations for the appropriate treatment needed to address specific distresses as well as the needed preventive or (corrective) treatment to correct or forestall applications of certain imminent distresses.
2. Back at the sub-district, a pavement engineer reviews the recommendations and estimates the resources needed to carry out the remedial or preventive work for budgeting.
3. The following fiscal year, maintenance is executed. Each basic management unit in the sub-district receives crew day cards, on a daily basis, that indicate the details of work to be done at each location as well as the expected accomplishments.

In order to evaluate the impact of preventive and corrective maintenance activities in the short-term, models were developed in this study to explain the magnitude of maintenance effectiveness as a response of several independent variables. The explanatory variables were attributes of the pavement sections during the year before the execution of the maintenance activity, such as pavement type, design and construction features, climatic conditions, and the level of maintenance activity in terms of a continuous expenditure variable in dollars. [Table 1](#) in Chapter 1 presents a list of the various questions that

short-term maintenance modeling seeks to answer. Maintenance expenditure in a given year depends on pavement attributes in previous year.

5.2.2 Usefulness of Short-term Models

As has been mentioned earlier, short-term maintenance models are useful to operators of maintenance management systems, because such models provide a basis to compare the effectiveness of maintenance treatments by type, by region, by material type, procedure, among other variants. The most important uses of short-term maintenance models, however, lie in their applicability to long-term evaluation of maintenance effectiveness, as follows because they enable the determination of the incremental change of pavement condition in response to the execution of a specific maintenance treatment at a certain point of time or usage. This makes it possible for PMS operators to successively adjust pavement condition to obtain extrapolated levels of pavement condition in response to future maintenance treatments.

Maintenance decision and expenditure models may be considered short-term models because they seek to predict the application of maintenance in any year given a set of explanatory variables associated with the previous year. Such models are potentially useful to operators of pavement management systems because they provide a means by which expected maintenance at a future year can be estimated. This is done by determining the expected value of annual pavement maintenance expenditure, giving due cognizance of the fact that maintenance does not occur every year, and is therefore subject to some probability of application.

5.2.3 Some Issues with Maintenance Application Models

Maintenance application is either a decision to carry out maintenance or a level of effort expended in maintenance. In the previous section, the modus operandi of Indiana's sub-district field teams in treatment selection has been described. Obviously, the decision of the inventory crew (and subsequently the execution of maintenance in the following year) at any location is influenced by type, frequency and severity of pavement surface defect, among other factors.

Response Variable for Maintenance Application models: These may take one of two forms:

- Discrete variable, i.e., maintenance decision models (where application is expressed as a probability: 1 if maintenance occurs, 0 if otherwise), or
- Continuous values, i.e., maintenance expenditure models (where application is expressed in terms of dollar expenditure, total man-hours, total energy, use, etc).

Explanatory Variables for Maintenance Expenditure models: Hypothetically, maintenance treatment expenditure could be estimated as a function of the application of each individual surface distress type that corresponds to that treatment. For instance, the probability or expenditure for crack sealing at a section could be expressed as a function of the Aggregate Cracking Index (ACI) at that section. However, in reality, such disaggregate data are not available for many years. Therefore maintenance expenditure can be estimated in one of two ways:

- As a function of an aggregate condition index
- As a function of the causal factors that determine the aggregate condition index

The latter approach involves the use of significant amount of data on climate, loading, age, subgrade and other causal factors, and may therefore be associated with a greater effort of data collection. The former approach may be described as a direct approach at estimating maintenance expenditure, while the later approach may be described as indirect. Mathematically, these are expressed as follows:

Direct functions

$$MEXP_t = f(COND_{t-1}) \dots\dots\dots (13)$$

$$\text{or } MEXP_t = f(\Delta COND_{t-2 \rightarrow t-1}) \dots\dots\dots (14)$$

Indirect function

$$MEXP_t = f(AGE_{t-1}, CLIMATE_{t-1}, PAVETYPE_{t-1}, LOADING_{t-1} \text{ etc.}) \dots\dots\dots (15)$$

Where

$MEXP_t$ = Maintenance expenditure in Year t

$COND_t$ = Pavement Condition in Year $t-1$

X_{t-1} = Value of any variable X , in Year $t-1$

It is seen that direct functions of maintenance expenditure use either the pavement condition at the previous year $COND_{t-1}$, or the change in pavement condition up to the previous year ($\Delta COND_{t-2 \rightarrow t-1}$).

There are certain issues associated with past studies on maintenance expenditure modeling that are worthy of mention. These are discussed below.

5.2.3.1 *Time Lag Effect*

From the formulations above, it is seen that indirect functions of maintenance expenditure use pavement attributes in the previous year. However, direct functions of maintenance expenditure involve the use of change in condition between the years $t-2$ and $t-1$, and not the years $t-1$ to t . However, most past studies that have estimated maintenance expenditure or decision as a direct function of the latter form (i.e., directly as a function of past change in pavement condition) have failed to consider this subtle difference, and consequently, such maintenance expenditure or decision models may have been incorrectly formulated, in some past studies, as shown in Equation (16).

$$MEXP_t = f(\Delta COND_{t-1 \rightarrow t}) \dots\dots\dots (16)$$

In the present study, care was taken to avoid errors of this nature, and any maintenance application model that was developed as functions of the change in pavement condition utilized the forms shown in Equation (14).

5.2.3.2 *The Issue of Endogeneity*

In order to estimate maintenance expenditure or decision, some past studies have combined both direct and indirect approaches in that they used both pavement condition (or change thereof) as well as pavement attributes to estimate maintenance application. Then pavement condition (or change thereof) is estimated as a function of pavement attributes. Variations of equations for this 2-stage procedure (based on whether maintenance application is expressed as an expenditure, and also on whether the pavement condition variable is condition at a given time or change in condition up to a given time) are as follows

(a) Using Maintenance Expenditure and Pavement Condition

Stage 1: Maintenance Expenditure Model

$$MEXP_t = f(COND_{t-1}, AGE_{t-1}, CLIMATE_{t-1}, PAVETYPE_{t-1}, LOADING_{t-1}, \text{etc.}) \dots\dots\dots(17)$$

Stage 2: Pavement Condition Model

$$COND_t = f(AGE_{t-1}, CLIMATE_{t-1}, PAVETYPE_{t-1}, LOADING_{t-1}, \text{etc.}) \dots\dots\dots (18)$$

(b) Using Maintenance Expenditure and Change in Pavement Condition (Maint. Effectiveness)

Stage 1: Maintenance Expenditure Model

$$MEXP_t = f(\Delta COND_{t-2 \rightarrow t-1}, AGE_{t-1}, CLIMATE_{t-1}, PAVETYPE_{t-1}, LOADING_{t-1}, \text{etc.}) \dots\dots\dots (19)$$

Stage 2: Maintenance Effectiveness (Change in Condition) Model

$$\Delta COND_{t-2 \rightarrow t-1} = f(AGE_{t-1}, CLIMATE_{t-1}, PAVETYPE_{t-1}, LOADING_{t-1}, \text{etc.}) \dots\dots\dots (20)$$

Formulations that follow any of the patterns shown above are normally carried out with caution because of the possible effect of endogeneity bias. Endogeneity bias occurs in a model system when the error terms of the equations that comprise the model system are correlated. This is due to the unobserved effects that are common to both equations. [Endogeneity bias has been known to be introduced in a model system when the measured extent of one dependent variable.](#) In the above case, pavement condition, or change in pavement condition (maintenance effectiveness) is an independent variable of the first equation, but is also the dependent variable in the second equation. The exogenous, or predetermined variables are *AGE*, *CLIMATE*, *PAVETYPE*, *LOADING*, etc., while maintenance application (expenditure or decision) and pavement condition (or change in condition) are the endogenous variables. The problem of endogeneity bias may arise due to the correlation in the error terms of the two equations.

Endogeneity bias is undesirable because the estimation of such models (where significant endogenous relationship exists between the two dependent variables) results in inconsistent and biased parameter estimates. The signs of the parameters and the magnitude of the t-statistics may be counter intuitive.

The presence of endogeneity in any 2-stage system of equations is detected by using the Hausman Specification test [Pindyck and Rubinfeld, 1991]. This involves estimating the model for the first dependent variable, and then finding its residuals. These residuals are calculated as the difference between the dependent variable and its predicted values using the model. The second stage model is then estimated with the residuals from the first model being added as an additional independent variable. Under the null hypothesis, the coefficient for the new residual term will be insignificant if endogeneity does not exist.

Endogeneity is typically addressed by using the reduced form of the equations. Because there is an exogenous variable that is excluded from each of the equations, the reduced form can be obtained by eliminating the dependent variables (in this case, maintenance application and pavement condition) from the right side of the equations. In their reduced forms, the equations express the endogenous variables in terms of exogenous variables. The excluded exogenous variables are included into the equation. The unbiased parameters can then be estimated because there is no endogenous variable on the right side of the equation.

5.2.3.3 *The Issue of Simultaneity*

Statisticians and economists have always argued that the interaction of variables in a model system has profound implications on both estimation of the model and interpretation of results. Such interaction is found in cases of autocorrelation, multicollinearity, endogeneity, etc. Yet another of such interaction, which has come under scrutiny in fairly recent times, is that of simultaneity. Simultaneity is defined as the simultaneous relationship that exists between a dependent variable and an independent variable, i.e., for example, Y is a function of X , and X is also a function of Y . The most common example of simultaneity is found in the classical economic concept of supply and demand: The supply of a commodity is a function of the demand for that commodity, and the demand is also a function of supply. This implies that either supply or demand should not be estimated individually (i.e., using the single equation approach), but jointly, or simultaneously.

In the context of pavement maintenance, the two variables in question are maintenance application (decision or expenditure) and pavement condition change. Pavement condition change can and

will be referred to as maintenance effectiveness for the rest of this section. As seen from a previous section, maintenance application and maintenance effectiveness can be expressed as follows:

$$MAINT_OCC = f(AGE, CLIMATE, LOADING, \Delta PAVE_COND) \dots\dots\dots(21)$$

$$\Delta PAVE_COND = f(MAINT_APP, AGE, CLIMATE, LOADING) \dots\dots\dots(22)$$

A review of available literature showed that each of the above equations have been estimated without the other, for various purposes in each study. Rarely have both been estimated, even separately, in a given study. Some researchers have recently argued that estimation of maintenance expenditure and maintenance effectiveness (pavement condition change) have been carried out separately and have therefore been subject to simultaneity bias [Ramaswamy and Ben-Akiva, 1990; Mohammad et al., 1997]. In other words, models that estimated pavement condition change as a function of maintenance decision or expenditure, among others, did so without recognizing that maintenance was in turn a function of pavement condition change among other variables. These researchers argue that formulating such problems as a system of simultaneous equations and estimating the model using standard econometric techniques yields more intuitive results. While it is true that maintenance effectiveness (change in pavement condition) obviously depends on maintenance application and vice versa, examining this relationship within the context of time-lag throws more light on the argument: maintenance expenditure in year $t-2$ affects maintenance effectiveness in year $t-1$, which in turn affects maintenance expenditure in year t , not year $t-2$, as illustrated in Figure 5-2. It is not certain that true simultaneity exists when maintenance expenditure in a given period, say t_N should be a function of maintenance application in a previous period, say t_M , and maintenance application at period t_M should be a function of maintenance effectiveness at period t_N as illustrated in Figure 5-3. On the other hand, it can be argued that the question of simultaneity is an estimation issue, not a data issue. In any case, this is an issue that probably requires further investigation.

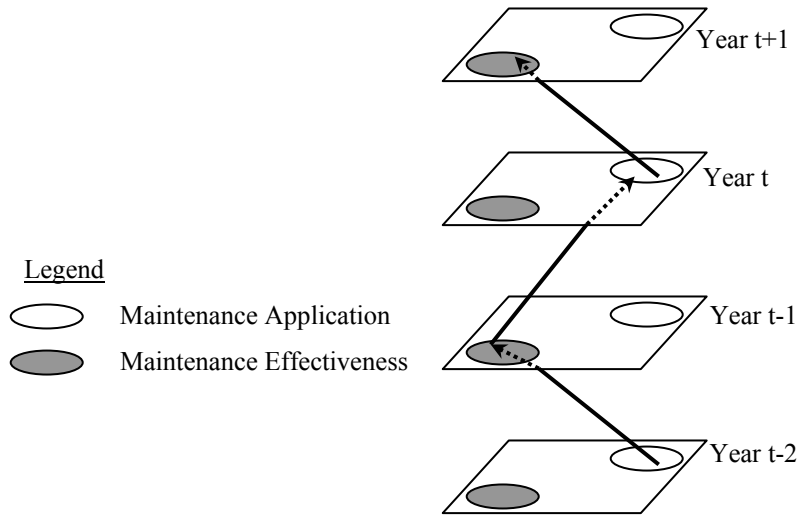


Figure 5-2: Spiral Nature of Maintenance Application/Maintenance Effectiveness Relationship

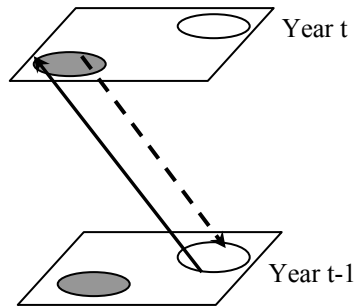


Figure 5-3: Supposed Cyclic Nature of Maintenance Application/Maintenance Effectiveness Relationship, as Probably Implied by Simultaneity Considerations

5.2.4 Annual Pavement Maintenance Expenditure Modeling Methods

An appropriate measure of the combined resources expended on pavement maintenance is the dollar amount. Pavement may be categorized by the source of work (in-house vs. by-contract), cycle of work (routine vs. periodic), or the role of the activity (corrective vs. preventive) as explained in Chapter 3.

A cursory examination of trends in levels of pavement maintenance expenditure in Indiana, from the maintenance module of the INDIPAVE 2000 database, shows that for a typical flexible pavement section, maintenance expenditure levels start from relatively very little annual amounts when the pavement is in new condition, and increases at an increasing rate due to routine maintenance carried out in-house over the years (Figure 5-4). Occasionally, seal coating or micro-surfacing is carried out either by contract or in-house. This results in a vertical jump in maintenance expenditure at that year. After a considerable period of time, i.e., in about 15 years (typically), flexible pavements receive a thin overlay. This typically results in a very significant jump in expenditure at that year. Also, in some years, some pavement sections receive no maintenance. Examination of the patterns also shows that the rate of increase in pavement expenditure from one year to the next is generally higher for old pavements than for new pavements. However, in the years following moderate preventive maintenance (seal coating), major preventive maintenance (thin overlay), or an extensive amount of corrective maintenance, annual maintenance levels on old pavements are typically reduced to levels similar to those for young pavements, but the level of service is not sustained for a period as long as that for young pavements.

The situation for rigid pavements is somewhat similar. However, such pavements have relatively little maintenance in their early years compared to flexible pavements. A plot of their annual maintenance expenditure, in their early years, would yield a gentle line or curve, and relatively minor treatments on such pavements are typically carried out in-house. As such pavements age, they require treatments that are typically carried out on contract, such as undersealing, retrofitting, and slab replacements. The relatively high costs of carrying out any work on contract translates to high maintenance costs for such pavements, and it is therefore not surprising that some rigid pavements, in their advanced years, are associated with higher levels of pavement maintenance than their flexible counterparts.

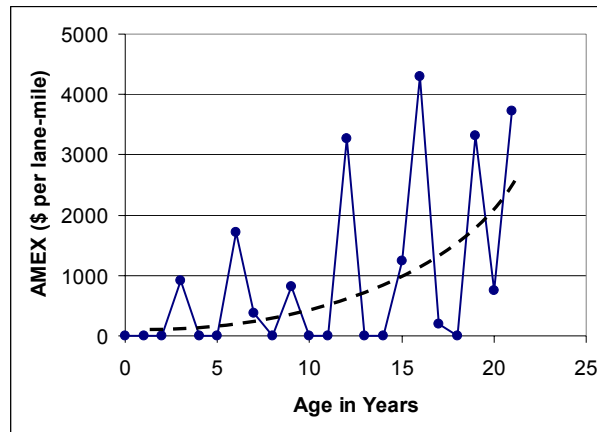


Figure 5-4: Typical Pattern of Annual Pavement Maintenance Expenditure

In Figure 5-4, the continuous saw-toothed line represents the hypothetical annual maintenance expenditure pattern for a given pavement section. In averaging or modeling annual pavement maintenance expenditure, several pavements in a given category are considered, and the overlapping effect of their individual expenditure trends yields a curvilinear smooth pattern (the broken line) that shows annual maintenance increasing an increasing rate with time.

Short-term maintenance expenditure models estimate annual pavement maintenance that a given pavement receives in any given point in time, or after a given level of accumulated usage, either in terms of time or accumulated loading. As discussed in Chapter 3, this is typically done in one of three ways:

- i) Averaging: Using the simple average of annual maintenance expenditure values that all pavements in a certain category and of a certain usage level (age group) receive, or
- ii) Modeling with averages: Calculating the average annual maintenance expenditure incurred by each pavement section over a period, and modeling these average values as a function of pavement attributes (to yield average annual maintenance expenditure models (AAMEX)), or
- iii) Modeling with individual values: Using the individual annual maintenance expenditures of each pavement section at each year, (rather than the average of such values) as the response variable in a model to estimate such expenditure as a function of pavement attributes (this yields annual maintenance expenditure models (AMEX)).

Using the first approach (i) identified above, pavements were categorized according to their surface type, functional class, region, and age group, and the average values of their annual maintenance expenditure in each category were determined (these are only approximate values of annual maintenance expenditure that may be used for sketch planning (see Section 8.2 of Chapter 8). Table 5-1 below further illustrates the structural difference in the second and third approaches:

Table 5-1: Dataset Structure for Pavement Maintenance Expenditure Modeling

Year Section	Year Y_1	Year Y_2	Year Y_K	MEAN
Section S_1	$AMEX_{S_1, Y_1}$	$AMEX_{S_1, Y_2}$	$AMEX_{S_1, Y_K}$	$AAMEX_{S_1}$
Section S_2	$AMEX_{S_2, Y_1}$	$AMEX_{S_2, Y_2}$	$AMEX_{S_2, Y_K}$	$AAMEX_{S_2}$
⋮	⋮	⋮	⋮
Section S_N	$AMEX_{S_N, Y_1}$	$AMEX_{S_N, Y_2}$	$AMEX_{S_N, Y_K}$	$AAMEX_{S_N}$

In the second approach of maintenance expenditure modeling, the average values of annual pavement maintenance expenditure ($AMEX$) for each pavement section (i.e., $AAMEX_{S_1}$, $AAMEX_{S_2}$, ..., $AAMEX_{S_N}$) are used as the response variables. The dataset size is N , and the model obtained can be described as an *average* annual maintenance expenditure ($AAMEX$) model. In the third approach of maintenance expenditure modeling, the individual values of $AMEX$ for each pavement section in each year (i.e., $AMEX_{S_1, Y_1}$, $AMEX_{S_1, Y_2}$, ..., $AMEX_{S_N, Y_K}$) are directly used as the response variables. No average values of maintenance expenditure are used. The dataset size is $N*K$, and the model obtained can be described as an annual maintenance expenditure ($AMEX$) model. For $AAMEX$ modeling, no data-point has a value of zero, and traditional least squares approaches maybe used. However, for $AMEX$ modeling, some data-points have zero value (as maintenance is not carried out in certain years), and therefore *TOBIT* models, which are associated with probabilities of maintenance, are considered more appropriate. For the present study, the $AAMEX$ method was used to estimate average values of pavement maintenance expenditure at any year given other attributes of the pavement.

5.2.4.1 *Methods of Data Organization for AAMEX Modeling*

For each pavement section in the database, the amounts of funds expended on crack sealing, shallow patching, seal coating, thin overlays, and all other pavement corrective and preventive maintenance carried out either in-house or by contract in each year, were summed up. The constant dollar values (1995\$) of all maintenance carried out on each pavement section, between 1991 and 1999 fiscal years, were computed, except for 1998 fiscal year, for which no data was available at the time of the analysis. The total dollar amount of all treatments on each pavement section at each year was determined as the annual maintenance expenditure (*AMEX*) for that pavement section at that year, and the average value was computed (*AAMEX*). This was used as the response variable in the model that estimated *AAMEX* as a function of pavement attributes such as region, functional class, age, and pavement type. For all pavement sections, age zero was taken as the year of reconstruction or rehabilitation.

As an initial step in *AAMEX* modeling, it was found necessary to carry out a simple descriptive analysis of the pavement annual maintenance expenditure data, with the objective of identifying any patterns that may be worthy of notice, to provide illuminating evidence of a-priori analysis results, or to assist in formulating explanatory variables in an appropriate manner.

The descriptive analysis involved a simple averaging of maintenance expenditure for each category of pavements. Categories were pavement type, age, location (region), and functional class. These categories were selected for the descriptive analysis because from the literature review, such pavement attributes are most likely to affect levels of pavement maintenance.

5.2.4.2 *Adjustments to Maintenance Expenditure for Inflation*

Due to the effect of economic inflation on the costs of pavement maintenance material, labor and equipment use from year to year, it is expected that the cost of a given maintenance treatment will not remain constant but will increase, even if the levels of such resources in a given situation remain the same. Similarly, the unit costs of pavement reconstruction and rehabilitation (per lane-mile) increase with time. The Federal Highway Administration (FHWA) regularly publishes updates of highway construction indices (for construction and rehabilitation) and consumer price index (for maintenance activities), as provided in

Appendix G. For the purposes of the present study, all costs indices were related to their 1995 values, and the resulting price indices are presented as [Table 5-2](#) below.

Table 5-2: Highway Price Trends and Consumer Price Index Based on 1995 Index Year
[Source of Original Data: Highway Statistics [FHWA (2), 1999]

INDEX	1983	1984	1985	1986	1987	1988	1989	1990	1991
Construction	71.9	76.0	83.7	82.9	82.0	87.4	88.4	89.0	88.2
Maintenance	65.4	68.2	70.6	71.9	74.5	77.6	81.4	85.8	89.3

INDEX	1992	1993	1994	1995	1996	1997	1998	1999
Construction	86.2	88.8	94.4	100.0	98.6	107.1	104.1	112.0
Maintenance	92.0	94.8	97.2	100.0	102.9	105.3	106.9	109.3

5.2.5 Short-term Maintenance Effectiveness Models- Methodology

5.2.5.1 The Process for Evaluating Maintenance Treatment Effectiveness in the Short Term

As shown in Figure 5-5, the three basic sequential issues often associated with effectiveness evaluation of any maintenance treatment in the short-term are as follows:

- (a) How should effectiveness be measured, and in what terms?
- (b) On what grounds can the maintenance treatment be deemed effective?
- (c) If the treatment is found to be effective for several pavements, can such effectiveness be modeled as a function of pavement and treatment attributes?

At the first stage, an appropriate measure of maintenance effectiveness is selected. Then a measure of pavement performance or condition (MOP) such as PCR or PSI, which best reflects the efficacy of the treatment is selected, and the MOE values are calculated in terms of the selected measure of pavement performance, for each pavement section under investigation. The next step assesses whether the treatment

was effective, using values of the computed measure of maintenance effectiveness (MOE). This may be done by testing the null hypothesis that the mean value of the MOE is less or equal to zero (the treatment was not effective) versus the alternate hypothesis that the mean exceeds zero (the treatment was effective) at a specified level of significance. After maintenance effectiveness has been thus confirmed, the third step would be to attempt model development for estimating maintenance effectiveness as a function of treatment attributes such as pavement and treatment characteristics (with the MOE values representing the model's dependent variable). The second and third stages typically utilize data from several pavement sections that received the same maintenance treatment. This study investigates the effectiveness of various maintenance treatments along a sequence of activities consistent with these three stages. Details and results of each stage are discussed below.

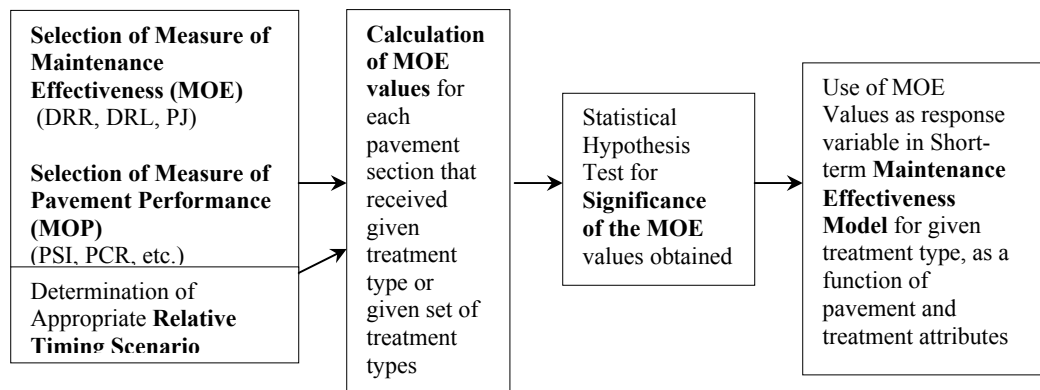


Figure 5-5: Sequence for Short-term Maintenance Effectiveness Evaluation

5.2.5.2 Measures of Pavement Performance/Condition

Maintenance effectiveness, or deterioration reduction, may be viewed as the increase in “positive” service attributes (or reduction in “negative” attributes) of an infrastructure system in response to treatment. In the context of highway pavements, such attributes may be improved surface condition (such as Present Serviceability Index (PSI) and Pavement Condition Rating (PCR)) or decreased surface roughness (Roughness Number (RN), International Roughness Index (IRI), etc). It is vital to consider what the treatment in question is meant to achieve in terms of pavement performance or condition, and how such

performance/condition is measured. In the context of seal coating for instance, the treatment is intended to correct extensive cracking, spalling, shallow surface failures, loss of skid resistance, and raveling, to name a few. Therefore, the measure of pavement performance (MOP) that should ideally be used to determine the effectiveness of this maintenance treatment should be an index that directly captures the extent and severity of such defects. One such index is the Pavement Condition Rating (PCR). However, given the lack of PCR data at the time of study, PSI was used in its stead. It is worth mentioning that the PSI is more directly associated with ride quality and indirectly with pavement surface defects. Therefore PSI was used with the assumption that the surface defects addressed by seal coating are ultimately manifested in ride quality.

5.2.5.3 *Measures of Treatment Effectiveness*

Having chosen an appropriate measure of pavement performance, it is necessary to determine the measure of maintenance effectiveness, in terms of the former. An adjustment in pavement condition due to the application of maintenance may take one of two forms: a modest improvement in current pavement condition measured instantaneously or after a finite time period or a reduction in the rate of deterioration subsequent to maintenance (Lytton 1987; Markow 1991; Smith et al. 1993). With regard to the number of monitoring periods used in the computation of deterioration reduction, there are many ways in which such reduction could be measured. The simplest is to use measurements taken at two points in time: one just before maintenance and another just after maintenance. The result of such computation would be an instantaneous performance jump due to maintenance. Another way is to use two measurements: one of which is taken at a specified time (say, 1 year) before maintenance and the other just after maintenance; or one in which measurement was taken just before maintenance and the other taken a specified time after maintenance. Yet another way is to use three measurements: one taken a specified time (say, 1 year) before maintenance, the other just before maintenance, and the third measurement a specified time after maintenance. The third method enables the evaluation of maintenance effectiveness not as a difference in deterioration values, but in terms of a reduction in the deterioration *rate*. From the discussion above, three measures of deterioration reduction are identified as possible measures that could be used to assess the

short-term effectiveness of seal coating:

- Performance Jump (PJ)
- Deterioration Reduction Level (DRL)
- Deterioration Rate Reduction (DRR)

5.2.5.4 Computation of Seal Coating Effectiveness Values

For each maintenance treatment, using annual condition data for pavement sections that received seal coating in the 1995 and 1996 fiscal years, the effectiveness of this maintenance treatment, in terms of performance jump and deterioration rate reduction, was determined for each pavement section. For a given treatment, only pavements that received that treatment and little or no other treatment were selected.

5.2.5.5 Statistical Test of Significance of Treatment Effectiveness Values

The statistical significance of the estimated Performance Jump (PJ) and deterioration rate reduction (DRR) values for each maintenance treatment were tested at a 95% level of confidence. This was done to investigate whether the effectiveness of the treatment received by the pavement sections are significantly greater than zero. As the PJ and DRR values are derived from PSI values (which are, in turn, average values of pavement condition over a stretch of highway pavement), the distribution of the PJ and DRR values can be considered as sampling distributions of means. Therefore, the formulated hypothesis for Performance Jump was therefore as follows:

$H_0: \mu_{PJ} \leq 0$ (the seal coating treatments were not effective)

$H_1: \mu_{PJ} > 0$ (the seal coating treatments were effective)

This is a 1-sided hypothesis test with the “rejection region” in the upper tail. Therefore, the critical value of the test statistic is $Z_\alpha = Z_{0.05} = 1.645$. The calculated value of the test statistic is given by:

$$Z^* = (\mu_{PJ} - 0) / (\sigma / \sqrt{n})$$

Where σ is the standard deviation, and n is the sample size.

5.2.5.6 Treatment Effectiveness Models

Preliminary scatter plots of treatment effectiveness were drawn in a bid to unveil any glaring trends in such effectiveness over the given ranges of explanatory variables. Besides initial pavement condition, other explanatory variables exhibited relatively little variation with respect to changes in response variable (i.e., treatment effectiveness). Linear, intrinsically linear, and non-linear functional forms were investigated for developing the treatment effectiveness models.

5.2.5.6 (a) Definition of Response Variables

Deterioration reduction is the increase in “positive” service attributes (or reduction in “negative” attributes) of an infrastructure system. In the context of highway pavements, this may be in the form of improved surface condition (PSI, PQI, PCR, etc) or decreased surface roughness (RN, IRI, etc). With regard to the number of monitoring periods used in the computation of deterioration reduction, there are many ways in which such reduction could be measured. The simplest is to use measurements taken at two points in time, one just before maintenance and another just after maintenance. The result of such computation would give the instantaneous performance jump due to maintenance. Another way is to use two measurements, one of which was taken a year before maintenance, and the other just after maintenance or one in which measurement was taken just before maintenance, and the other taken a year after maintenance. If data is available, three measurements could be used: one a year before maintenance, the other just before or just after maintenance, and the third measurement a year after maintenance. The second method enables the computation of the reduction not in the values of deterioration, but the reduction of the *rate* of deterioration. Figure 5-6 illustrates these measures of short-term maintenance effectiveness.

The point A corresponds to the state of the pavement a year before maintenance, while point D is the state of the pavement just before maintenance is carried out. Point F is the state of the pavement just after maintenance, while point E is the state of the pavement a year after maintenance. Points W and Z are included for the sake of geometrical construction. C_i and t_i represent the condition of the pavement and the time of monitoring measurement, respectively, corresponding to point i . Table 5-3 provides a description of various measures of maintenance effectiveness in the short-term that can be inferred from the above figure, and identifies previous researchers that have implicitly used some of these measures in their analyses.

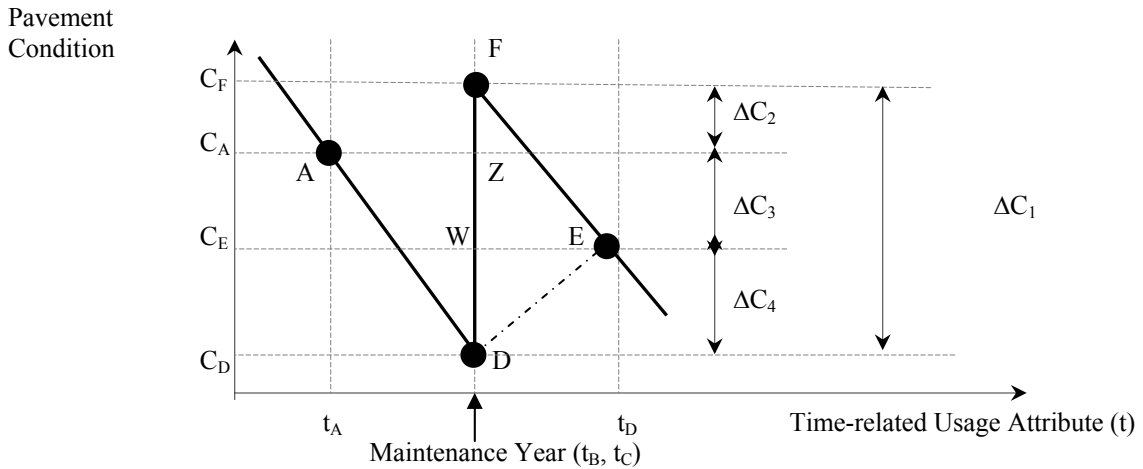


Figure 5-6: Various Measures of Short-term Maintenance Effectiveness (Deterioration Reduction)

Table 5-3: Various Measures of Short-term Maintenance Effectiveness (Deterioration Reduction)

Deterioration Reduction Measure	Computation	Synonymous Names/Descriptions	Comments	References
ΔC_1	$C_F - C_D$	-Instantaneous Deterioration Reduction -Performance Jump (CJ) -Vertical Elevation in Condition	Ideal but impractical	[Colucci et al., 1985]
ΔC_2	$C_F - C_A$	-Deterioration Reduction Level (DRL) -Decrease in Roughness -Increase in CSI	Misses effectiveness denoted by ZD	[Li et al., 2000]
ΔC_3	$C_A - C_E$	Same as for ΔC_2	Avoids critical timing issue	Mohammad et al., 1997; Fwa et al., 1987]
ΔC_4	$C_E - C_D$	Same as for ΔC_2	Misses effectiveness denoted by DW	Madanat et al., 1995]
$\Delta Rate$	$(C_E - C_D) - (C_D - C_A)$	Deterioration Rate Reduction	Sometimes D is not known. Data intensive	

The literature review (Chapter 3) showed that an adjustment in pavement condition due to the execution of maintenance may take one of two forms: (i) a reduction in the rate of future deterioration, or (ii) a modest improvement in the measure of current pavement condition [Lytton, 1988; Markow,1991]. It has been indicated that not only could either of these two phenomena be experienced, but that both could be experienced simultaneously [Mamlouk and Zaniewski, 1998] as illustrated in Figure 5-7 (c).

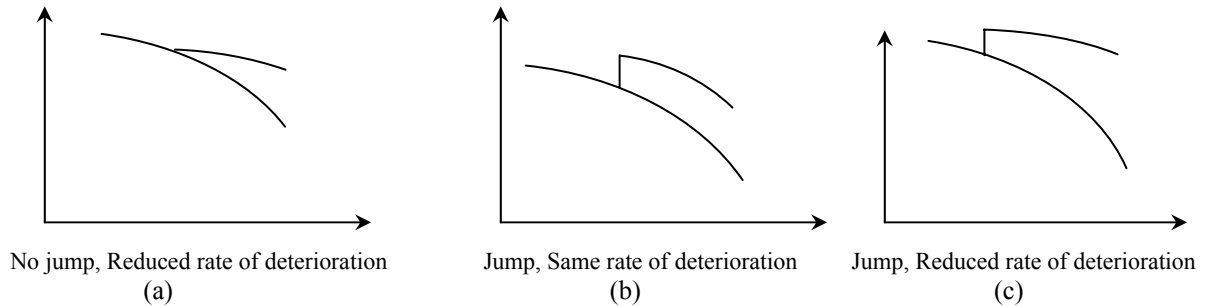


Figure 5-7. Hypothetical Performance Jump or/and Trend after Maintenance

Against the background of the foregoing discussion, three measures of deterioration reduction have been used or at least mentioned in past studies, as follows:

- Deterioration Reduction Level (DRL)
- Performance Jump (PJ)
- Deterioration Rate Reduction (DRR)

Finer details of each measure, as well as the advantages and disadvantages associated with the use of each measure of deterioration reduction are discussed in the next section.

5.2.5.6 (b) Description of Three Measures of Deterioration Reduction

Deterioration Reduction Level (DRL)

Sometimes referred to as the Subsequent Change in Deterioration or simply the Change in Deterioration (e.g., Δ IRI, Δ PSI, $\%$ Δ IRI, etc.), Deterioration Reduction Level is the reduction of

infrastructure deterioration between two consecutive points in time, typically 1 year. There are typically two ways in which deterioration reduction levels have been computed:

- (i) Difference in deterioration one year before maintenance and that just after maintenance, as illustrated as ΔC_2 in Figure 5-6.
- (ii) Difference in deterioration just before maintenance and that 1-year after maintenance, as illustrated as ΔC_3 in Figure 5-6.

As has been pointed out in Table 5-3, both these measures of deterioration reduction levels miss a vital component of that measure because in (i) the condition of the pavement either just before maintenance is not considered, and in (ii) the condition of the pavement just after maintenance is not accounted for. The use of deterioration reduction levels is common among most literature on this subject [Fwa et al., 1987; Mohammad et al., 1987; Madanat and Mishalani, 1995]. Furthermore, non-consideration of the relative timing between maintenance and deterioration measurements contributes greatly to incorrect conclusions about maintenance effectiveness if the DRL measure is used. This is discussed in detail in subsequent sections.

Performance Jump

Performance jump (PJ) is simply the vertical, or instantaneous elevation in the performance or condition of a pavement due to maintenance. This concept has often been the subject of discussion (Lytton, 1987), but has seen relatively little application [Colucci-Rios and Sinha, 1985; Rajagopal and George, 1991]. By providing a measure that involves just-before and just-after measures of deterioration, PJ avoids the time-related pitfalls of the DRL measure, and therefore offers what is probably the best means to assess maintenance effectiveness in the short-term. However, because agencies typically do not carry out deterioration measurements before and after maintenance, it is hard to obtain data for PJ computation. It was therefore necessary as part of the present study, to derive geometrical relationships between PJ and the other measures of deterioration so that PJ can be found given the other measures (and a few assumptions). This is presented in subsequent sections of this chapter. Obviously, the smaller the duration of a given maintenance

activity and the smaller the time or usage interval between deterioration measurements and maintenance, the more accurate the value obtained for performance jump.

Deterioration Rate Reduction

Like Performance Jump, Deterioration Rate Reduction has been mentioned at least twice in literature, but discussion on this measure has been only conceptual [Lytton, 1987; Markow, 1991; Markow, 1994]. Deterioration Rate Reduction is the difference in the slope of the deterioration curve before-maintenance and the after-maintenance. It is worth noting that the DRR concept is more readily appreciated and applicable to a performance curve where all kinks due to performance jumps have been smoothed out to yield a continuous line.

5.2.5.6 (c) Maintenance and Deterioration Monitoring Relative Timing

In the hypothetical situation (Figure 5-8), just-before-maintenance and just-after-maintenance deterioration measurements are carried out. This means that for each year of maintenance of a pavement section, two sets of measurements are taken to monitor deterioration, while only one set is taken for non-maintenance years. In such an ideal case, all 3 measures of deterioration can be computed with ease.

In the field situation, however, agencies do not have the resources to carry out two sets of deterioration measurements in one year (Figure 5-9). Therefore years of maintenance have only one set of deterioration measurements. The question that arises is two-fold: (i) was the maintenance activity (or the bulk of it) carried out before the conduction of the deterioration measurements (ii) if deterioration measurements were conducted before maintenance, how can the value of deterioration measurement after maintenance be obtained?

Obviously, computing maintenance effectiveness on the assumption that maintenance was carried out before the deterioration measurement if in actual fact the contrary was the case, would lead to underestimating the reduction on deterioration, and vice versa. In cases where the relatively high levels of maintenance, such as thin overlays, were carried out, such mistakes can lead to the erroneous conclusion that the maintenance activity resulted in little or no maintenance effectiveness (deterioration reduction). It is

therefore imperative that the relative timing of the two activities (maintenance occurrence and deterioration monitoring) is known, so that the effectiveness expressions can be correctly formulated.

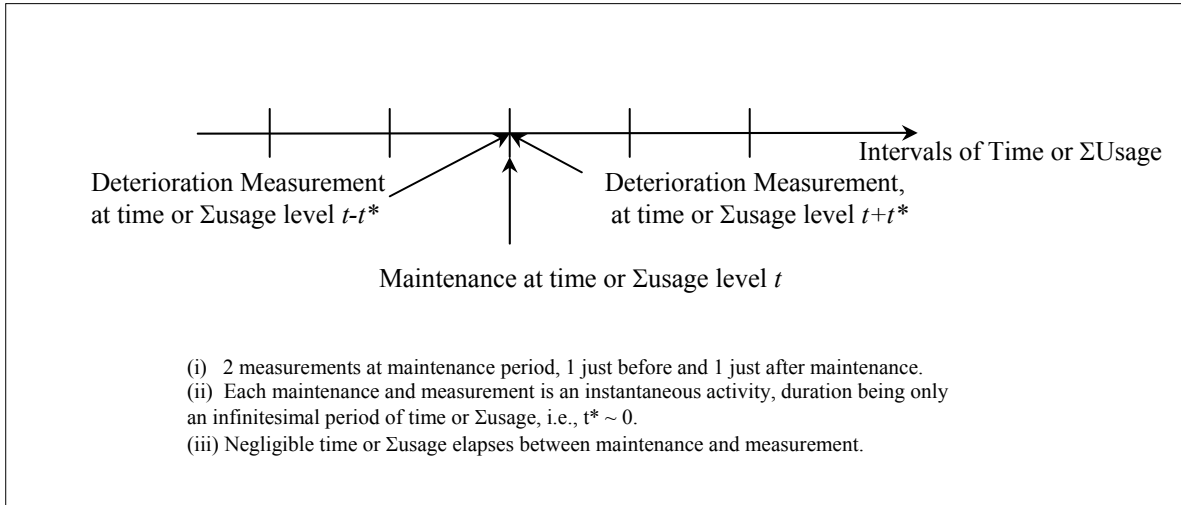


Figure 5-8: Relative Timing of Maintenance and Monitoring: The Hypothetical Case

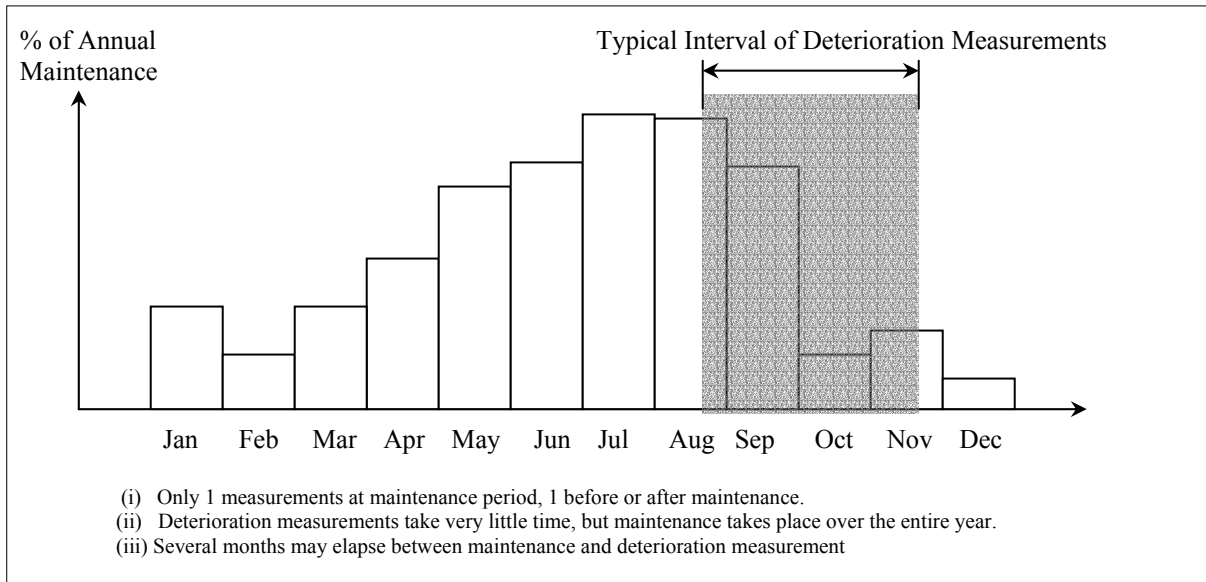


Figure 5-9: The Actual Case for Relative Timing of Maintenance and Monitoring

Effect of Incorrect Consideration of Relative Timing of Maintenance and Deterioration Measurements: In analyzing the effectiveness of maintenance given deterioration data for two or three years in the vicinity of time of the maintenance treatment, a careful investigation should be carried out to ascertain whether the treatment preceded the deterioration measurement or whether it was after. Incorrect consideration of timing can result in conclusions that typically underestimate maintenance effectiveness, or even result in an inference that is contrary to the real situation.

In order to illustrate the effect of incorrect consideration of relative timing of maintenance and deterioration measurements, consider a state road that received chip sealing treatment in April 1995. Pavement deterioration measurements were carried out on this stretch of road in the summer of 1994, 1995, and 1996. The results were 2.71, 2.88, and 2.59 PSI units for 1994, 1995, and 1996 respectively. Two methods (regarding relative timing) for calculation of deterioration reduction are shown in the example below. The first method is associated with an implicit, albeit apparently invalid, assumption that maintenance was carried out after the deterioration measurement in that year.

Methods of Computing Change in deterioration due to Maintenance in 1995:

$$(a) \text{ Traditional computation: change in deterioration} = \text{PSI}_{1996} - \text{PSI}_{1995} = 2.59 - 2.68 = -0.09$$

The traditional computation (Figure 5-8) estimates that the increase in the level of deterioration after seal coating was 0.09.

$$(b) \text{ Proposed computation (Figure 5-9): change in deterioration} = \text{PSI}_{1996} - \text{PSI}_{1995, \text{BM}}$$

But PSI just before maintenance in 1995, $\text{PSI}_{1995, \text{BM}}$, can be estimated as follows:

$$\text{PSI}_{1995, \text{BM}} = \text{PSI}_{1994} - ((\text{PSI}_{1995, \text{AM}} - \text{PSI}_{1996})/1) * 1 = 2.62$$

$$\text{PSI}_{1994} + (1 * 1 * 2.68 - 2.59)/1$$

$$\text{Therefore, change in deterioration} = 2.59 - 2.62 = -0.03$$

The proposed computation estimates that the increase in the level of deterioration a year after maintenance is 0.03. Therefore the use of the traditional method of DRL computation, in this case, yields an underestimation of maintenance effectiveness. The proposed computation is different from the traditional computation because it considers that fact that the seal coating activity in that year was carried

out before the conduction of the deterioration measurement. This is often the case in Indiana, where for any given year, maintenance (or the bulk thereof) for a given fiscal year, is carried out before roughness measurements for the corresponding calendar year. The illustration shows that maintenance effectiveness may be incorrectly computed if the relative timing between maintenance and monitoring is not considered.

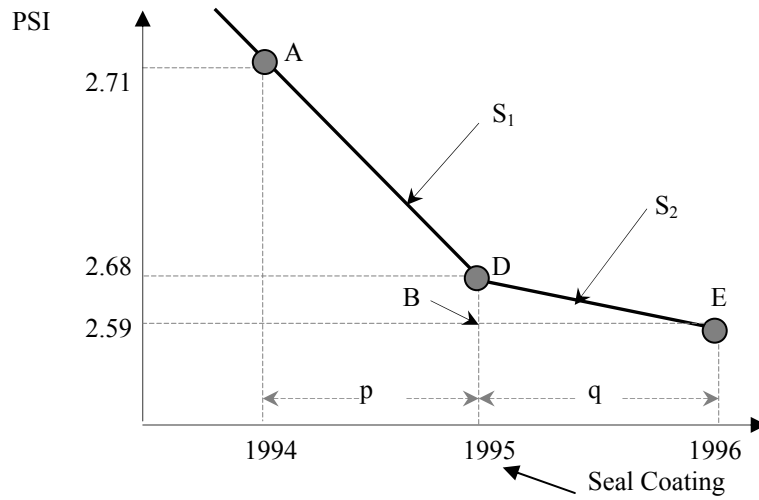


Figure 5-10: Implication of Using the Traditional Computation

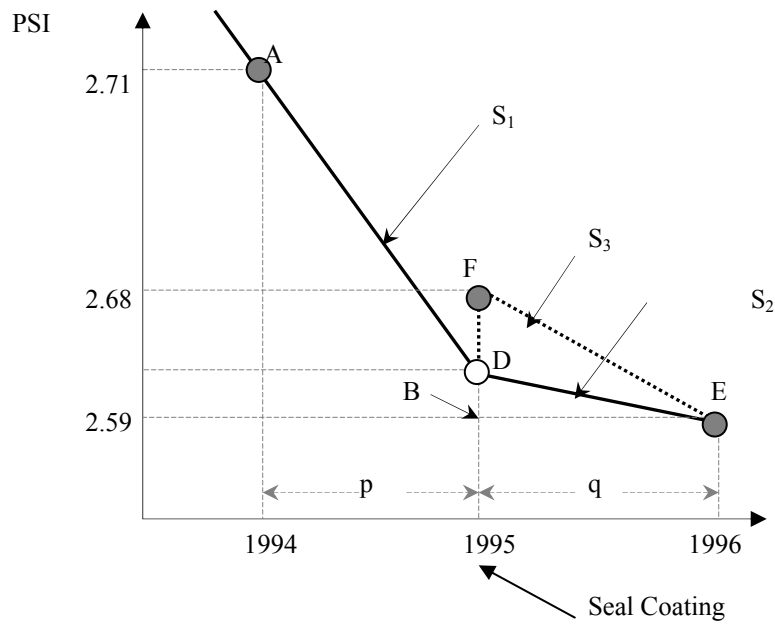


Figure 5-11: Correct Computation based on Actual Timing Situation

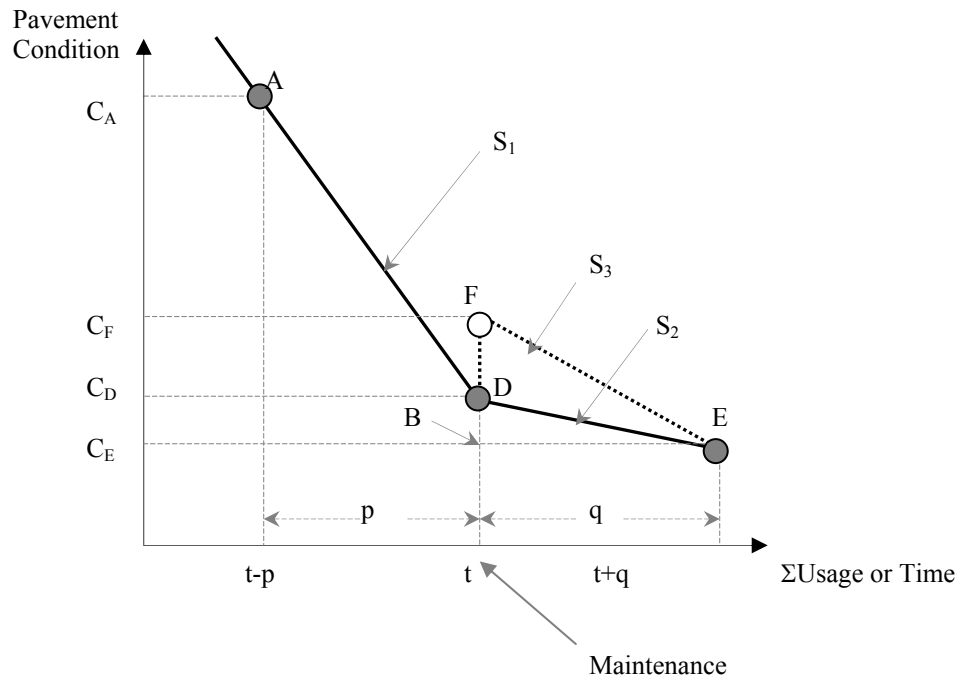
The literature review showed that most past studies have calculated change in deterioration using method (a). This would have been appropriate if pavement sections typically receive the bulk of their maintenance activities after the deterioration measurements. Often however, this is not so. Most pavement sections, for a fiscal year, receive the bulk of their maintenance before the conduction of roughness and other surface condition measurements. Indeed, by the time such measurements begin, maintenance for that fiscal year is over. It is therefore obvious that past studies, in ignoring the relative timing of maintenance and deterioration measurements, have grossly underestimated the effectiveness of maintenance in some cases, and may have even concluded that maintenance was not effective whereas it actually was.

The next section presents derivations of mathematical expressions for the three measures of short-term maintenance effectiveness, for each of the two relative timing scenarios, and also presents a derivation of the mathematical relationships between each pair of the three measures.

5.2.5.6 (d) Derivation of Expressions for Measures of Deterioration Reduction

(i) Relative Timing Scenario 1: Maintenance is carried out after Deterioration Measurement

This scenario considers the case where the bulk of maintenance activities are executed after the deterioration measurements, as illustrated in [Figure 5-10](#).



Legend

- Points at which deterioration measurement is taken
- Points at which deterioration measurement is not taken

The Y-axis represents the condition (or level of deterioration) of the infrastructure

A: Point of deterioration curve at a period p before maintenance

D: Point of deterioration curve just before the execution of maintenance

E: Point of deterioration curve at a period q after maintenance

F: A virtual point representing level of deterioration just after maintenance

B: A virtual point for purposes of geometry

C_A , C_F , C_D and C_E are the levels of deterioration that correspond to the above points.

S_1 : Slope of the deterioration curve before maintenance

S_2 : Slope of the deterioration curve after maintenance (is virtual because F is unknown)

S_3 : Slope of the deterioration curve before maintenance (is real because D and E are known).

k = Ratio of slope after maintenance to slope before maintenance = S_2/S_1

Figure 5-12: Relative Timing Scenario 1: Maintenance after Monitoring

(a) Vertical elevation in Performance, or Performance Jump(PJ), due to maintenance at year t is represented

by the line DF.

$$PJ = C_F - C_D$$

$$\text{But } C_F = C_E + S_3 * q$$

= $C_E + k * S_I * q$ where k is factor by which the slope changes after maintenance

$$\begin{aligned} \Rightarrow PJ &= C_E + S_3 * q - C_D = C_E + k * S_I * q - C_D = C_E + k * [(C_A - C_D) / p] * q - C_D \\ &= C_E + (k * q / p) (C_A - C_D) - C_D \dots\dots\dots (23) \end{aligned}$$

If slope after maintenance is same as same before maintenance, then $k=1$, and

$$PJ = C_E + (q/p)(C_A - C_D) - C_D \dots\dots\dots (24)$$

Also, if monitoring period after maintenance is same as same that before maintenance,

then $p = q = 1$, and

$$PJ = C_E + (C_A - C_D) - C_D = C_E + C_A - 2C_D \dots\dots\dots (25)$$

(b) Deterioration Reduction level (DRL) or subsequent change in deterioration due to maintenance at year t .

$$DRL = C_E - C_D \dots\dots\dots (26)$$

Both C_E and C_D are known.

(c) Deterioration Rate Reduction (DRR) due to maintenance at year t .

$DRR =$ deterioration rate after maintenance – deterioration rate before maintenance

$$DRR = \frac{C_E - C_D}{(t + q) - t} - \frac{C_D - C_A}{t - (t - p)} = \frac{C_E - C_D}{q} - \frac{C_D - C_A}{p} \dots\dots\dots (27)$$

where symbols have their usual meanings.

Notice that this expression is independent of k (the ratio of after-maintenance and before-maintenance slopes) also referred to as the “ k ” factor. This is because DRR refers to slope reduction using a continuous smoothed out performance curve that is free of any kinks due to performance jumps. Therefore DRR incorporates a virtual, rather than real slope after maintenance, and therefore the use of k cannot yield the desired result. This means that unlike the other measures of deterioration reduction, DRR is independent

of k , and therefore obviates the problems due to non-availability of k or possible errors inherent with the assumption of k factors for each maintenance treatment.

When $p=1$ and $q=1$,

$$DRR = (C_E - C_D) - (C_D - C_A) = C_E + C_A - 2C_D \dots\dots\dots(28)$$

It can be noticed that when $k=1$, Equation 23 is the same as the Equation 28.

(d) Relationships Between the three Deterioration Reduction Measures, Relative Timing Scenario 2

Deterioration Reduction Level and Deterioration Rate Reduction:

Deterioration Reduction Level is given by:

$$DRL = C_E - C_D \dots\dots\dots(29)$$

Also, Deterioration Rate Reduction is given by:

$$DRR = \frac{C_E - C_D}{q} - \frac{C_D - C_A}{p} = \frac{DRL}{q} - \frac{(C_D - C_A)}{p} \dots\dots\dots(30)$$

Making DRL the subject of the equation yields

$$DRL = q[DRR + \frac{(C_D - C_A)}{p}] \dots\dots\dots(31)$$

When $p=1$ and $q=1$,

$$DRL = DRR + (C_D - C_A) \dots\dots\dots(32)$$

$$DRR = DRL - C_D - C_A \dots \dots \dots (33)$$

Also,

Performance Jump and Deterioration Reduction Rate:

Performance Jump is given as follows:

$$PJ = C_E + \frac{k * q}{p} (C_A - C_D) - C_D$$

$$\frac{p * (PJ - C_E + C_D)}{k * q} = C_A - C_D \dots \dots \dots (34)$$

$$DRR = \frac{C_E - C_D}{q} - \frac{C_D - C_A}{p}$$

$$C_D - C_A = p \left[\frac{(C_E - C_D)}{q} - DRR \right]$$

$$C_D - C_A = p \left[\frac{(C_E - C_D)}{q} - DRR \right].$$

$$C_A - C_D = DRR - p \left[\frac{(C_E - C_D)}{q} \right] \dots \dots \dots (35)$$

From the Equations (34) and (35),

$$\frac{p(PJ - C_E + C_D)}{k * q} = p[DRR - \frac{(C_E - C_D)}{q}]$$

$$PJ = q * k * DRR - \frac{k}{q}(C_e - C_D) + (C_e - C_D)$$

$$PJ = q * k * DRR - (\frac{k}{q} - 1)(C_e - C_D) \dots \dots \dots (36)$$

$$PJ = q * DRR - (\frac{1}{q} - 1)(C_e - C_D) \dots \dots \dots (37)$$

When $q = 1$, $PJ = DRR$

Deterioration Reduction Level and Performance Jump:

From Equations (29) and (34),

$$DRL = C_E - C_D$$

$$PJ = C_E + \frac{k * q}{p}(C_A - C_D) - C_D$$

$$PJ = (C_E - C_D) + \frac{k * q}{p}(C_A - C_D)$$

$$PJ = DRL + \frac{k * q}{p}(C_A - C_D) \dots \dots \dots (38)$$

Also,

$$DRL = PJ - \frac{k * q}{p} (C_A - C_D) \dots \dots \dots (39)$$

When $k=1$

$$PJ = DRL + \frac{q}{p} (C_A - C_D) \dots \dots \dots (40)$$

$$DRL = PJ - \frac{q}{p} (C_A - C_D) \dots \dots \dots (41)$$

When $p=1$ and $q=1$,

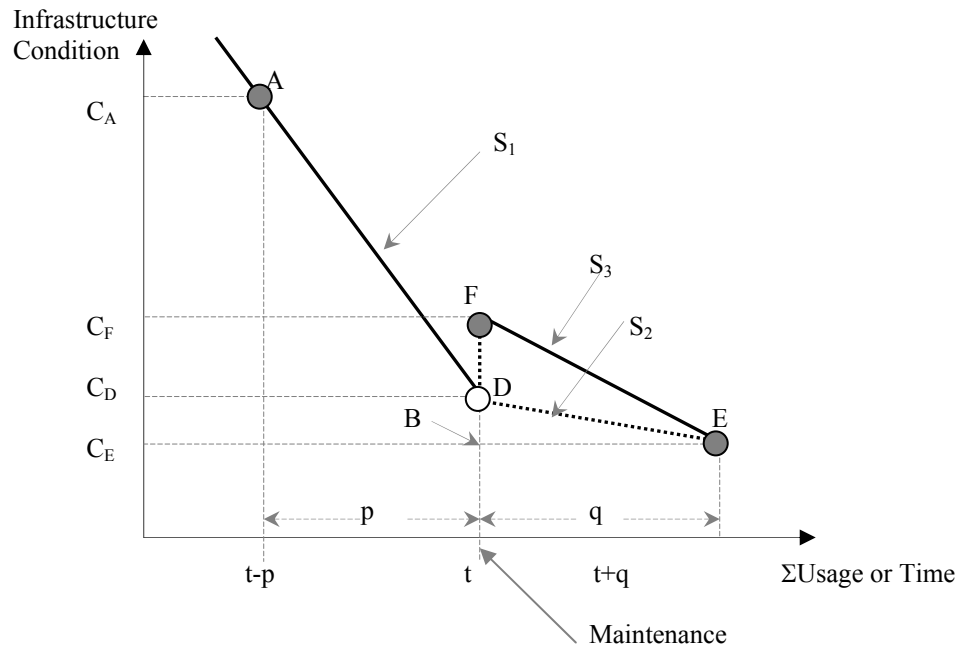
$$PJ = DRL + C_A - C_D \dots \dots \dots (42)$$

and

$$DRL = PJ - C_A + C_D \dots \dots \dots (43)$$

(ii) Relative Timing Scenario 2: Maintenance is carried out before Deterioration Measurement

This scenario considers the case where the bulk of maintenance activities is executed before the deterioration monitoring measurement, as illustrated as [Figure 5-13](#).

Legend

- Points at which deterioration measurement is known
- Points at which deterioration measurement is known

The Y-axis represents the condition (or level of deterioration) of the infrastructure

A: Point of deterioration curve at a period p before maintenance

D: Point of deterioration curve just before the execution of maintenance

E: Point of deterioration curve at a period q after maintenance

F: A virtual point representing level of deterioration just after maintenance

B: A virtual point for purposes of geometry

C_A , C_F , C_D and C_E are the levels of deterioration that correspond to the above points.

S_1 : Slope of the deterioration curve before maintenance

S_2 : Slope of the deterioration curve after maintenance (is virtual because D is unknown)

S_3 : Slope of the deterioration curve before maintenance (is real because F and E are known).

[Figure 5-13](#): Relative Timing Scenario 2- Maintenance before Monitoring

(a) Vertical elevation in performance, Performance Jump, PJ , due to maintenance at year t is represented by the line DF .

$$PJ = C_F - C_D$$

But $C_D = C_A - (S_I * p)$

$$\Rightarrow PJ = C_F - C_A + (S_I * p)$$

Because C_D is unknown, S_I is also unknown. However, S_3 is known, and if k (the slope ratio (S_I/S_3)) is known, S_I can be found.

Let $S_I = k * S_3$

$$\Rightarrow DF = C_F - C_A + (p * k * S_3)$$

but $S_3 = (C_F - C_E) / q$

Therefore the expression for performance jump is given as follows:

$$PJ = C_F - C_A + p * k / q * (C_F - C_E) \dots\dots\dots(44)$$

If rate of deterioration before maintenance is same as that after maintenance (i.e., if $k = 1$),

$$PJ = C_F - C_A + \frac{p}{q} (C_F - C_E) \dots\dots\dots(45)$$

If the interval of deterioration measurements is one year (i.e., if $p = q = 1$),

$$PJ = 2C_F - C_A - C_E \dots\dots\dots(46)$$

(b) Deterioration Reduction level (DRL) or Subsequent Change in Deterioration due to maintenance at year t .

$$DB = C_E - C_D \dots\dots\dots(47)$$

But $C_D = C_A - (S_I * p)$

$$\Rightarrow DF = C_E - C_A + (S_I * p)$$

$$= C_E - C_A + (p * k * S_3)$$

but $S_3 = (C_F - C_E) / q$

$$DRL = C_E - C_A + \frac{k * p}{q} (C_F - C_E) \dots \dots \dots (48)$$

If slope factor = 1,

$$DRL = C_E - C_A + \frac{p}{q} (C_F - C_E) \dots \dots \dots (49)$$

If the interval of all deterioration measurements is 1 year (i.e., $p = q = 1$),

$$DRL = C_F - C_A \dots \dots \dots (50)$$

This result can also be proved using geometrical construction.

(c) Deterioration Rate Reduction level (*DRR*) due to maintenance at year *t*.

DRR = deterioration rate after maintenance – deterioration rate before maintenance

$$= S_2 - S_1$$

$$DRR = \frac{C_E - C_D}{(t + q) - t} - \frac{C_D - C_A}{t - (t - p)} = \frac{C_E - C_D}{q} - \frac{C_D - C_A}{p} \dots \dots \dots (51)$$

$$= \frac{C_E}{q} + \frac{C_A}{p} - C_D \left(\frac{1}{p} + \frac{1}{q} \right) \dots \dots \dots (52)$$

But $C_D = C_A - (p * k / q)(C_F - C_E)$. Therefore Equation (52) becomes

$$DRR = \frac{C_E}{q} + \frac{C_A}{p} - \frac{p+q}{pq} \left[C_A - \frac{pk}{q} (C_F - C_E) \right] \dots \dots \dots (53)$$

$$DRR = \frac{C_E}{q} + \frac{C_A}{p} - \frac{p+q}{pq^2} [qC_A - pk(C_F - C_E)] \dots \dots \dots (54)$$

If $k=1$, Equation (54) becomes:

$$DRR = \frac{C_E}{q} + \frac{C_A}{p} - \frac{p+q}{pq^2} [qC_A - p(C_F - C_E)] \dots \dots \dots (55)$$

If the intervals of monitoring is 1 year, i.e., when $p=1$ and $q=1$,

$$DRR = C_E + C_A - 2[C_A - (C_F - C_E)] = 2C_F - (C_A + C_E) \dots \dots \dots (56)$$

It is noticed that this is the same as the expression for performance jump when $k=1$.

(d) Relationships Between the Three Deterioration Reduction Measures, Relative Timing Scenario 2.

Deterioration Reduction Level and Deterioration Rate Reduction:

The Deterioration Reduction Level is given by:

$$DRL = C_E + C_A + \frac{p^*k}{q} (C_F - C_E) \dots \dots \dots (57)$$

$$C_F - C_E = \frac{q}{pk} (DRL - C_E - C_A) \dots \dots \dots (58)$$

Also, Deterioration Rate Reduction is given by:

$$DRR = \frac{C_E}{q} + \frac{C_A}{p} - \frac{p+q}{pq^2} [qC_A - pk(C_F - C_E)] \dots\dots\dots (59)$$

Making $C_F - C_E$ the subject of this equation yields

$$C_F - C_E = \frac{q}{pk} \left[C_A - \frac{(qC_A + pC_E - pq * DRR)}{p+q} \right] \dots\dots\dots (60)$$

Equating the Equations (59) and (60) yields:

$$\frac{q(DRR - C_E - C_A)}{pk} = \frac{q}{pk} \left[C_A - \frac{(qC_A + pC_E - pq * DRR)}{p+q} \right] \dots\dots\dots (61)$$

Making DRL the subject of the equation,

$$DRL = 2C_A + C_E - \frac{(qC_A + pC_E - pq * DRR)}{p+q} \dots\dots\dots (62)$$

Also, making DRR the subject of the equation yields

$$DRR = \frac{(qC_A + pC_E - (p+q)(2C_A + C_E - DRL))}{pq} \dots\dots\dots (63)$$

Performance Jump and Deterioration Reduction Level:

From Equation (57), Deterioration Reduction Level is given as follows:

$$DRL = C_E + C_A + \frac{p * k}{q} (C_F - C_E)$$

Also, from Equation (45) Performance Jump is given as follows

$$PJ = C_F - C_A + \frac{k^* p}{q} (C_F - C_E)$$

The second and third terms of the expressions for DRL and PJ are equal. Therefore the other terms can be equated as follows:

$$DRL - C_E = PJ - C_F \dots\dots\dots(64)$$

DRL and PJ can then expressed as follows:

$$DRL = PJ - C_F + C_E \dots\dots\dots(65)$$

$$PJ = DRL + C_F - C_E \dots\dots\dots(66)$$

Performance Jump and DRR :

From Equation (65), $DRL = PJ - C_F + C_E$

Substitution into (63) yields the following

$$DRR = \frac{(qC_A + pC_E - (p + q)(2C_A + C_F - PJ))}{pq} \dots\dots\dots(68)$$

Also, making PJ the subject of the equation yields the following:

$$PJ = 2C_A + C_F - \left(\frac{qC_A + pC_E - pqDRR}{p + q} \right) \dots\dots\dots(69)$$

With the expressions derived for DRL , PJ and DRR , and the relationships determined between these measures of deterioration, such measures can be obtained using basic year-to-year deterioration data, and given a set of assumptions of the behavior of the slope of the deterioration curve after maintenance.

5.3 Methodology For Evaluating Long-Term Maintenance Cost-effectiveness

Unlike that for the short-term, evaluation of the effectiveness of maintenance in the long-term does not consider individual treatments at a point in time, but involves maintenance strategies (a group of one or more treatments carried out over a period of time).

In the present study, long-term evaluation of maintenance effectiveness begun with development of performance models expressed as a function of maintenance, among other factors. This was followed by determination of a zero-maintenance curve by equating the performance curve to zero. Strategies were then formulated for each category of pavements. For a given strategy, the effectiveness is represented by the extra benefit offered by that strategy (either in terms of extended service life or as increased area under the performance curve, relative to the zero-maintenance curve). The implementation of a series of maintenance activities has the effect of providing small performance jumps any time maintenance is carried out. Pavements with maintenance therefore have a curve that has a jagged or saw-toothed shape. Smoothing out the jagged curve yields a line that has a slope gentler than that of the zero-maintenance curve, indicating the impact of maintenance.

The benefit of each maintenance treatment is assessed by finding the incremental area gained due to the treatment. This was done for the entire life cycle of the pavement under that strategy. Another important assumption made in the current study is that the benefits of different treatments at a point in time, or the benefits of the same or different treatments at different points in time are independent of each other, so the total benefit is simply a sum of the benefits of the individual treatments. Salvage values were assumed to be zero, as data for salvage computation were not available at the time of the present study.

The cost of each maintenance treatment was determined using cost models developed as part of the present study. Formulation of maintenance strategies for the entire life cycle of the pavement was carried out to ensure that the inclusion of a sufficient range of maintenance scenarios, and that such formulations were as realistic as possible. The cost of each strategy was computed by summing up the agency cost (total cost) of the individual treatments as well as the user costs (expected delay and safety costs due to maintenance work zones).

The incremental benefits and incremental costs of each strategy over the zero-maintenance scenarios were determined, and the incremental cost-benefit ratio was found as the cost-effectiveness index of that strategy. Unlike the methodology adopted in previous studies, [Anderson et al., 1979; New York DOT, 1992, Geoffroy, 1996], the present studies considers that corrective maintenance (such as shallow and deep patching) will be, by default, be part of any strategy, no matter the composition (preventive maintenance components) of the strategy. The strategy with the highest value of cost-effectiveness was adjudged the most desirable strategy for the pavement category in question. The entire procedure for the evaluation of maintenance in the long-term is included in the framework chart as shown as Figure 5-14.

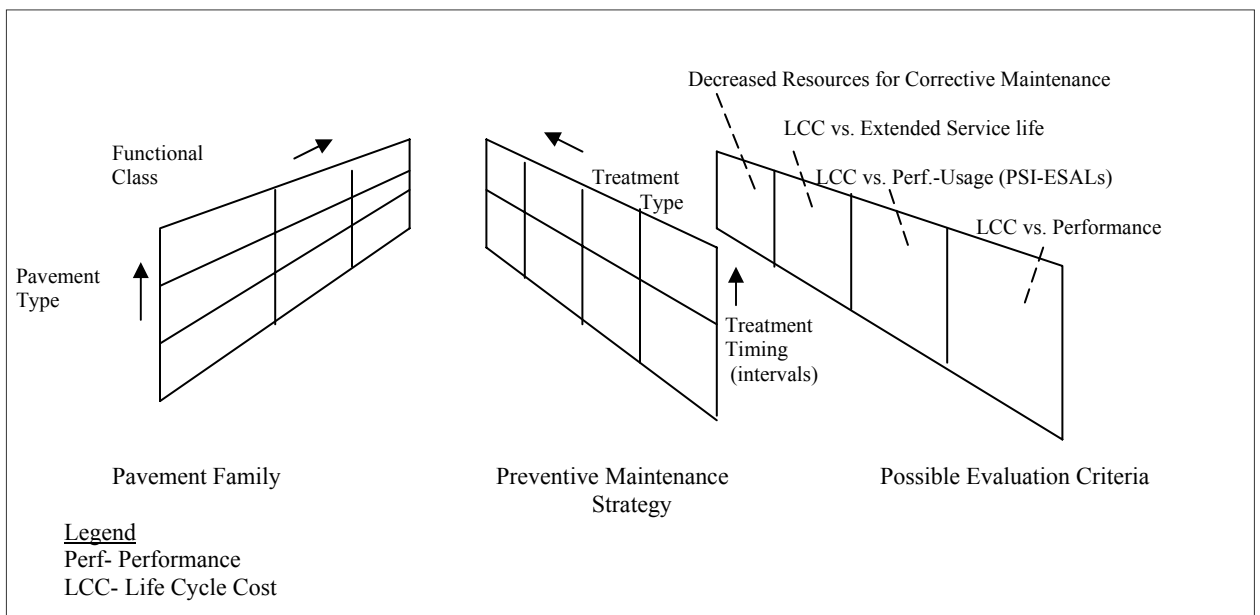


Figure 5-14: Long-term Evaluation of Maintenance Effectiveness

5.3.1 Performance Modeling

Pavement performance analysis was carried out as part of this study to describe past performance and to predict future performance of each category of pavements. Determination of trends in pavement performance (especially as a function of maintenance, among other factors) over a period of time or cumulative loading is vital to the current study for the following reasons. First, it would make it possible to

determine the zero-maintenance curve, a reference point from which benefits or costs of any maintenance strategy can be measured. Secondly, it would enable the estimation of life cycle benefits either in the form of area under the curve, or extension to pavement service life, which are necessary for cost-effectiveness studies. Indeed, effective and reliable maintenance and rehabilitation plans hinge on the ability of highway agencies to make good predictions of future pavement performance. Failure to predict performance correctly may lead to inappropriate choices of preventive maintenance activities and timings, and would ultimately result in inefficient and ineffective utilization of scarce resources.

In a bid to develop good performance models for this study, due cognizance was taken of the recommendations made by past researchers [Darter, 1980; Lytton, 1987] who noted that the following considerations are significant in pavement performance prediction and modeling:

- an adequate database developed using data from in-service pavements
- inclusion of all variables that significantly affect pavement performance
- an adequate functional form of the model
- role of statistics and mechanics in developing an efficient model
- modification of each model to represent the effects of maintenance
- adherence to established and proper statistical criteria for assessing model precision
- limitations and uses of specific models.

Considerable effort was expended in assembling a large database containing all possible pavement-related data for about 10,000 segments of 1-mile in-service pavements on Indiana's state highway network. This task is discussed in detail in Chapter 6 of this report (Data Collection and Collation). A discussion of other important considerations for good performance models, i.e., choice of a good set of explanatory variables, an appropriate response variable, and a performance model type that adequately explains the relationship between the selected explanatory variables and response variable, and model validation techniques, is provided as follows:

5.3.1.1 Response Variables Considered

There are generally four types of response variables for performance models as shown in Figure 5-15 [Lytton, 1987].

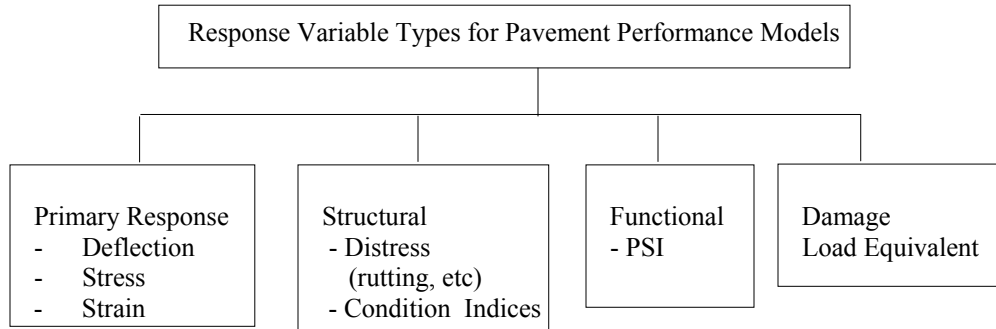


Figure 5-15: Types of Response Variables for Performance Models

Collection and processing of primary response data, such as deflection, often involve a great deal of time and expense. Also, as regards structural response variables, non-standardization of such variables to indicate structural conditions renders those variables largely unsuitable. Therefore, responses variables typically used for performance modeling are those that describe the functional performance of the pavements, such as roughness. However, some distress types such as cracks generally have no noticeable impact on roughness, as illustrated schematically in Figure 5-16.

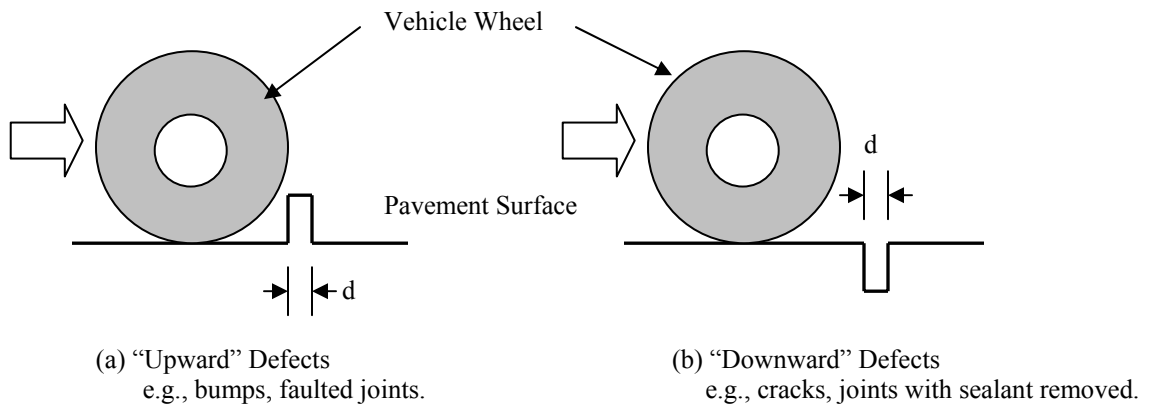


Figure 5-16: Relative Impacts of Pavement Defect "Direction" on Roughness

“Upward” surface defects indicated in [Figure 5-16](#) refer to bumps and shoved material on flexible pavements, and faulted joints on rigid pavements. Such cases typically involve the deformation of pavement material in a direction upwards from the plane of the pavement surface. Upward defects readily show in roughness measurements, as the wheel climbs over and down such defects. “Downward” defects refer to cracks, and joints with removed or deteriorated sealant. Such instances involve the loss or absence (void) of pavement material in a direction perpendicularly downwards from the plane of the pavement surface. Unlike upward defects, downward defects do not easily translate into higher roughness, especially for low values of “d” (i.e., very thin cracks). Even the use of laser equipment for roughness measurements may be ineffective in capturing the presence of large cracks (or more appropriately, the effect of cracks on roughness) especially when cracks are filled with soil. This means that an extensively cracked pavement surface may nevertheless have a low roughness value, erroneously indicating that the pavement is in good or fair condition. For large values of “d” (e.g., potholes), the effect of the defect on roughness increases.

In spite of this limitation of roughness measurements in assessing true pavement deterioration, roughness was used in the present study due to the following reasons: (i) comprehensive data on other measures of pavement condition (such as PCI, PCR) are generally not available (ii) the State of Indiana, like many other states, has roughness data for most of their highway sections, over a relatively long period of time, (iii) public perception of pavement performance has been found to be directly related to pavement roughness, (iv) there exists relationships between roughness and other common aggregate measures of pavement performance such as PSI [Gulen et al., 1994; Darter et al., 1994], (v) roughness can be related to the deterioration of pavement structures, (vi) new technology (such as lasers) make collection of roughness data very easy, safe and convenient, therefore it is likely that many states will continue to use roughness as a measure of pavement deterioration.

Roughness is expressed in counts per unit length of road and is measured by equipment mounted on a vehicle at constant speed on the road. For each pavement section and year, PSI values were derived from roughness values using the relationship derived in previous studies.

Many response variables are based on subjective indices that use a predetermined set of distress measures selected at a time when less developed data collection technologies were used. The emergence of

a large variety of automated technologies, however, has made available large quantities of data that need new methods for processing and analyzing this data to yield useful results. These facts, coupled with the current lack of an ambiguous and objective approach to directly measure pavement performance, motivated a research effort that explored the existence of a two-component performance measure for pavement deterioration [Ben-Akiva et al., 1991]. These researchers used a latent response variable to represent functional performance and another response variable to represent the structural integrity of the pavement. The latent approach was not considered in the present study due to currently unavailable or unreliable data on pavement layer material thicknesses and types, and subgrade quality, which are essential for computation of pavement structural integrity.

5.3.1.2 Explanatory Variables

From the literature review and questionnaire survey, a number of variables provide explanations for the variations of pavement performance. These include the following “primary” factors:

- Pavement type
- Highway class or route type
- Usage (cumulative traffic loading, or age)
- Environmental region
- Types and levels of annual maintenance

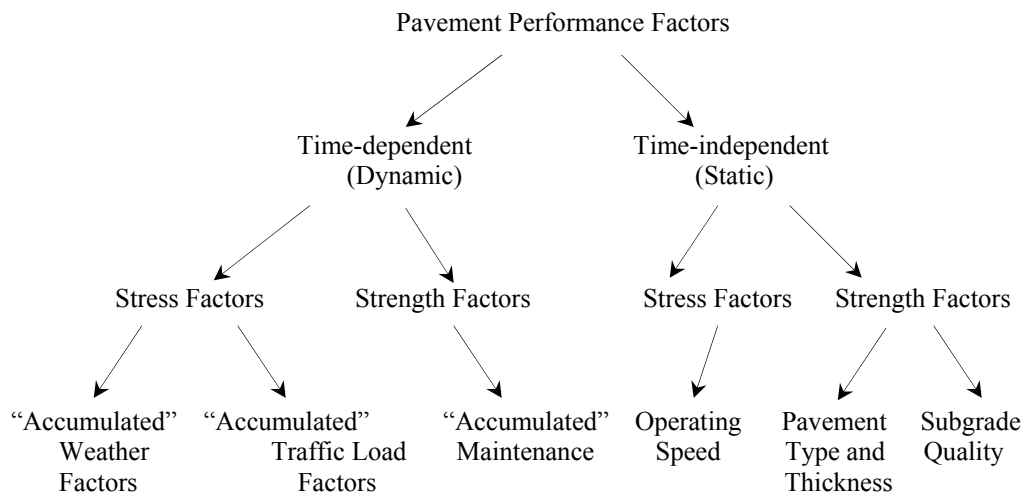
Other “secondary” factors are as follows:

- Subgrade quality
- Subsurface drainage
- Pavement thickness
- Design and construction features
- Topography and nature of surface geology
- Climatic features (freeze-thaw cycles, precipitation, freeze index, etc).

The latter group of factors is described as secondary because in the absence of data on such factors, they may be represented by surrogate variables included “primary” factors. For instance, highway

class or route type may be considered a surrogate dummy variable for subgrade quality and design and construction features, because design and construction standards and specification typically vary by route type. To some extent, highway class may also be used as a surrogate for pavement thickness and subsurface drainage as Interstate pavements are generally associated with presence of subsurface drains [Christopher and McGuffey, 1997] and greater thicknesses. Also, a dummy factor indicating environmental region could be used to represent topography, nature of the surficial geology and climate, especially in the case of Indiana where such features vary considerably by region.

The major dependent variables identified above fall under two categories: “dynamic” (time-dependent) factors whose values change with as the pavement increases in age, such as accumulated values of loading, weather effects, and maintenance; and “static” (time-independent) factors that generally remain fairly constant with the passage of time, such as subgrade strength, pavement type, and functional class. Under each category, are two sub-categories: stress factors, which contribute to pavement deterioration, and strength factors, which militate against pavement deterioration. Typical examples of such factors are show as [Figure 5-17](#). “Accumulated” time-dependent factors may refer to cumulative value of the factors, or the total “moment” of such factors. A discussion of the various factors and the concept of moments are presented in subsequent sections in this chapter.



[Figure 5-17](#): Factors Affecting Pavement Performance

5.3.1.2.1 Time-Dependent Stress Factors

(a) Climatic Factors

As regards temperature conditions, pavements in relatively cold regions exhibit different patterns of deterioration compared to those in warmer climates. The colder regions in the study area (Northern Indiana) are not in a permanently frozen state, but experience relatively warm weather at certain times of the year. Low temperatures and freeze-thaw transitions result in volume changes in the pavement layer aggregates as well as any moisture that occupy the voids of the aggregate matrix. With regard to moisture conditions, drainage (or lack thereof) is a major factor influencing pavement deterioration: pavements in areas with high precipitation deteriorate faster than those in drier areas, all other factors being equal.

Besides affecting long-term pavement performance, environmental features also have a direct impact on the immediate observed effectiveness of maintenance treatments. For instance, crack sealing has been observed to be generally more effective in warmer climates with little rainfall. However, at areas with freezing temperatures and significant precipitation, the life span of the sealant is greatly reduced. Also, chip seals are less effective where rainfall is more severe, and weather more frequently in warmer areas. Furthermore, thin overlay treatments are especially susceptible to low temperatures and high moisture. A more detailed discussion on the climatic variations across the state is provided in Chapter 2 (Trends Analysis). There are generally three ways in which weather factors can be included in a pavement performance model:

- By a factor representing climatic region
- Using climatological weather parameters, such as precipitation, freeze index, etc
- Using a single index that represents the combined severity of weather parameters

In the present study, a single index was developed to represent the relative effects of the various weather parameters, based on the opinions of pavement managers at a sub-district level in Indiana that was obtained through an agency survey. Details of the development of this index are provided in Chapter 6.

(b) Pavement Usage

Pavement usage is measured as the number of vehicles that use the pavement, or more appropriately, the total loading experienced by a pavement. In order to account for the heterogeneity in traffic loading, a common measure of traffic loading, ESALs is widely accepted for practice. INDOT, as well as the 13 metropolitan planning organizations in the state, currently collect data on traffic volumes, vehicle classifications, and sometimes, vehicle weights for the HMPS program as well as other uses [Labi and Fricker, 1998]. There are methods that use these raw data types to generate annual ESAL values. The cumulative ESAL, or CESAL, indicates how much loading a pavement has taken in its lifetime. A large CESAL value could be due to a high volume of heavy loading in a relatively short period of time, or a low volume of loading over a long period. For this reason, both age and usage were initially considered as explanatory variables in this study, and appropriate steps were taken to address any statistical bias that may result from possible correlation between these two variables. Pavement usage data in terms of daily traffic levels were obtained in addition to loading data, as such data are useful for developing maintenance guidelines. For instance, road sections that have high volumes of traffic have generally been found to be unsuitable for certain kinds of preventive maintenance, such as chip sealing, not necessarily from a cost-effectiveness viewpoint, but because loose aggregates pose a safety hazard. Plots of pavement performance versus accumulated usage (ESALs), and also versus accumulated weather effects for various pavement types and in various regions, are presented in Appendices A to F.

(c) Pavement age

Pavement age has often been considered as a representation of the combined effects of load and weather effects, and is often used in lieu of these variables, especially in situations of lack of data. The aging process starts right after the material is newly laid: oxidation of asphaltic cement in flexible pavements results in the materials becoming brittle and susceptible to cracking, especially under traffic loading. Also, Portland cement in concrete slabs in rigid pavements under a chemical reaction with the ambient air that slowly degrades the concrete [Neville, 1995; Geoffroy, 1996]. The age of a pavement can

be expressed as a primary age (when the pavement was constructed) or a secondary age (when the pavement was last resurfaced). The current study uses the latter meaning to represent age, for three reasons:

- pavement construction history data is usually difficult to obtain,
- the present study is primarily concerned about maintenance between rehabilitation activities, so a zero age should correspond to the time of resurfacing, and
- the aging phenomena described above apply only to topmost layer, and aging of the underlying layers is relatively insignificant, as they are shielded from the weather.

Plots of pavement performance versus age, for various pavement types and in various regions are presented in Appendices A to F.

5.3.1.2.2 Time Related Strength Factors

(a) Maintenance History

Obviously, the more maintenance a pavement receives, the better the condition of the pavement, all other factors else being equal. This is because maintenance treatments directly address the pavement defects that translate into low PSI values. For example, crack sealing closes up surface cracks, bump planing eliminates surface ripples and bumps that cause roughness, and patching replaces dislodged surface material. Previous studies have advocated the inclusion of maintenance an explanatory variable in pavement performance modeling [Lytton, 1989, Markow, 1994]. However, relatively very few researchers have actually done so [Fwa and Sinha, 1987; Al-Suleiman et al., 1994]. There are four ways in which maintenance has been used, or could be used in pavement performance modeling:

- Whether or not maintenance was carried out in the previous year
- The level of maintenance in the previous year
- Cumulative maintenance since last rehabilitation
- Total maintenance moment since last rehabilitation

Some studies have attempted to incorporate the effect of maintenance by including a variable representing the amount of maintenance carried out in the previous year [Ramaswamy and Ben-Akiva, 1990], or the maintenance decision (maintenance or no maintenance) in the previous year [Mohammad et

al., 1997]. It is however doubtful that maintenance history only for the previous year can adequately explain the effects of maintenance on pavement performance in a given year. A more appropriate approach would be to consider maintenance history in its entirety. Obviously, the lack of maintenance data is the reason for the neglect or sparing use of such a vital variable in pavement performance modeling.

The present study considers maintenance history of each pavement from the time of reconstruction or rehabilitation, as provided in the maintenance module of the INDIPAVE 2000 database. In cases of missing data, annual pavement maintenance expenditure models are used for imputation. For the present study, performance models were developed using the maintenance moment, the concept for which is explained in Section 5.3.1.4.

5.2.1.2.3 Time-independent Strength Factors

(a) Subgrade material

The subgrade is the lowest structural layer that supports the entire pavement, and it is typically desired that this layer should have a high shear and bearing strength, low plasticity, and low moisture content. The subgrades of the state highways in Indiana are either natural ground or engineering material fill material. While the natural surficial soils of northern Indiana are dominated by those of glacial origins, the southern part of the state has soils of a residual nature i.e., derived from decomposition of the parent rock. The general shallowness of the Wisconsinian drift in northern Indiana results in problems of subsoil drainage, especially when interbedded cohesive soils are found with the granular drift [Yoder and Witczak, 1975; Fenelon et al., 1994]. The degree of glaciation is most pronounced at the northeastern region of the state, i.e., the Fort Wayne District, a region where pavements experience the most distress per unit mile, all else being equal, and subsequently, the highest levels of maintenance per mile. Further details on the spatial variation of surficial soils is presented in Chapter 2 .

Mechanical properties of subgrades include particle size distribution, such as well-graded or uniformly graded soil, predominantly fine soil (such as clay or silt) or coarse soil (such as gravel or sand). Generally, coarse materials exhibit greater strengths and are relatively stable upon wetting. On the other extreme, clay soils are stable only when dry, and rapidly lose their strength as they absorb more moisture.

The Plasticity Index is a measure of the extent to which a subgrade material can absorb moisture without succumbing to shear failure. The subgrade CBR or resilient modulus, if known, is probably the best method of assessing soil strength. However, the variability of surface geology and hence, subgrade characteristics even over small stretches of highway, coupled with the tediousness of determining these values, probably makes the acquisition of CBR and resilient modulus difficult for studies of this kind. Because state department construction specification typically varies by road functional class (Interstates have the most stringent quality assurance standards) subgrade quality in the present study is represented by an indicator variable such as route type or functional class.

(b) Subsurface Drainage

Subsurface drainage is a vital element in the design and performance of pavement systems. Exclusion of subsurface drainage considerations assuredly leads to premature failure of the pavement and ultimately, high-life cycle costs. The use of a variable to indicate the presence or otherwise of subdrains (also referred to as “underdrains”) may cause problems of statistical bias because subdrains are not needed at all sections. For example, a result that concludes that better condition is attained where no subdrain exists may be reflective of a serious modeling error in which most of the pavement sections used for the model are located at sections free of possible inundation and therefore do not require subsurface drainage. Therefore only those sections located in areas prone to subsoil inundation and therefore need subsoil drains need to be used for modeling the effectiveness of such underdrain maintenance or for development of performance curves that must include the “subdrain presence” variable. From available INDOT data, it is not possible to determine, and therefore isolate those sections that need such drains for modeling. Therefore this variable was not considered for the modeling process.

(c) Pavement type

PCC pavements respond to load and weather effects in a different manner as AC pavements, as these two materials have different chemical and physical properties. In the present study, performance model are built for each type of pavement, i.e., PCC, AC and PCC-over-AC.

(d) Structural capacity of pavement (pavement thickness, or structural number)

Structural capacity is represented by slab thickness and structural number for rigid pavements and flexible pavements, respectively. Higher structural capacities translate to higher pavement conditions, all else being equal. It is worth noting that the SHRP LTPP SPS-3 and SPS-4 appropriately consider both subgrade type and structural number in the stratification levels in the design of that experiment [Hadley et al., 1994; Hanna, 1994].

(e) Design and Construction Features

Design and construction features also play a vital role in pavement performance. As regards flexible pavements, the asphalt content, amount of air voids in the asphalt mix, degree of compaction, and rheological properties of the asphalt binder all affect the durability of the pavement surface layer. In the case of rigid pavements, the presence and size of dowels, joint spacing, water cement ratio, and air content are important predictors of the life span of the concrete slab. For all pavement types, the type of base material (natural gravel, dense crushed aggregate, or stabilized aggregates) affect the durability of the overlying pavement. Stabilized aggregates typically offer greater stability and ultimately, pavement performance, albeit at greater cost.

(f) Pavement Surface Smoothness Just After Last Rehabilitation

Pavement surface smoothness just after last rehabilitation may be considered a design and construction feature. The AASHTO pavement design equations imply that a higher level of initial smoothness leads to a longer pavement life [AASHTO, 1993]. This is probably because an uneven pavement surface increases the dynamic loading of effects of truck traffic on the pavement, which in turn induces more deterioration. A subsequent study found that a 25% increase in initial smoothness generally corresponds to a minimum of 9% increase in pavement life [Smith et al., 1996]. In this regard, a pavement with a smooth initial pavement will outlast another with an initial rough surface, all other factors and conditions being equal.

5.3.1.3 The Concept of Pavement Experience as a Universal Explanatory Variable for Dynamic Factors

As seen in Figure 56, pavement deterioration is influenced in part by three dynamic factors (loading, weather, and maintenance). The accumulated value of each of these factors increases over time, and it is logical to consider their accumulated, rather than their annual, values, in explaining the pavement response (condition) at a given point in time. This situation is analogous to vehicle use. The condition of a vehicle at a point in time is the culmination of the load (mileage), weather effects (pronounced rusting occurs at coastal areas), and maintenance (regular oil changes, tune-ups, etc. enhance vehicle life). For this reason, given two identical cars of the same age, it will be expected that one that has had less loading (low odometer reading), less weather effects (used in an inland city such as Indianapolis), and more maintenance (meticulous regular preventive maintenance and prompt corrective maintenance) will, at any given time, be in a better condition than the poorly maintained one with higher mileage driven in a coastal city such as Tampa. The difference in their condition is expected to increase as their age increases. Another analogy is in the area of human medicine, where each individual of a pair of twins are expected to have different levels of health at any point in time, given two extremes of external exposure (one does excessive manual labor in a severe environment but with no medical care, and another does regular work in a mild environment and does regular health check ups and treatment).

Pavement “experience” may be defined as the accumulated effects of the dynamic factors of deterioration on a pavement. Previous researchers have used various ways to express pavement experience, the most common being pavement age, as evidenced from the literature review (Chapter 3). The popular use of this variable stems from the fact that it is relatively easily available (no field data collection is needed, as field records at most highway offices typically provide documentation on construction dates. Another reason for the widespread use of the age variable is that it embodies, in a general sort of way, the accumulated effects of traffic loading, weather severity and maintenance, as was aptly recognized by some researchers [George et al., 1991]. However, as demonstrated by the above examples on pavements, vehicles, and humans, age alone may not be a sufficient predictor of the condition of any system that is subject to these three dynamic variables.

In cases where data on the dynamic pavement deterioration factors are available, and where more accuracy in the deterioration trend (performance curve) estimation is desired, pavement experience can be expressed in terms of the individual factors, i.e., loading, weather, and maintenance, which are typically measured in ESALs, freeze indices (in degree-days), and dollars expended on maintenance. Some studies have used annual values of these factors, with or without taking due consideration of the number of times (age) these annual factors have been experienced to date. Where annual values of the dynamic factors have been used as independent or stand-alone terms in performance models, they cease to be dynamic in the true sense of the word and become variables that reflect static conditions. For example, consider the equation below [Sebaaly et al., 1995]:

$$PSI = 3.27 + C_l + 2.86 * e^{-6 * ESALs} - 0.56 * \text{Structural number} - 0.13 * \text{Year}$$

In this equation, pavement loading, expressed as ESALs, is included in the model obviously to reflect the fact that some pavement have higher annual loading than others, and therefore reflects a static factor such as functional class or route type. The dynamic variable in this model is Year, which reflects the age of the pavement.

In past studies where the individual dynamic deterioration factors have been considered, the most common practice has been to express pavement experience simply as the accumulation of that factor, such as cumulative ESALs [Daleiden et al., 1993], or accumulated freeze indices [Madanat and Archilla, 2000].

Variables to represent maintenance application (either as a 0-1 decision or as a dollar amount) have been used rather sparingly in the past, obviously due to lack of data. Even in such studies, the consideration of maintenance has typically stopped at using a variable to indicate maintenance decision only in the previous year [Mohammad et al., 1997] or the level of resources expended on maintenance only in the previous year [Ramaswamy and Ben-Akiva, 1990]. Clearly, assessing the influence of maintenance by considering its occurrence in only the previous year is inadequate, especially if maintenance in the previous year is not a true reflection of the average or sum of maintenance received by the pavement section in its lifetime, which is typically the case.

From the above discussion, it can be seen that pavement experience, in its totality, can be expressed as the accumulation of the effects of load, weather, and maintenance as follows:

$$\text{“Negative” Experience} = \Sigma\text{Load} + \Sigma\text{Weather} - \Sigma\text{Maintenance} \dots\dots\dots (69)$$

$$\text{“Positive” Experience} = \Sigma\text{Maintenance} - \Sigma\text{Load} - \Sigma\text{Weather} \dots\dots\dots (70)$$

(Experience will refer to “Negative” experience for the rest of this discussion, but the concepts are applicable if positive experience were used instead). Because the units of the dynamic factors are not additive, it may be more appropriate to express pavement experience as the sum of weighted factors (Equation 71) or as a product of the factors (Equation 72).

$$\text{Experience} = \Sigma L/(L_{MAX}) + \Sigma W/(W_{MAX}) - \Sigma M/(M_{MAX}) \dots\dots\dots (71)$$

Where *L*, *W*, and *M* refer to load, weather and maintenance.

Equation (71) utilizes the ratio of the factors to their maximum possible values, thereby obviating the problem of dimensions. The use of maximum possible values is tainted by problems of subjectivity, and Equation (72) may be preferred instead.

$$\text{Experience} = (\Sigma\text{Load} * \Sigma\text{Weather}) / \Sigma\text{Maintenance} \dots\dots\dots (72)$$

Equation (72) inherently assumes that the weights of each factor on pavement experience (and consequently, pavement damage) are equal. To avoid the undue restrictiveness of this assumption, the equation may be modified by including indices that reflect the weight, or contribution of each factor to pavement deterioration, as shown in Equation (73):

$$\text{Experience} = (\Sigma\text{Load}^l * \Sigma\text{Weather}^w) / \Sigma\text{Maintenance}^m \dots\dots\dots (73)$$

Equation (73) provides a general form which, wittingly or unwittingly, have been used by past researchers to express pavement experience. In studies that considered only loading, an implicit assumption was made that *w* = *m* = 0, leaving experience only as cumulative loading. Also, studies that utilized pavement age the only dynamic factor inherently assumed that *l* = *w* = *m* = 0, and the annual value of each factor was taken uniformly as 1 unit.

From Equation (73) it is seen that pavement experience can be expressed in terms of maintenance-load-weather (*MLW*) units which may take any of the below values depending on the values of the indices *l*, *w*, and *m*, and units in which the factors are measured:

- *CESALs*
- *CESAL.AWU*

- *CESALAFI.MTCE*, etc.

AWU refers to accumulated weather units, which may be expressed by individual weather parameters such as freeze index, number of freeze thaw cycles, precipitation, an aggregate index representing the combination of these factors, or weather severity indices established by previous research such as the Thornwaite moisture index, “*e*” [Martin, 2000], or the climate coefficient, “*m*”, used in World Bank’s HDM [Paterson and Attoh-Okine, 1993]. *MTCE* refers to accumulated maintenance received by a pavement over its lifetime in constant dollar value, but could refer to the amount of material used, man-hours expended, or other measure of resource use.

5.3.1.4 Expressing Pavement Experience: Cumulative Approach versus Method of Moments

The above discussion shows how pavement experience may be expressed in terms of the accumulation of individual dynamic pavement deterioration factors. This approach involves simple addition of the annual values of these factors, i.e., the cumulative approach. An alternative approach would be to provide weights to each annual occurrence of each dynamic factor, so that the more recent the factor, the greater its effect on the current pavement condition, all else being equal. This approach of measuring pavement experience may be termed the Moment Approach as illustrated in Figure 5-18.

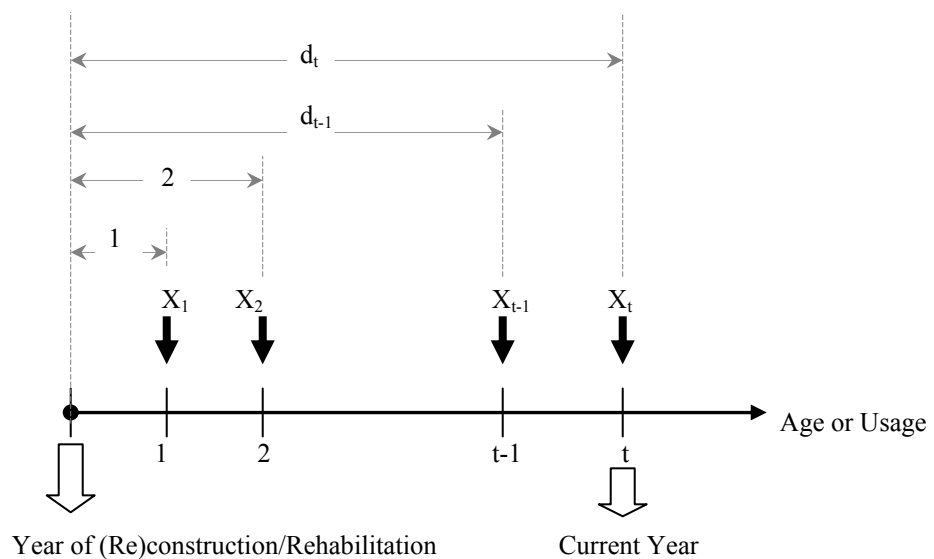


Figure 5-18: The Moment Approach to Expressing Pavement Experience

In the figure, X_i refers to the occurrence level of factor X at year i , t is the current year, d_i represents the weight of the factor occurrence X_i , i.e., the recentness (or nearness) to the occurrence of that factor to the current year. d_i is measured by the distance of the instance of factor occurrence from time of rehabilitation or (re)construction.

The total moment of factor X_i is therefore given as follows:

$$MOMENT = \sum_{i=1}^t (X_i * d_i) \dots \dots \dots (74)$$

An assumption associated with the concept of moment, as expressed in Equation (74), is that the effect of any factor X diminishes at a linear rate with the passage of time, hence the first order of the “distance” variable. However, in reality, the order may be different from 1. Further research may be necessary to investigate this issue.

5.3.1.5 Selection of the Type and Form of Performance Model

The term “model type” could refer to the type of response variable being used, or the structure of the model itself. For purposes of the present study, the latter definition is used, as the type of response variables has already been discussed under a separate section.

Two main pavement performance types have been identified: deterministic and probabilistic models. Deterministic models consist of the following [Lytton, 1987]:

- purely mechanistic models (relationship between a response parameter such as stress or strain, and deflection),
- mechanistic empirical models (relationship between a response parameter, such as roughness, cracking, and traffic loading),
- purely empirical models.

Probabilistic models, unlike deterministic models, typically yield a range of response variables. Examples include survivor curves, and transition process models such as Markov and semi-Markov models). Probabilistic models are typically used for network level performance modeling.

Over the past decades, deterministic performance models, especially the “classic” regression type, have become very popular. A large number of different deterministic-based pavement performance prediction models have been developed for various state and local pavement management systems. However, it is important to recognize that it may not always be appropriate to apply deterministic models to all situations of pavement management due to the following reasons:

- Uncertainties in the behavior of pavements under varying conditions of traffic and climate,
- The difficulties in quantifying the factors that substantially affect pavement condition,
- The magnitude of errors associated with conditions and performance measurements, and the statistical biases that are inherent in the subjective evaluations of pavement condition.

Probabilistic models facilitate the prediction of pavement condition on a network level [Lytton, 1987]. Although significant progress has been made in probabilistic modeling of pavement performance, the use of such models is constrained by the difficulties in establishing transitional probability matrices.

Regression performance models:

It is stated that the actual performance curve for a section of pavement could be determined by performing a regression analysis of time-condition data [Geoffroy, 1996]. Indeed, most past studies on pavement maintenance have used this model type [Sharaf et al., 1988; Geoffroy et al., 1992; Joseph, 1992]. In Indiana, a regression equation was used to obtain a relationship between pavement performance (in terms of PSI) and pavement age [Al-Mansour et al., 1994]. In a fairly recent study statistical regression was used to obtain predictive pavement performance models for various pavement types, traffic levels, and environmental conditions among others [Daleiden et al., 1993]. Indiana DOT, for its Pavement Management System, uses a performance model derived using statistical regression. Nevada has comprehensive performance models for each rehabilitation and maintenance treatment it commonly employs [Sebaaly et al., 1996].

The use of regression techniques assumes that the errors are normally distributed and their variance is homogeneous, i.e., their distribution about the mean error does not systematically vary with the variation of the predicted value of the dependent variable. If they do, the problem could be addressed by transforming the dependent variable. However, if the transformation is complex, a mathematical formulation of the model could be bogged down by technical difficulty. The assumptions underlying the use of regression need to be verified in the data before it is used [Mouaket et al., 1990]. Furthermore, the accuracy of regression models can be adversely affected by any correlated explanatory variables. For example, a model that has pavement age and cumulative ESALs, without correcting for biases, could suffer because the greater the age of a pavement, the greater the likelihood of a high cumulative ESALs.

Regression model types have served the performance-prediction needs of pavement management systems for a long time, and many pavement engineers have become comfortable with the simplicity of that modeling technique. However, some researchers have expressed a desire for better models that can provide greater levels of prediction accuracy. This has led to the investigation of the use of other methods of modeling pavement performance, such as econometric models.

Econometric models

The past decade has seen a rise in the use of econometric modeling techniques in an attempt to explain pavement behavior in response to various environmental, usage and design factors [Ramaswamy and Ben-Akiva, 1990; Madanat and Mishalani, 1995; Mohammad et al., 1997] These techniques have also been used to model and predict the probability or amounts of maintenance. Most of such studies have generally been limited to research purposes, but some of the results they provide have been shown to be more consistent with actual observation, compared to those offered by traditional methods, and may therefore be better suited for practice. Econometric techniques are available to help avoid biases such as selectivity bias, simultaneity bias and endogeneity bias, which, according to some researchers, are often encountered from sample selection procedures and the cause/effect cycle of maintenance decisions and changes in pavement performance.

Reliable and accurate models are vital to effective planning and budgeting, and a key aspect of acceptability and applicability of any research results is that such results should be able to present

recommendations that as much as possible, are developed from studies that effectively utilized real life observations. Any modeling technique that promises to improve upon existing methods of pavement performance prediction is therefore worthy of consideration. This is important if the study results are to be geared towards acceptance and implementation by state DOT maintenance engineers and pavement managers.

5.3.2 Determination of the Zero-Maintenance Curve

For each pavement category, the zero-maintenance curve was simply obtained by equating the maintenance term of the performance curve to zero, and finding the resulting model. However, it is worth noting that because there is no in-service pavement that received zero maintenance, any zero maintenance model is only a result of extrapolation, and is therefore assumed to be a representation of the performance curve for a pavement that receives no maintenance over its entire life cycle.

5.3.3 Development of Cost Models for Various Maintenance Treatments

Maintenance policies are comprised of strategies that are simply a “collection” of one or more maintenance treatment types carried out various points in time on a given pavement. The costs of the treatments are a necessary input to cost-effectiveness modeling, and they provide a quantitative measure of the cost aspect of any strategy. Maintenance treatment cost models are different from maintenance expenditure models in that the former are treatment specific, while the latter are specific to a pavement section. Maintenance treatment cost models are therefore more appropriate for assessing the costs of maintenance strategies. Typically, factors that affect maintenance treatment costs belong to two groups: pavement attributes (such as location, condition, etc) treatment attributes such as type (alternative material or process), work source (in-house or by-contract). Average values as well as models for the cost of various maintenance treatments typically carried out in Indiana are provided in Chapter 8.

5.3.4 Formulation of Maintenance Strategies

As stated earlier in this report, a thorough cost-effectiveness evaluation of preventive maintenance is best carried out on the basis of strategies, not just treatments. A strategy may consist of none, one, or multiple treatments, each applied at its own frequency, or at a time when the pavement surface condition reaches a certain threshold. In particular, each strategy is a combination of treatment type(s) and treatment timings.

- The treatment type(s) criterion: For each pavement family the application of various appropriate preventive maintenance treatment types and rehabilitation activities are considered, depending on whether the pavement is rigid or flexible. A list of standard preventive maintenance treatment and rehabilitation activities is provided in Chapter 3.
- The timing (frequency of treatment(s) or pavement condition thresholds) criterion: The “timing” of a preventive maintenance activity is one of the most important factors in the long-term cost-effectiveness of that activity. The literature review (Chapter 3) confirmed that policies that utilize pavement usage (in terms of age or cumulative loading) as a criterion for timing have been used often by many agencies. Some policies use pavement condition for timing preventive maintenance activities, even though that involves more frequent monitoring of pavement condition. The “pavement condition trigger” in this context, is the minimum level of pavement condition at which preventive maintenance should be carried out. It is important to realize that timing intervals need not be uniform (Figure 5-19).

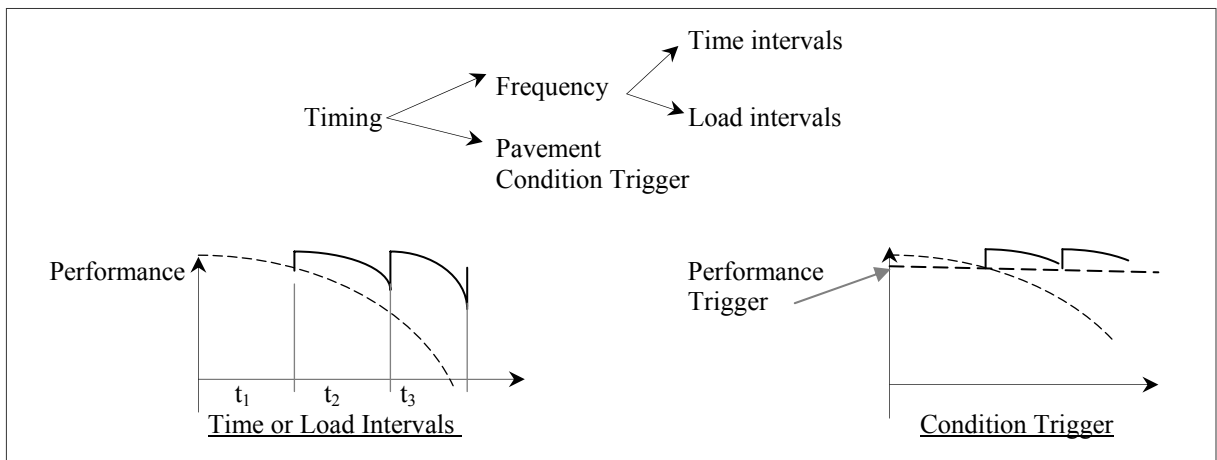


Figure 5-19. Treatment Timing Criteria

Examples of strategies are as follows:

- “Carry out crack sealing every 4 years and thin HMA overlay every 8 years”,
- “Carry out crack sealing any time cumulative loading (CESALs) reaches 5 million ESALs, and thin HMA overlay any time CESALs reaches 10 million ESALs.”, or
- “Carry out crack sealing anytime Aggregate Cracking Index falls below 3.5 units.”

A sample of a preventive maintenance strategy formulated on the basis of pavement age is shown as [Table 5-4](#).

Table 5-4: Sample of a Preventive Maintenance Strategy

Preventive Maintenance Strategy			
Year	Preventive Maintenance Activities		
	Seal Cracks	Micro-surfacing	Thin HMA Overlay
2			
4	X		
6		X	
8	X		
10			
12			X
14			
16	X		
18		X	
20	X		
22			
24			X

All strategies consist of a selection of preventive maintenance treatments that are carried out at certain years. Unlike preventive maintenance, corrective maintenance is generally not programmable, and therefore cannot be an option in each strategy. Lower levels of total preventive maintenance, over the entire life cycle, translate to higher levels of total corrective maintenance, and vice versa. Therefore, each strategy also consists of corrective maintenance treatments that are carried out every year, but whose levels is a function of the amount of preventive maintenance treatments. Given a certain combination of preventive maintenance treatments in a strategy, the cost was computed, and the corresponding total cost of corrective

maintenance was determined using an approximate trade-off relation developed for total corrective maintenance and total preventive maintenance in the given life cycle of a pavement. In that respect, the present study differs from previous studies that have formulated and evaluated various maintenance strategies based only on preventive maintenance strategies.

5.3.5 Computation of Life Cycle Costs and Benefits Associated with Each Strategy

5.3.5.1 Life Cycle Costs

Each strategy consists of a number of treatments. The life cycle cost of each strategy is the sum of costs of each set of treatments that comprise the strategy. Costs were discounted to their equivalent present worth using an interest rate of 6% to account for the time value of money as reflected in the opportunity costs of maintenance investments. The cost components associated with each strategy are as follows:

- Basic (maintenance treatment cost) i.e., equipment, labor, materials, and overheads
- User cost of delay due to work-zone speed restrictions
- User cost of safety due to increased expected value of crashes at work-zones

The overall cost of a set of treatments in a given year, per lane-mile, is therefore given by the following expression:

$$COST = \sum_{i=1}^N [(MC_i + DC_i + SC_i) * PWF(r, Y)] \dots \dots \dots (75)$$

Where $COST$ = Total costs of all maintenance treatments in a given year

MC_i = Cost of maintenance treatment type i , per lane-mile

DC_i = Delay cost due to maintenance treatment type i , per lane-mile

SC_i = Safety cost due to maintenance treatment type i , per lane-mile

N = Number of treatments (associated with the strategy) carried out in a given year

PWF = Present worth factor at an interest rate (r) and number of years since rehabilitation (Y) that the treatment i is applied.

An important assumption made in the present study is that the costs of different treatments at a point in time, or the costs of the same or different treatments at different points in time are independent of each other, so the total cost is simply the arithmetic sum of the cost of the individual treatments.

- **Basic Cost:** The basic cost of each type of maintenance treatment was obtained using maintenance accomplishment cost models developed as part of this report.
- **Delay Cost:** The delay cost per lane-mile of each occurrence of a maintenance treatment type was calculated as follows:

$$\text{Delay cost for all vehicles} = \text{amount of delay per vehicle} * \text{unit cost of delay per vehicle} * \text{average number of vehicles affected for duration of the maintenance activity}$$

where,

$$\begin{aligned} \text{Amount of delay per vehicle} &= \text{increase in travel time in work zone} \\ &= 1\text{-mile}/(\text{decrease in travel speed in work zone}) \end{aligned}$$

Information on decrease in travel speed in work zone is provided in Appendix H.

Average number of vehicles affected for duration of maintenance activity

$$\begin{aligned} &= \text{average number of vehicles in 1 hour} * \text{average duration of maintenance activity } i \text{ in} \\ &\text{hours} = (AADT/24)*DUR_i \end{aligned}$$

$AADT$ = average AADT of pavement family under consideration

DUR_i = duration of maintenance activity per lane-mile (see Appendix H)

Values of delay costs (travel time values) are provided in Table 67 in Appendix H.

- **Safety Cost:** The safety cost of each occurrence of a maintenance treatment type was calculated as follows:

$$\text{Total safety cost} = \text{expected number of crashes during maintenance activity} * \text{unit cost of each crash}$$

where, Expected number of crashes during maintenance activity

$$\begin{aligned} &= \text{probability that a vehicle will get involved in a crash during the maintenance} \\ &\text{activity } (PC_i) * (\text{Average number of vehicles for duration of maintenance} \\ &\text{activity}) \end{aligned}$$

Average number of vehicles on road category for duration of maintenance activity
 = average number of vehicles in 1 hour * average duration of maintenance activity in
 hours = $AADT/(24 * DUR_i)$

PC_i was estimated on the basis of crash rates presented in Appendix H. It is assumed that the crash rates that re associated with each pavement family are increased by a factor of two during maintenance (see Appendix H). Unit crash costs that were used in the analysis are provided in Appendix H.

5.3.5.2 Determination of Life Cycle Benefit of Each Strategy

Reduced user vehicle operating cost due to smoother pavement due to maintenance was considered a benefit (negative cost), and the area under the performance curve is a surrogate for such benefits. The life cycle benefit of each strategy was computed as the increase in the area of the pavement performance curve due to the various jumps in the curve at points indicating maintenance activity. For small increments in time (1-year), it may be assumed that the performance curve is linear, and the benefit of each treatment is calculated as the area of a rectangle. The height of the rectangle is the jump in performance curve due a specific maintenance treatment (as determined from the short-term effectiveness models), and the length of the rectangle is the time that elapses till the end of service life. This was done for the entire life cycle of the pavement under that strategy. Another important assumption made in the current study is that the benefits of different treatments at a point in time, or the benefits of the same or different treatments at different points in time are independent of each other, so the total benefit is simply a sum of the benefits of the individual treatments. Salvage values were assumed to be zero.

5.3.5.3 Evaluation Cost-effectiveness of Each Strategy

The cost-effectiveness of each strategy was computed as the incremental cost benefit ratio (IBC) of that strategy relative to the zero-maintenance strategy as follows:

$$IBC_{STRATEGY_i} = \frac{LCB_{STRATEGY_i} - LCB_{ZERO-MAINTENANCE_STRATEGY}}{LCC_{STRATEGY_i} - LCC_{ZERO-MAINTENANCE_STRATEGY}}$$

where LCB and LCC represent life cycle benefit and life cycle cost, respectively.

5.3.5.4 *Selection of “Optimal” Strategy*

For a each pavement family, the “optimal” strategy was selected as the strategy corresponding to the highest cost-effectiveness. Plots of cost-effectiveness versus total maintenance cost were plotted to provide an insight into how the cost-effectiveness of various strategies vary compared to each other. A detailed discussion of results is presented in Chapter 8.

5.3.5.5 *Selection of “Optimal” Level of Preventive Maintenance Expenditure*

Besides attempting to identify the best combination of specific treatments as explained in Section 5.3.5.5 above, the study proceeded to determine the “optimal” level of preventive maintenance expenditure for each pavement family. This was done using plots of cost-effectiveness versus total maintenance cost, and determination of the annualized maintenance expenditures corresponding to the maximum value of cost effectiveness over pavement life cycle. A detailed discussion of results is presented in Chapter 8.

5.4 Methodology for Trade-off Modeling

5.4.1 Rehabilitation Interval /Maintenance Trade-offs

Rehabilitation/maintenance trade-off modeling was carried out for each of the five pavement families identified earlier in this chapter. The response variable used for these models was the rehabilitation interval in terms of years between construction and rehabilitation or between two successive rehabilitation activities. The explanatory variable is the annualized maintenance expenditure per lane-mile, in constant (1995) dollar value. However, two pavement sections in a given family with the same annualized maintenance expenditures may have very different intervals of rehabilitation because of different levels of annual traffic loading or/and different levels of weather severity. In other words, for a given level of maintenance and all other factors being the same, a heavily loaded rigid pavement located in a region of severe weather can be expected to have shorter intervals of rehabilitation than a lightly loaded rigid pavement located in a region of favorable weather. Therefore, considering only maintenance expenditure

may occlude the true relationship between rehabilitation and maintenance, and it is therefore necessary to “normalize” the maintenance variable using levels of traffic loading and weather severity.

The trade-off modeling was preceded by plotting the scatter of observed rehabilitation interval versus average annual maintenance, and then plotting rehabilitation interval versus average annual maintenance normalized in the manner described in the previous paragraph. The scatter plots provided hints of possible shape of the rehabilitation/maintenance trade-off model, and helped to decide which functional forms could be considered for further investigation. Only those functional forms that could yield reasonable engineering interpretation of model shapes were selected for consideration. These forms included the modified exponential curve, the logarithm curve, the logistic curve, and the Gompertz curve. Such models were expected to throw light on the following salient characteristics of maintenance variability:

- Zero-maintenance effect, i.e., what is the rehabilitation interval if no maintenance is carried out. This is usually taken as the intercept of the curve on the ordinate axis,
- Maintenance effectiveness cap, i.e., the longest rehabilitation interval that could be achieved regardless of how much maintenance is done. Geometrically, this is could be represented by the horizontal asymptote, if any, to the curve,
- Maintenance expenditure limit, i.e., the maintenance expenditure beyond which increases in expenditure yields insignificant additional benefit (increased rehabilitation interval). This is typically represented as the point on the abscissa that corresponds to the turning point of the curve.

5.4.2 Preventive Maintenance/Corrective Maintenance Trade-off

The objective of carrying out preventive maintenance on pavements is to ultimately extend the life of the pavement by reducing the rate of deterioration. Associated with this objective is the decrease in the levels of future corrective maintenance activities, as pavement longevity suffers if such corrective maintenance is not carried out. In Chapter 2, temporal trends in pavement maintenance expenditure by treatment role (preventive versus corrective) provided a hint that corrective maintenance increases as preventive maintenance decreases, and vice versa. An earlier study in Indiana showed that if higher levels

of preventive maintenance (crack sealing) are carried out in the Fall season, there is reduced amount of corrective maintenance (patching) the following Spring season [Sharaf and Sinha, 1984]. In the present study, to model the trade-off between preventive maintenance and corrective maintenance, the total preventive maintenance administered to a pavement over a 3-year period was determined, and expressed in constant dollars. This was used as an explanatory variable to predict the level of corrective maintenance that a pavement is expected to receive in the subsequent 3-year period. Therefore corrective maintenance expenditure for the pavement section in the 3-year period following preventive maintenance was calculated and used as the observed values for the response variable. Only one model was developed for all pavement families, regardless of weather regime and loading levels within each family. It is expected that future research can investigate this trade-off by functional class, surface type, region, and other pavement attributes. Indeed, the trends in maintenance expenditure by treatment role (Section 2.6.7 in Chapter 2) showed a systematic increase and decrease of the fractions of corrective and preventive pavement treatments, respectively, of the annual maintenance budget, as one goes from areas of relatively severe weather to areas of relatively mild weather. While this state of practice is not necessarily appropriate, it is indicative of the possible influence of weather severity on preventive/corrective maintenance trade-offs, and therefore furnishes a good argument for consideration of weather effects in such modeling. Future work in this area could involve further stratification of the preventive/corrective maintenance trade-off models with due consideration given to pavement type, functional class, weather regime, and loading, and other variables.

5.4.3 Functional Forms used for Trade-off Models

Linear models, which can be estimated by Ordinary Least Squares (OLS), are typically flexible enough to allow for a great variety in the shape of the resulting model. However, linear regression rules out several useful functional forms especially those are inherently non-linear and therefore cannot be “linearized” using an appropriate transformation. The difficulty of estimating models that are inherently non-linear is well known, but has been made relatively easy in recent years with the advent of user-friendly computer software.

A non-linear regression model is one for which first order conditions for least squares estimation of the parameters are non-linear functions of the parameters [Greene, 2000]. Therefore, non-linearity, as defined, relates to the techniques needed to estimate the parameters, and not the shape of the regression function. For instance, the general regression model in Equation 76 can be considered.

$$Y_i = h(X_i, \beta) + \varepsilon_i \dots\dots\dots (76)$$

Where

Y = is the response variable,

X is the explanatory variable, β is the parameter to be estimated, and ε is the error term, for the i th observation

The same values of the parameters that minimize (one-half of) the sum of squared deviations is given as follows:

$$S(\beta) = \frac{1}{2} \sum_{i=1}^n \varepsilon_i^2 = \frac{1}{2} \sum_{i=1}^n [y_i - h(X_i, \beta)]^2 \dots\dots\dots (77)$$

Where S is the sum of squared deviations. Equation 77 will yield the maximum likelihood estimators as well as the non-linear least squares estimators. The first order conditions for minimization of $S(\beta)$ are:

$$\frac{\partial S(\beta)}{\partial(\beta)} = - \sum_{i=1}^n [y_i - h(X, \beta)] \frac{\partial h(X_i, \beta)}{\partial \beta} = 0 \dots\dots\dots (78)$$

For the special case of the linear model, this will yield a set of linear equations, which can be solved for their parameters, but in the more general case, this will yield a set of non-linear equations that lack an explicit solution, and will therefore need an iterative procedure for solution.

Using the selected functional form for rehabilitation and maintenance trade-off for example, the first order conditions for estimating the parameters by non-linear least squares are found as follows:

Rehabilitation/maintenance functional form: $y_i = \beta_1 - [\beta_2 * (\beta_3 \wedge X_i)] + \varepsilon_i$

Based on this functional form, the first order conditions are as follows:

$$\frac{\partial S(\beta)}{\partial(\beta_1)} = -\sum_{i=1}^n [y_i - \beta_1 - \beta_2(\beta_3 \wedge X_i)] * 1 = 0 \dots \dots \dots (79)$$

$$\frac{\partial S(\beta)}{\partial(\beta_2)} = -\sum_{i=1}^n [y_i - \beta_1 - \beta_2(\beta_3 \wedge X_i)] * \beta_3 \wedge X_i = 0 \dots \dots \dots (80)$$

$$\frac{\partial S(\beta)}{\partial(\beta_3)} = -\sum_{i=1}^n [y_i - \beta_1 - \beta_2(\beta_3 \wedge X_i)] * (\log e \beta_3 * \beta_3 \wedge X_i) = 0 \dots \dots \dots (81)$$

As stated previously, these equations are non-linear in their parameters and therefore lack an explicit solution. Therefore they need to be solved iteratively. Computing the non-linear least squares estimator by minimizing the sum of squares is a standard problem in non-linear optimization that can be solved using any of several methods such as the Gauss-Newton procedure [Greene, 2000]. In each iteration the parameter estimates of the previous iteration are updated by regressing the non-linear least squares residuals on the derivatives of the regression functions. The process is said to converge when the value of the update is zero. However, it is worth noting that the algorithms sometimes get “locked up” in an errant iterate from which it is not possible to compute the residuals for the next iteration. The choice of seed values is very important in order to avoid such situations. Therefore the method adopted in the present study for solving such problems was first to fit the data to the functional form using a spreadsheet optimization tool, and then using the resulting coefficient estimates as seed values for non-linear least squares optimization in standard statistical software packages. This step produces the asymptotic standard errors and confidence intervals associated with the coefficient estimates, β_1, β_2 , and β_3 .

Unlike the case of linear regression, non-linear regression does not yield an unbiased estimate of σ^2 , the true variance of the error term ε from the regression residuals. Therefore the estimated coefficients are not normally distributed. As a result, the statistical tests used to estimate the fit of a linear regression model are not directly applicable to those estimated using non-linear techniques [Pindyck and Rubinfeld, 1991]. For instance, the t-statistics and the F-test cannot be used to perform hypothesis tests on the significance of the parameter estimates and the overall fit, respectively, of the non-linear model. Notwithstanding this, computer programs that perform non-linear estimation via the linearization approach typically calculate t-statistics or standard errors for the last linearization of the iterative process. Unlike the t and F statistics, the R^2 can be applied in its conventional sense to a non-linear regression model [Pindyck and Rubinfeld, 1991].

5.4.4 Determination of Marginal Effects

Trade-off analyses necessarily involve investigation of the marginal effects of “benefits” (increased rehabilitation interval, or decrease in levels of corrective maintenance) in response to unit, or specified increases in “cost” (such as increase in annual maintenance expenditure or increase in levels of preventive maintenance). Having obtained the function that explains the relationship between the benefits and costs, the marginal effects can be found either as a derivative of the rehabilitation/maintenance or the corrective maintenance/preventive maintenance trade-off functions. Derivatives measure the change in benefits per unit change in costs, while elasticities measure the percentage change in benefits in response to a unit percentage change in costs. The dimensionless feature of elasticities render that measure more attractive for comparison of the marginal effects of benefits and costs for various systems that have different units or different levels of a given unit of benefit or costs measurement, and was therefore used in the present study.

Computation of Marginal Effects:

Denoting elasticity of the response variable $f(x)$, or Y with respect to a given explanatory variable x as E ,

The general marginal effects model was derived as follows:

Elasticity (E_x) = % change in response variable / % change in explanatory variable

$$= \frac{\Delta f(x)}{f(x)} / \frac{\Delta x}{x}$$

$$= \frac{f(x + \Delta x) - f(x)}{f(x)} / \frac{\Delta x}{x}$$

$$= \frac{f(x + \Delta x) - f(x)}{\Delta x} / \frac{f(x)}{x}$$

$$E_x = \frac{x * f'(x)}{f(x)} \dots\dots\dots(82)$$

Where $f'(x)$ = derivative of the response variable with respect to the explanatory variable.

For the linear regression model, the elasticity is constant, regardless of slope. However, in the case of the non-linear model such as that obtained for maintenance rehabilitation trade-off, the value of elasticity depends on the value of x : the smaller the value of x , the smaller the elasticity, and the larger the value of x the larger the elasticity.

5.4.1.1 Elasticity of Rehabilitation Interval with respect to Maintenance (M) Load (L) and Weather Levels (W)

The selected functional form for maintenance/rehabilitation trade-off is as follows:

$$y = A - BC^x \dots\dots\dots(83)$$

$$\frac{dy}{dx} = -B \ln C * C^x \dots\dots\dots(84)$$

Elasticity is given as follows:

$$\begin{aligned} &= \frac{x * f'(x)}{f(x)} = \frac{(-B \ln C * C^x) * x}{A - BC^x} \\ &= \frac{(Bx \ln C * C^x) * x}{BC^x - A} \dots\dots\dots(85) \end{aligned}$$

5.4.1.1.1 Elasticity with Respect to Maintenance (E_M):

$$Y = A - B * C^{M/(L*W)} = A - B * C^{M/k}, \text{ where } k \text{ is a constant}$$

$$\begin{aligned} dY/dM &= dY/dU * dU/dM, \text{ (chain rule)} && \text{(where } Y = A - B * U \text{ and } U = C^{M/k}) \\ &= -B * \ln C^{M/k} * C^{M/k} \end{aligned}$$

$E_M = M * f'(M) / f(M)$ becomes:

$$E_M = \frac{-M * B * C^{M/k} * \ln C^{M/k}}{A - B * C^{M/k}} \dots\dots\dots(86)$$

Using the above expression the percentage change in a pavement's rehabilitation interval for a unit change in maintenance expenditure, given the weather and traffic level of the pavement, can be estimated.

5.4.1.1.2 Elasticity with Respect to Traffic Loading, (E_L):

$$Y = A - B * C^{M/(L*W)} = A - B * C^{k/L} \quad \text{where } k \text{ is a constant}$$

$$\begin{aligned} dY/dL &= dY/dU * dU/dV * dV/dL \text{ (where } Y = A - B * U \text{ and } U = C^V \text{ and } V = k/L) \\ &= (-B) * (\ln C^V * C^V) * (-k/L^2) \end{aligned}$$

$$= B * k * L^{-2} * C^{k/L} * \ln C^{k/L}$$

$E_L = L * f'(L) / f(L)$ becomes:

$$E_L = \frac{B * k * C^{k/L} * \ln C^{k/L}}{L * (A - B * C^{k/L})} \dots\dots\dots(87)$$

The above expression makes it possible to determine the impact of a unit change in traffic loading levels (due to loading regulation or deregulation), under specified levels of weather severity and maintenance expenditure.

5.4.1.1.3 Elasticity with Respect to Weather Severity, (E_W):

$$Y = A - B * C^{M/(L * W)} = A - B * C^{k/W}, \text{ where } k \text{ is a constant}$$

$$dY/dW = dY/dU * dU/dV * dV/dW \text{ (where } Y = A - B * U \text{ and } U = C^V \text{ and } V = k/W)$$

$$= (-B) * (\ln C^V * C^V) * (-k/W^2)$$

$$= B * k * W^{-2} * C^{k/W} * \ln C^{k/W}$$

$E_L = W * f'(W) / f(W)$ becomes:

$$E_W = \frac{B * k * C^{k/W} * \ln C^{k/W}}{W * (A - B * C^{k/W})} \dots\dots\dots(88)$$

Using the above expression, the incremental effects of continental or regional weather changes (such as global warming) on pavement rehabilitation interval, can be predicted.

5.4.1.2 Elasticity of Corrective Maintenance with respect to Preventive Maintenance

The selected functional form for preventive and corrective maintenance trade-off is as follows:

$$y = C - A^x B^x \dots\dots\dots(89)$$

$$\frac{dy}{dx} = C * (A^x B^x) * B^x * \ln B * \ln A \dots \dots \dots (90)$$

The elasticity of corrective maintenance with respect to preventive maintenance is as follows:

$$\begin{aligned} &= \frac{x * f'(x)}{f(x)} = \frac{(x * C * A^x B^x \ln B \ln A)}{C * A^x B^x} \\ &= x * B^x \ln B * \ln A \dots \dots \dots (91) \end{aligned}$$

5.5 Chapter Summary

The framework, methods adopted, and underlying theories for each of the three major aspects of the study are discussed in this chapter. Evaluation of the effectiveness of maintenance in the short-term was preceded by an overview of current practices of pavement defect diagnosis and prognosis during inventory at a sub-district or district level. This enabled a clear demarcation of the separate times associated with pavement inventory, maintenance decision, maintenance execution (expenditure), and maintenance effectiveness. Maintenance effectiveness was defined as either the instantaneous change in pavement condition, the change in condition from one year to the next, or the reduction in the rate of pavement deterioration across a two-year period due to maintenance treatment. The effect of non-consideration or inadequate consideration of the relative timing between maintenance in a given year and condition monitoring measurements in that year, were investigated. Mathematical formulas were derived for estimating each of the three measures of short-term maintenance effectiveness, for each of the two relative timing scenarios (maintenance before monitoring, and maintenance after monitoring). Also, relationships between each pair of maintenance effectiveness measures were derived, so that given the value of one, the other can be calculated under a certain given assumptions.

This chapter also discusses how the present study evaluates the effectiveness of maintenance in the long-term. Methods of developing pavement performance curves, for each pavement family, were described, and the manner of formulation of maintenance strategies (none, one or several treatments spaced out over the entire life of the pavement) was discussed. Steps used to calculate incremental area due to any strategy, given the effectiveness of each treatment in the short-term and the pavement performance curve, were described in this chapter. The computation of overall costs associated with each strategy, i.e., maintenance treatment accomplishment costs and delay/safety costs associated with work zones was described. Long-term effectiveness of a strategy was defined as the incremental benefit-cost associated with that strategy, i.e., increase in benefits over the zero-maintenance strategy, divided by increasing in costs over the zero maintenance scenario. For each pavement family, simulation modeling, using the performance trend and jump functions, was utilized to determine the cost-effectiveness of various strategies, and plots of cost-effectiveness versus maintenance expenditure were plotted. Finally, the chapter shows how optimization tools were used to determine the most cost-effective strategy for each pavement family.

To investigate existence of relationships between rehabilitation intervals and maintenance, simple scatter plots were used to obtain an initial insight, and this was followed by examination of the scatter plots when maintenance is normalized by extenuating factors such as traffic loading and weather severity. Functional forms for modeling were chosen in order to fit the data points in a satisfactory manner while providing means for engineering interpretation of the model shape. Such engineering interpretation includes the maintenance effectiveness cap, the maintenance expenditure limit, and the zero-maintenance effect. Methods for marginal effects analysis to estimate the impacts of marginal changes in maintenance expenditure (or changes in load and weather severity) were presented in this chapter. Finally, methods of investigating relationships and developing models for trade-off between preventive maintenance and subsequent corrective maintenance were provided.

CHAPTER 6: DATA COLLECTION AND COLLATION

6.1 Data Collection

Pavement performance is subject to a wide gamut of stress-related and strength-related factors, which act holistically to explain performance at a given point in time. In this respect, any study that professes to investigate the effectiveness of pavement maintenance must necessarily be preceded by a comprehensive acquisition of as much data on all of such factors, as possible.

Data collection for the present study covered pavement-related characteristics for approximately one-half of the entire state highway network. The culmination of this effort was a Joint Transportation Research Program (JTRP) pavement data warehouse (INDIPAVE 2000) that consists of data from varied sources such as Program Development Division's PMS, Operations Support Division's MMS, FHWA's HPMS, Purdue University Department of Agronomy's Climate Center, and other data sources. Data already contained in INDOT's PMS database, mainly pavement condition and linear referencing data, served as a nucleus around which INDIPAVE 2000 was built. The tasks of data collection and collation included collection and processing of raw data, physical and logical design of the database, and database development.

6.1.1 Description of Data Collected and Sources

Table 6-1 shows the various data types collected in relation to their usage in various aspects of the study. The table also indicates the source of each data category and/or type. This is followed by a description of the manner in which raw data was collected from the various sources.

Table 6-1: Application of Collected or Processed Data

Aspect of the Study		Short-term Effectiveness	Long-term Effectiveness	Trade-off Analyses	Case Studies
Data Category	Data Type				
ROAD SEGMENT IDENTIFICATION	<u>For Each Segment:</u> Segment ID # Starting milepost Ending milepost Functional Class National Highway System Class Number of lanes	●	●		●
PAVEMENT CONDITION	<u>For Each Segment and Year:</u> International Roughness Index Rutting Index Pavement Quality Index Pavement Condition Rating Present Serviceability Index Cracking Index Faulting Index	●	●		●
TRAFFIC DATA	<u>For Each Segment and Year:</u> Traffic Volume % Single Unit Trucks % Multiple Unit Trucks Gross Vehicle Weight ESAL (and Cumulative ESAL) Average Operating Speed	●	●	●	●
DESIGN & CONSTRUCTION FEATURES	<u>For Each Segment:</u> Surface Type and Thickness HMAC Asphalt Content Air Voids in HMA PCC Elasticity Modulus Layer types and Thicknesses Subgrade %Fines, CBR, etc.	●	●		●
GEOTECHNICAL DATA	<u>For Each Segment:</u> Natural Ground (if different from subgrade) %Fines, California Bearing Ratio, etc. <u>For each County:</u> Surface Geology	●	●		
CLIMATE DATA	<u>For Each county:</u> Normal Air Temperature Normal Precipitation Air Freeze-Thaw Cycles Air Freeze Index Age	●	●	●	
ROUTINE MAINTENANCE DATA	<u>For Each Segment and Year:</u> MMS Segment Reference Treatment Types Treatment Levels Treatment Costs	●	●	●	●
REHABILITATION DATA	<u>For Each Segment and Rehab. Year:</u> Rehabilitation Type Rehabilitation Expenditure Thickness of New Layer		●	●	●

6.1.1.1 Road Segment Identification Data

There are two systems in the INDIPAVE database by which a road segments can be assessed for their data: by their Milepost Datafile segment identification numbers (which are numbered from 1 to 9901), or by their Contract Datafile segment identification numbers (which are numbered from 1 to 930).

Each segment number in the Milepost Datafile corresponds to a milepost number established by INDOT's Linear Referencing System (LRS), which assigns a reference number to each individual 1-mile pavement section on the network. Each segment in the Contract Datafile, on the other hand, corresponds to the code number of the last rehabilitation contract carried out on that section.

6.1.1.2 Pavement Condition

The Pavement Condition data-file includes data on the standard aggregate measures of pavement deterioration. These include the International Roughness Index (IRI), Present Serviceability Index (PSI), Pavement Condition Rating (PCR), and Pavement Quality Index (PQR) , as well as a few disaggregate measures (Cracking Index and Faulting Index). Data on roughness (IRI) are available for most sections from the year 1994 to 1999; PCR and PQR data are available only for 1994 and 1999. PSI data was derived from IRI values using established relationships [Darter et al., 1994; Gulen et al., 1994].

6.1.1.3 In-House (Force Account) Maintenance Data

In-house maintenance covers not only routine maintenance activities such as crack sealing, but also includes periodic maintenance activities such as chip sealing. INDOT's Operations Support Division (OSD) is responsible for supervising all in-house maintenance activities. This includes maintenance of the pavement and shoulders, road furniture, drainage facilities and right-of-way of all roads on the state highway network. The state is divided into 38 maintenance zones, known as "highway sub-districts" that are equipped with requisite plant, manpower, and resources to carry out maintenance work. Work done by the sub districts is recorded on crew day cards, from which summarized data is collected and entered into the MMS database. The Central Office of the Operations Support Division, based at Indianapolis, oversees the planning, scheduling, and performance monitoring of maintenance activities at

the various sub districts. This office also synthesizes and stores maintenance information on expenditure, resource usage, productivity, and other information. The information is stored on storage media such as floppy disks and accessed with the aid of Operations Support Division's Work Management System, a Cobol-based software that enables the user to customize the maintenance information in any desired reporting format.

For each fiscal year and for each district, relevant information that was extracted for this study are as follows:

- i. Activity Data File: This an inventory of the activities performed within each sub district, synthesized from the crew day cards submitted by the foremen and work supervisors. Information such as service levels, average daily production, features and resource data assignments are available on this file.
- ii. Location List File: This is a record of features and activities at specific faculties and locations that are significant enough to track their maintenance costs individually, e.g., it may be sought to record what work was done during the past year on a specific stretch of road. The location list identifies each location so that the MMS software can record costs and work accomplished.

From the above MMS files, data was synthesized as follows to yield, for each milepost (or contract) segment, type, level, and cost of work activity in each 6-month period, i.e., Fall (August-December) and Spring (January to June). Non-pavement routine maintenance activities were excluded from the synthesis.

6.1.1.4 Traffic Data

Traffic data collected for the study included traffic volume, % single unit trucks, % multiple unit trucks, gross vehicle weight, ESAL (and Cumulative ESAL), and average operating speed. Data was collected from the Statistics Unit of INDOT's Program Development Division, either in the form of periodic publications, or by accessing data outputs from field measurements. The Traffic Statistics Unit is

responsible for the collection, processing, and analysis of traffic data such as vehicle counts, vehicle classification, and truck weights.

Traffic volumes (in terms of AADTs) on state road segments are available in County Flow Map publications, which are released every year. Each year's publication shows the most current AADT on each state highway segment. The AADT's reported in these documents are derived from raw 48-hour vehicle counts that are carried out under the statewide coverage count and HPMS programs. These counts are adjusted using relevant growth and seasonal factors generated from the Automatic Traffic Recorders (ATR) and Weigh-in-Motion (WIM) stations, to yield annualized statistics.

Statewide Vehicle Classification reports, also released by the Traffic Statistics Unit, provide data on the split of vehicles according to the FHWA vehicle classification scheme. These reports are available in a summarized form (where only percent commercial vehicles) are provided, or in a detailed form, where vehicular traffic is categorized by each level of the entire FHWA vehicle classification range. The source of the data for these reports is the Coverage Count and HPMS programs. However, the WIM stations (and since 1998, the ATR stations) provide vehicle classification data at a total of 92 locations. Vehicle classification under both programs is carried out on the basis of vehicle axle configuration, and not by vehicle length.

Both AADTs and classifications are reported on a segment-by-segment basis. A "segment" is defined as the road section between two major intersections, and a major intersection is where two or more state roads cross, as an interchange, intersection or overpass. By virtue of this definition, some segment lengths in the database are less than the unit of measurement (1-mile).

Monitoring of traffic weights, unlike volume and classification is rather limited: raw data on truck weight are collected only at the 35 WIM sites in the State (as of the year 2000). Even though these stations are distributed across road functional classes and regions, their statewide coverage (only 35 sites) is far inferior to that of the coverage count sites. Unlike the other two count types, truck weight monitoring is not carried out at the coverage count sites (as truck weight reporting is not yet a federal HPMS requirement). Therefore, in order to estimate pavement loading levels (gross vehicle weights) due to traffic at various points on the state highway network, models were developed, using WIM data, to estimate levels of total

traffic loading (gross vehicle weights) as a function of road functional class, region, and primary data (volume and classifications) generated from the statewide coverage counts. Pavement loading data in terms of ESALS, a more generally used measure, were computed using factors derived by analyzing data at the Weigh-in-Motion sites in 1980s and in the year 2000 [Gulen et al., 2000].

6.1.1.5 Pavement Rehabilitation and Maintenance Contract Data

INDOT's Program Development Division (PDD) supplied data on the costs, types, dates, and other details on resurfacing and other contract activities designed to improve pavements on the state highway network. These data, obtained from hard copies supplied by this division is not yet stored in PDD's PMS database. Such contract activities covered not only rehabilitation (overlays), but also included maintenance work of a localized (minor) scale (such as concrete patching and under-sealing), moderate scale (e.g., chip sealing), and major maintenance (thin HMAC overlays). Such data, which was obtained from construction record files at the PDD, included the dates and location of the contracts, the expenditure involved, the length of construction, layer thicknesses (in case of overlay), and in some cases, specification of materials used. The division's annual pavement surface reports were also a valuable source for locations of resurfacing contracts as well as the age and pavement condition at the time the contracts were carried out.

6.1.1.6 Geotechnical/Subgrade Data

For certain classes of highways, the subgrade is not the natural ground, but a thick (typically 24 inches) layer of imported fill material. Data on the mechanical properties of the subgrade were obtained from soil test results found in previous geotechnical site investigation reports. Such investigations, a necessary prerequisite to reconstruction or lane-widening, have been carried out at various locations on the state roads. Where imported fill material was used to replace or fill over the existing natural ground, efforts were made to collect soil test results from the construction division of INDOT.

6.1.1.7 *Climate Data*

Historical data on spatial and temporal weather patterns in the State of Indiana were obtained from the Indiana Climatic Center Internet Web Site maintained by the Agronomy Department of Purdue University [NCDC, 2000]. Raw data such as daily air temperatures and precipitation were collected. These were later processed to obtain secondary weather statistics such as freeze index, number of freeze thaw cycles, average winter temperature, annual precipitation, etc., using established methods [Huang, 1993]. Data was obtained for each county, and all highway segments located in a county were assigned the weather attributes of that county. At the time of the present study, data on pavement temperatures, from which pavement freeze-thaw could be computed, were not available.

6.1.2 **Challenges Encountered in Data Collection**

6.1.2.1 *Differences in Referencing Systems of INDOT's PMS and MMS.*

INDOT's PMS and MMS, as previously mentioned, were valuable sources of information for pavement condition and in-house maintenance activities, respectively. However, a few problems arose during the consolidation of data from the PMS and the MMS, as these systems use different referencing schemes. While PMS uses a linear referencing scheme based on mileposts, MMS uses a system that defines a road segment as a section between two road intersections within the same county, or an intersection of two roads and a county line, or a section of road between two state or county lines. A general form of the MMS referencing scheme is as follows:

W-X-Y-Z

Where: *W* is the road functional class (I- Interstates, S- Other State Highways)

X is the road name or number, e.g., for I-65, *X* is 65

Y is the county code (from 1 to 92)

Z is the count of the segment along the road in a South-North or West-East direction, from the Southern-most and Westernmost county line, respectively.

On the other hand, PMS uses the linear referencing scheme, in which a road is divided into 1-mile segments in increasing order from the south to the north, or from the east to the west. For instance I-65, RP

45+00 to 46+00 is that point on the I-65 that is between 45 and 46 miles from the southernmost point of the I-65 (i.e., where it intersects with the Kentucky State line). In order to solve his problem, the state map of Indiana was used as the common reference. On this map, work locations indicated in the PMS and MMS were plotted. This way, the maintenance segment that corresponds to each contract segment was determined.

6.1.2.2 Differences in Reporting Periods

The second problem is the difference in reporting periods for pavement condition (from the PMS) and that for pavement maintenance (from the MMS). Data for the PMS such as pavement roughness, is reported on calendar year basis. However, MMS utilizes a fiscal year reporting scheme, from July of one year to July of the next. Fortunately, MMS summary reports are available for each half fiscal year, so it was possible to obtain data for a calendar year by adding the data from the second session (Spring) of a fiscal year to the first session (Fall) of the following fiscal year. This way it was possible to view maintenance data either by fiscal year, calendar year, or even for every 6-month period.

6.2 Data Preparation

In most cases it was not possible to use raw data directly for the analyses, and therefore it was necessary to convert primary data into secondary data that was more useful for the study. An elaborate and extensive effort was therefore expended on processing of the collected data to yield summary statistics or parameters of interest to the study. Data for which significant processing was carried out were as follows:

- Traffic loading (in terms of ESALs) for each segment at each intermediate year between successive years of ESAL factor derivation (i.e., 1980 and 2000).
- Traffic loading (in terms of Gross Vehicle Weight) for each segment at each year
- Weather attributes (Number of Freeze-Thaw Cycles and Freeze Index) from daily temperature records.

6.2.1 Determination of Levels of Traffic Loading

The Equivalent Single Axle Load (ESAL) concept, which is a measure of the damage caused by an equivalent traffic load on a pavement, has been used by the Indiana Department of Transportation (INDOT) to design its pavements. The ESAL values currently used were prepared and validated in the late 1970's. However, in response to the trend in pavement loading over the past two decades, Gulen et al. [2000] estimated new average ESAL values that better reflects loading levels on the State's highway pavements. However, because the study period ranged between 1992 and 1999, it was decided to model the rate of change of ESALS with time, rather than to use values for either period or average of the values, to estimate ESALS for the pavement sections at any year within the study period.

ESAL factors were developed by INDOT using data from Indiana's 35 Weigh-in-Motion stations and methodology established by AASHTO methods. Factors were developed separately for flexible and rigid pavement types [Gulen et al., 2000]. For flexible pavements, a structural number of 5 was assumed, while a slab thickness of 10 inches was used for rigid pavements. For both pavement types, a terminal PSI of 2.5 was assumed. The study results indicated, among others, that the average factors varied by functional class (Interstates vs. non-Interstates), but did not provide a further break down of the factors by functional class. Factors were developed separately for Multiple Unit Trucks and Single Unit Trucks. Table 20 presents the ESAL factors that were developed in the Gulen et al. study and compares the results to the factors developed in 1980.

Table 6-2: Comparison of 1980 and 2000 ESAL Factors [Gulen et al., 2000]

Pavement Type	Vehicle Class	1980 ESAL Factors	2000 ESAL Factors (based on 1998-1999 data)
Flexible	Single Unit Trucks	0.316	0.600
	Multiple Unit Trucks	0.860	1.300
Rigid	Single Unit Trucks	0.230	0.900
	Multiple Unit Trucks	1.115	2.000

Figure 6-1 below shows the variation of ESAL factors from the late 1970's to 1999, for the various pavement types and vehicle classes. A linear growth of ESAL factors is assumed.

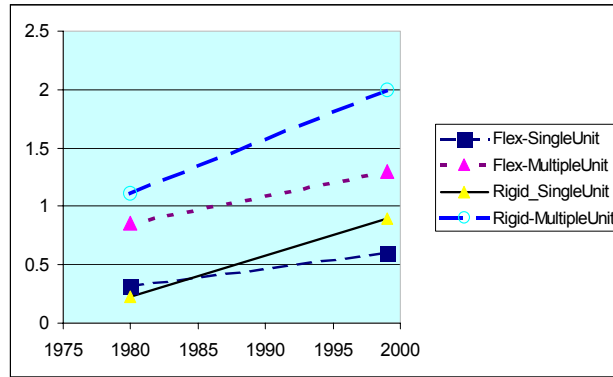


Figure 6-1: Variation in ESAL Factors, 1980-1999 [Gulen et al., 2000]

Using the linear trend assumption that yielded the ESAL growth pattern shown in Figure 6-1, the ESAL factors can be extrapolated for any intermediate year as follows:

$$ESAL_{YEAR_i} = \frac{ESAL_{1999} - ESAL_{1980}}{19} * (YEAR_i - 1980) + ESAL_{1980} \dots\dots\dots(92)$$

With the above linear interpolation formula, ESAL factors for any of both types of vehicles on any of both pavements and in any year between 1980 and 1999 were found for each pavement section in the database and at each year. Relevant formula for each category are tabulated below:

Table 6-3: Formula for ESAL Factor Interpolation

Pavement Type	Vehicle Class	Formula for ESAL Interpolation (1980-2000)
Flexible	Single Unit Trucks	$ESAL_{YEAR_i} = 0.0098 * (Year\ i - 1980) + 0.316$
	Multiple unit Trucks	$ESAL_{YEAR_i} = 0.0152 * (Year\ i - 1980) + 0.86$
Rigid	Single Unit Trucks	$ESAL_{YEAR_i} = 0.0231 * (Year\ i - 1980) + 0.23$
	Multiple unit Trucks	$ESAL_{YEAR_i} = 0.0305 * (Year\ i - 1980) + 1.115$

The AASHTO formula for calculating ESALs is as follows [Mannering and Kilareski, 1998]:

$$\begin{aligned}
 \text{Total Annual ESALs} &= (\text{Annual Volume of Multiple Unit Trucks} * \text{ESAL factor for Multiple Unit Truck}) \\
 &\quad + (\text{Total Annual Volume of Single Unit Trucks} * \text{ESAL factor for Single Unit Truck}) \\
 &= \text{Overall AADT} * 365 * [(\% \text{ MUT} * \text{MUT}_{\text{ESAL_FACTOR}} + (\% \text{ SUT} * \text{SUT}_{\text{ESAL_FACTOR}})] \dots\dots (93)
 \end{aligned}$$

Equation (93) is modified to incorporate the effect of directional distribution of traffic (D_d) and relative lane occupancy (L_f) as follows:

$$\begin{aligned}
 \text{Total Annual ESALs} &= \text{Overall AADT} * 365 * (D_d * L_f [(\% \text{ MUT} * \text{MUT}_{\text{ESAL_FACTOR}}) \\
 &\quad + (\% \text{ SUT} * \text{SUT}_{\text{ESAL_FACTOR}})] \dots\dots\dots (94)
 \end{aligned}$$

For a flexible pavement in a given year i , Equation (94) becomes:

$$\begin{aligned}
 \text{Total Annual ESALs} &= \text{Overall AADT} * 365 * (D_d * L_f [(\% \text{ MUT} * \{0.0152 * (\text{Year}_i - 1980) + 0.86\}) \\
 &\quad + (\% \text{ SUT} * \{0.0098(\text{Year}_i - 1980) + 0.316\})]] \dots\dots\dots (95)
 \end{aligned}$$

For a rigid pavement in a given year i , Equation (95) becomes:

$$\begin{aligned}
 \text{Total Annual ESALs} &= \text{Overall AADT} * 365 * (D_d * L_f [(\% \text{ MUT} * \{0.0305(\text{Year}_i - 1980) + 1.115\}) \\
 &\quad + (\% \text{ SUT} * \{0.0231(\text{Year}_i - 1980) + 0.23\})]] \dots\dots\dots (96)
 \end{aligned}$$

- Where D_d and L_f and are directional and lane factors respectively.
 MUT = Number of Multiple Unit Trucks (FHWA Classes 8-13)
 SUT = Number of Single Unit Trucks (FHWA Classes 5-7)
 Year_i = Year for which ESAL is sought
 Values for D_d and L_f are shown as follows:

Table 6-4: Values of Lane Occupancy Factor, L_f [TRB, 1994]

Number of Lanes in One Direction	Lane Occupancy Factor
1	1.0
2	0.8-1.0
3	0.6-0.8

Table 6-5: Adjustment Factors for Directional Distribution on Two-Lane Highways, D_d [TRB, 1994]

Directional Distribution	100/0	90/10	80/20	70/30	60/40	50/50
Adjustment Factor	0.71	0.75	0.83	0.89	0.94	1.00

6.2.2 Grouping (Regionalization) of Pavement Sections by Weather Conditions

As discussed in Chapter 5, inclusion of climatic characteristics in pavement analyses may be done using individual weather attributes, a single weather index to represent all weather effects, or using dummy variables to represent various climatic regions within the study area. As part of SHRP's LTPP program, states were grouped into zones having similar environmental features, especially with regard to wetness and freezing, as shown as Figure 6-2.

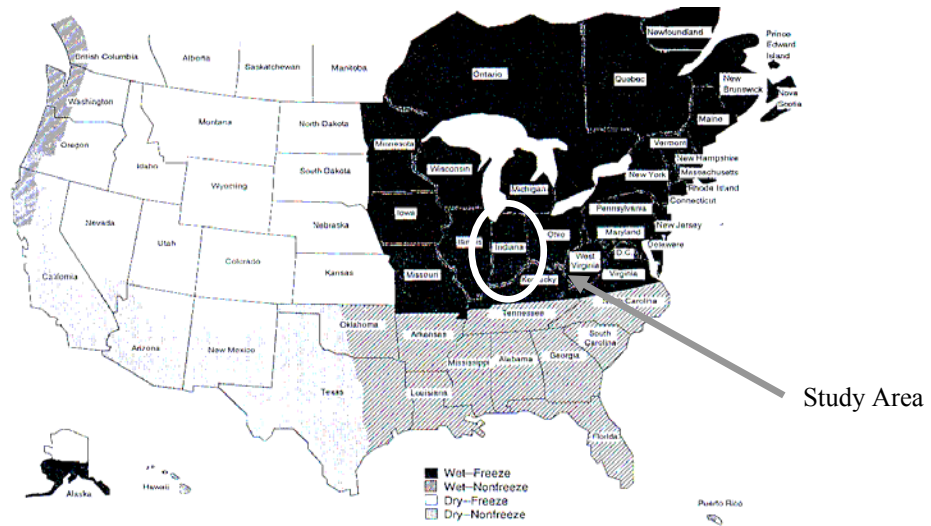


Figure 6-2. Climatic Zones for the SHRP-LTPP Experiments

The LTPP characterized the State of Indiana as having a wet-freeze environment [Daleiden *et al.*, 1993]. This characterization based on threshold minimum precipitation and freeze index of 508 mm/year and 83°C days respectively, was obviously made relative other regions in the country. Data from the Indiana Climate Center confirm that most parts of Indiana indeed experience a considerable amount of precipitation and freezing temperatures during certain parts of the year. As mentioned in Chapter 2, freezing temperatures are generally experienced between November to March, while relatively warm temperatures occur from April to October. The November-March period is not characterized by a permanently frozen state, but rather experiences a series of freeze/thaw cycles, especially at the fringes of the cold season. The average freeze-free period ranges from 179 days in northeastern Indiana to 199 days in the southwest [Fenelon *et al.*, 1994]. The interaction of tropical and polar air masses over the state typically results in significant levels of precipitation. Precipitation occurs almost throughout the year, but is somewhat greater during March-July due to the frequency and intensity of showers in this period. Average annual precipitation is about 39 inches, but is about 5 inches higher than average in the south and 5 inches below average in the North.

Notwithstanding the above generalized characterization of Indiana's weather, an argument can be made for a climatic subdivision of the state. The state covers over 200 miles in the North-South direction, and over 100 miles in the East-West direction. There are significant differences in its physical geography features, from north (near the great lakes) to the central (the plains) and to the south (the mountain ranges). From [Figure 6-2](#) it is seen that the state is only approximately 100 and 60 miles away from neighboring zones characterized by LTPP as "dry-freeze" and "wet non-freeze", respectively. Also [Figure 6-3](#) shows that the depth of frost penetration, a major determinant in pavement failure varies by as much as from 30 inches in southern Indiana to 65 inches in the northern part of the state [Yoder and Witczak, 1975]. Furthermore, the freeze index (a measure of severity of frost in a region, in degree-days), varies considerably across the state. These trends suggest that there are significant variations in weather patterns across the state to warrant division of the state into zones on the basis of the weather, rather than adopting LTPP's implied blanket characterization of the state as a uniform wet-freeze environmental zone.

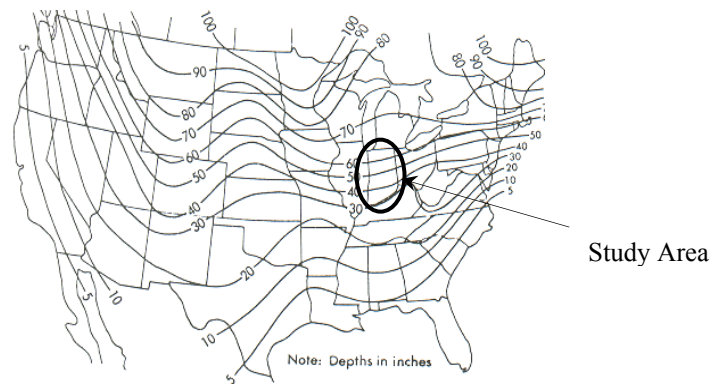


Figure 6-3: Variation of the Depth of Frost Penetration in Inches [Yoder and Witczak, 1975]

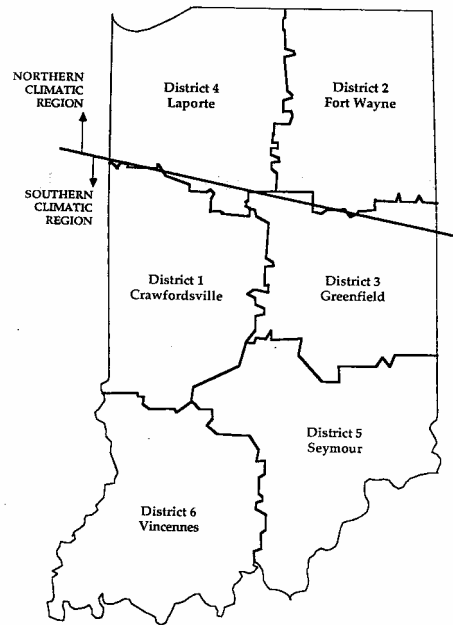


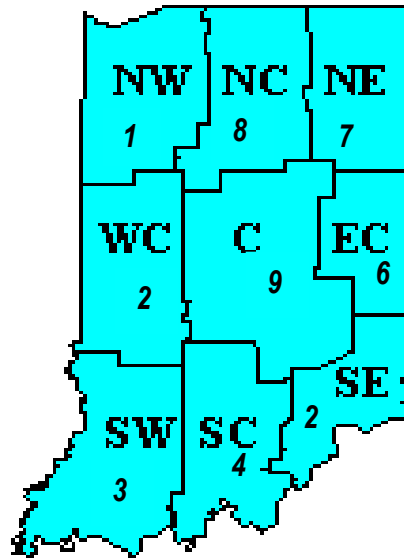
Figure 6-4: Climatic Zoning System used in Previous Studies [Mouaket and Sinha, 1990]

Previous pavement studies specifically carried out for the state of Indiana have used weather zone boundaries that were roughly based on highway administrative districts (see [Figure 6-4](#)). Highway administrative districts lying next to each other in a horizontal direction were taken as a group, yielding 2 zones. Such zoning efforts were not inappropriate, given data limitations at the time. However, that system of zoning needs to be checked from a statistical perspective, and modified if necessary. This was carried out in the present study.

This part of the present study demarcates the state into various climatic zones of relative wetness and freeze conditions. This is done using a statistical technique called multivariate cluster analysis. In this procedure, elements are grouped into clusters according to the similarity (or dissimilarity) of their attributes. Clusters are formed in such a way that within each cluster of elements, there is minimum variability of element attributes, while there is maximum variability from one cluster to another.

Elements: The elements used for clustering were weather divisions, each of which is a collection of counties ([Figure 6-5](#)).

Attributes: The attributes used for clustering were precipitation and temperature. Precipitation and temperature data are available in the form of “normal” and “average” values.



[Figure 6-5](#): Indiana Climatic Center Weather Divisions [NCDC, 2000]

6.2.2.1 *Data for Weather-based Clustering*

Climatologists use the term “normal” to refer to weather statistics calculated over a standard 30-year time interval. Current normals are based on available weather observations taken during the years 1961-1990. Normals are updated at the end of every decade, and new normals are calculated based on observations made from 1971-2000.

“Average” values shown for each month represent the mean of temperatures over the 30-year period for that month. The use of yearly averages was avoided as that statistic obviously masks temperature variations within the year, cannot be used as an effective attribute for clustering. Temperature data are available in the form of average maximum, average minimum, and average mean monthly temperatures, while precipitation data are in the form of average monthly values. Climate data are presented in 9 geographical divisions, as shown as [Figure 6-5](#). This section of the study seeks to aggregate any two or more of these divisions to form larger zones. The study uses cluster analysis and statistical significance testing to achieve this objective.

6.2.2.2 *Method Used for the Clustering Process*

Cluster procedures identify hierarchical clusters of observations in data-sets, such that there is maximum homogeneity within each cluster and maximum heterogeneity between clusters. Cluster analysis techniques include Average Linkage, Complete Linkage, Density Linkage (including Wong’s Hybrid and Nearest-neighbor methods), Maximum-Likelihood for mixtures of spherical multivariate normal distributions, and Ward’s minimum variance method. The differences between the various clustering techniques lie in the method of calculating the “distance” between any two clusters, but all of them are based on the usual agglomerative hierarchical clustering procedure. Each observation, or element, begins in a cluster by itself i.e., at the initial stage the number of clusters equals the number of elements. Then the two closest clusters are merged to form a new cluster that replaces the two old clusters. Merging of the clusters is repeated until the specified number of clusters is reached, or until the specified level of similarity between clusters is attained.

Before performing cluster analysis on elements having two or more attributes measured on different scales such as precipitation and temperature data, it is necessary to consider scaling or transforming the variables. If this were not done, temperature data (which have relatively large absolute values and variances) would unduly have more effect on the resulting clusters than precipitation data. Therefore monthly factors, or ratios as defined below, were used.

$$MPF_i = AMP_i / AMP_{for\ all\ zones} \dots\dots\dots (97)$$

Where MPF = Monthly precipitation factor for zone i
 AMP = Average monthly precipitation for zone i , in inches
 $AMP_{for\ all\ zones}$ = Average monthly precipitation for all 9 zones, in inches

Monthly factors for minimum, average, and maximum temperature were defined in a similar fashion. The rationale for using monthly factors, rather than the raw monthly values for cluster analysis, was to bring temperature and precipitation values to a common scale for clustering while preserving month-to-month as well as inter-zonal variations within these data values.

6.2.2.3 Results

The cluster formed by the analysis ate shown in the cluster dendrogram (Figure 6-6) and Table 6-6.

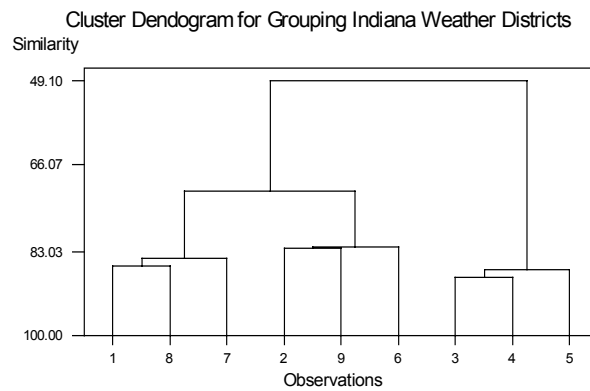


Figure 6-6: Cluster Dendrogram Formed using both Temperature and Precipitation Attributes

Table 6-6: Clusters Formed using both Temperature and Precipitation Attributes

Cluster	Composition (Observation #s and descriptions)
1	#1 (Northwestern) #8 (North Central) #7 (Northeastern)
2	#2 (West Central) #6 (East Central) #9 (Central)
3	#3 (Southwestern) #4 (South Central) #5 (Southeastern)

The above analyses using temperature and precipitation data showed a grouping of the State of Indiana into 3 environmental regimes as follows:

1st Cluster: Northwestern, North Central, and Northeastern ICC divisions

2nd Cluster: West Central, East Central, and Central ICC divisions

3rd Cluster: Southwestern, South Central, and Southeastern ICC divisions

These are illustrated as in Appendix K. Table 6-7 shows the composition and characteristics of each climatic region. The constituent counties, as well as the characteristics of each climatic zone, are indicated. This provided a means to study long-term effectiveness or trade-off analyses by region, and is an alternative to using weather severity indices to individual weather parameters for such analyses.

Appendix L shows an alternative grouping of the state using county, rather than ICC zone, climate data. This grouping scheme shows a rather large northern region, and a relatively small southern region that consists primarily of counties in the lower Wabash basin in the south-western part of the state. The climatic regions based on ICC zones were used in some aspects for the present study for grouping the state's pavements on the basis of climatic characteristics because they are close approximations of the highway administrative regions, as well as the regionalization of the state based on topography and surface geology (see Chapter 2).

Table 6-7: Climatic Regions Formed Using Cluster Analysis

Region	ICC Climatic Zones	Description	Characterization	Counties
1	Northwestern North Central Northeaster	Relatively lower temperatures Relatively little precipitation	Low Wetness/High Freeze/ Low Freeze-thaw	Lake, Newton, Porter, St. Joseph, Laporte, Elkhart, Steuben, Starke, Marshall, Kosciusko, Noble, Dekalb, Pulaski, Fulton, Whitney, Allen, Benton, White, Cass, Carroll, Miami, Wabash, Huntington, Wells, Adams.
2	West Central Central East Central	Relatively lower temperatures Relatively little precipitation	High Wetness/High Freeze/ High Freeze-thaw	Warren, Fountain, Vermilion, Vigo, Clay, Parke, Tippecanoe, Montgomery, Putnam, Owen, Clinton, Boone, Hendricks, Morgan, Howard, Tipton, Hamilton, Marion, Morgan, Johnson, Shelby, Hancock, Madison, Grant, Blackford, Delaware, Henry, Rush, Decatur, Jay, Blackford, Randolph, Wayne, Fayette, Union, Franklin.
3	Southwestern South Central Southeastern	Relatively higher temperatures Relatively greater precipitation	High Wetness/ Low Freeze/ High Freeze-thaw	Sullivan, Knox, Gibson, Posey, Vanderburg, Warrick, Pike, Daviess, Greene, Martin, Dubois, Spence, Perry, Crawford, Orange, Lawrence, Monroe, Brown, Jackson, Washington, Floyd, Harrison, Clarke, Scott, Jennings, Jefferson, Ripley, Dearborn, Ohio, Switzerland.

6.2.3 Development of Index for Weather Severity

As stated in Chapter 5, there are three ways in which weather factors can be considered for pavement performance or maintenance effectiveness studies: By using a dummy variable representing climatic region, using disaggregate weather parameters such freeze index, or by using a single aggregate index that embodies the effect of all weather factors, such as the climatic coefficient, “*m*”, used in World Bank’s HDM model [Paterson and Attoh-Okine, 1993]. Climatic variations across Indiana, even from the northern tip to the southern tip of the state, may not be large enough to warrant the use of even two different values of the “*m*” coefficient for pavements in the state. Even SHRP’s LTPP characterized the entire state as a uniform wet-cold region [Daleiden et al., 1993]. However, as has been aptly recognized earlier in the previous section, there may be significant variations in weather across the state to cause marked variations in weather induced pavement deterioration, the most notable being the fact that freeze index varies from 0 in southern Indiana, to 650 degree-days in northern Indiana.

A questionnaire survey of sub-districts and districts was carried out to obtain the perceptions of pavement managers on the relative weights of pavement deterioration factors including weather variables,

among others. Based on the average weight, w_i assigned to each factor i , an index was developed to represent weather severity in each weather zone (county) as follows:

$$\text{Index} = w_P * P + w_{FI} * FI + w_{FTC} * FTC \dots\dots\dots (98)$$

Where w_P = average weight assigned to precipitation

P = average annual level of precipitation in weather zone

w_{FI} = average weight assigned to freeze index

FI = average annual freeze index in weather zone

w_{FTC} = average weight assigned to the number of freeze thaw cycles

FTC = average annual number of freeze-thaw cycles experienced in weather zone

Because the factors are measured in different units, each factor term was normalized by the maximum value of the factor among all the weather zones (counties) in the state. Maximum values, P_{max} , FI_{max} , and FTC_{max} were obtained from “normal” (30-year average) weather data. Therefore, the index was rewritten as follows:

$$\text{Index} = w_P * (P/P_{max}) + w_{FI} * (FI/FI_{max}) + w_{FTC} * (FTC/FTC_{max}) \dots\dots\dots (99)$$

From the weather data-file of INDIPAVE 2000, the values of P_{max} , FI_{max} , and FTC_{max} are 47.95 mm, 889 degree-days, and 71 respectively. From the questionnaire survey, the average weights assigned to precipitation, freeze index, and freeze-thaw cycles were 0.3, 0.35, and 0.35 respectively.

Therefore the index for weather severity, or Weather Severity Level (WSL), for any pavement section located in a weather zone (county) k , is given as follows:

$$\begin{aligned} WSL_k &= 0.30 * (P_k/47.95) + 0.35 * (FI_k/889) + 0.35 * (FTC_k/71), \text{ or} \\ WSL_k &= (6.257 * P_k + 0.394 * FI_k + 4.930 * FTC_k) * 10^{-3} \dots\dots\dots (100) \end{aligned}$$

Where P_k = annual precipitation in weather zone k , in mm

FI_k = annual level of freeze index in weather zone k , in degree-days

FTC_k = annual number of freeze thaw cycles in weather zone k

Using the above formula, the weather severity level of each county was calculated (presented in Appendix J). Also, a statewide weather severity contour map based on the county weather severity levels was plotted (presented as Appendix I).

6.3 Database Development

In order to manage the large amounts of data for this study and also to serve as a data warehouse for future JTRP studies, a database was designed and implemented for all collected and collated data. This database, christened INDIPAVE 2000 [Labi and Sinha, 2000], consists of data on various pavement attributes, including condition, maintenance, subgrade, climate, and traffic. This database was designed as a relational type that facilitates data management tasks such as querying, sorting and reporting in desired formats. Data may be queried either for each of the 9902 1-mile segments or for each 930 contract sections. Contract sections, typically from major intersection to major intersection, have lengths ranging from 0.25 to 15 miles.

INDIPAVE 2000 consists of 11 data-files (i.e., tables) linked to each other through any of the two central data-file (road segment identification, which), each representing a data category (Figure 6-7). Within each table, the first column represents the primary key for the data-file. For each row, the first column in the primary key to that row, i.e., a unique identifier using which data contained in other columns of that row can be assessed. Data-files are linked to each other by foreign keys, so that it is possible to assess various data types columns in various data-files that correspond to a given primary key. For example, it is possible to obtain the 1995 ESALs, 1995 freeze index, and 1996 maintenance costs for a given road segment and store this information separately in new data-file.

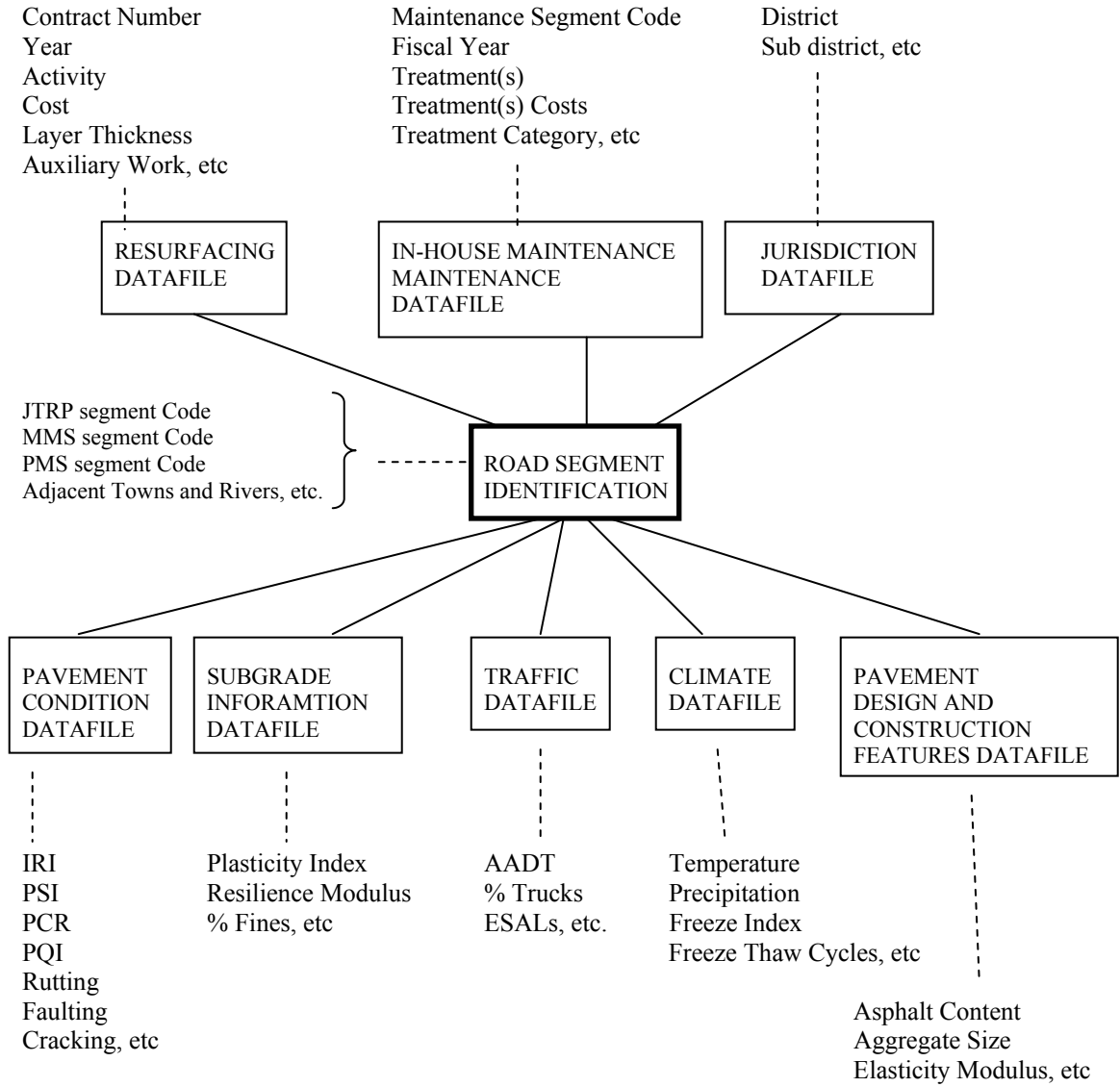


Figure 6-7: Entity Relationship Schema for INDIPAVE 2000 Database

6.4 Chapter Summary

Data management for the present study consisted of identification of data needs, collection and processing of requisite data from various sources, and the design and implementation of a relational database that was christened INDIPAVE 2000. Data included in this database includes road segment identification jurisdiction information, pavement condition, traffic data, design and construction features,

geotechnical data, weather data (by county), maintenance data, and rehabilitation/(re)construction data. The reason for the large effort expended in data collection and collation is that each of all four aspects of the study (evaluation of maintenance effectiveness in the short- and long-terms, trade-off analyses, and case studies) typically require large amounts of data.

Problems encountered in data collection include differences in road referencing and reporting period schemes used by the various sources of data. These problems were overcome by selecting the scheme associated with one data source, and meticulously relating individual data from the other sources to that of the selected source. Data preparation involved the transformation of raw data to “secondary “ data of use to the study, such as estimation of traffic loading in ESALs from raw count data, and computation of freeze indices and freeze-thaw cycles from hourly temperature data. Applying Delphi-like techniques to the results of the district/sub-district questionnaire survey, all weather factors (precipitation, freeze index and freeze thaw), were combined into a single measure of weather severity for use in pavement performance modeling and trade-off analyses.

CHAPTER 7: RESULTS OF SHORT-TERM EFFECTIVENESS EVALUATION

7.1 Short-term Maintenance Effectiveness Models

The ability of various maintenance treatments in reducing pavement deterioration in the short run was modeled using a variety of functional forms. Effectiveness was generally expressed in terms of performance jump (which is the instantaneous increase in pavement condition) or a reduction in the rate of pavement deterioration. Because the before-maintenance and after-maintenance values of pavement condition within any given year are generally not available, the value of performance jump used is a value obtained through extrapolation (Chapter 5). The INDOT PMS software requires values of performance jump to account for changes in the shape of the pavement performance curve in response to the application of maintenance. Using the relationships derived in Chapter 5, other measures of short-term maintenance effectiveness, deterioration rate reduction (DRR), and deterioration reduction level (DRL) can be estimated from performance jump (PJ). For crack sealing model, effectiveness was measured in terms of DRR. Because INDOT typically carries out each year's roughness measurements in late Fall (October) which is generally after the completion of all maintenance for the corresponding year, the "Maintenance before Monitoring" scenario, as described in Chapter 5 is appropriate for the present study, and the expression derived for maintenance effectiveness for this scenario was used. The data from over 5000 1-mile pavement sections was considered for use in developing models to estimate the short-term effectiveness models for various maintenance treatments. Maintenance effectiveness models were built or investigated for the following treatments:

- Thin overlay
- Micro-surfacing
- Chip sealing
- Crack sealing using traditional sealant

- Crack Sealing using crump rubber
- Join/Bump Grinding

For each variable investigated, a two-sided hypothesis test was used for the coefficient in order to determine whether that variable has a significant influence on the response variable (maintenance effectiveness) at 20% significance. The hypothesis test formulation as follows:

H_0 : The coefficient of the variable X_i is equal to zero (i.e., the variable has no significant influence)

H_1 : The coefficient of the variable X_i is not equal to zero (i.e., the variable has a significant influence)

The critical value of the t-statistic corresponding to 10% significance is 1.64, so the null hypothesis was rejected if the absolute value of the t-statistic exceeded this value.

Maintenance effectiveness was investigated for various treatment types in each category of maintenance: major preventive maintenance (thin overlays and micro-surfacing), moderate preventive maintenance (chip sealing), and minor preventive maintenance (crack sealing). Modeling was carried out using only data for pavements that received a given type of treatment. Therefore sections that received multiple treatments were not included in the analysis.

Because the response variable is the performance jump or deterioration rate reduction, a continuous variable was used, as these measures take on continuous values. The response variable was calculated using methods discussed in Chapter 5. “Initial pavement condition” refers to the condition of the pavement before a treatment is administered.

After several trials involving a variety of mathematical forms, models that best explain the effectiveness of the maintenance treatments, for each treatment type, are presented below.

7.1.1 Thin Overlay Effectiveness Model

Thin overlays are used as a preventive maintenance treatment applied to pavements in fair condition. These involve the laying and compaction of a hot mix asphalt (HMA) layer of less than 1.5” thickness over the entire roadway surface with a view to arresting the initiation or development of imminent surface defects. In a typical year, this treatment accounts for 40–50% of the entire budget of all activities that fall under the “maintenance” category, and is carried out solely by contract.

Using annual road condition and maintenance records for pavements that received thin overlays in 1995, models were estimated to predict the jump in pavement performance due to this treatment. The model form that best suited the data was of the following form:

$$PJ = A / (B + C * D^{PSI}) \dots\dots\dots (101)$$

PJ = Performance jump experienced by a pavement section due to thin HMAC overlay, in PSI units.

PSI = Condition of pavement at time of maintenance, in PSI units

A, B, C and *D* are coefficient estimates.

The logistic functional form indicated above was selected for modeling of thin overlay effectiveness because it not only provided an opportunity to explain the resulting model from an engineering perspective, but it also provided a fit to the observed data better than other functional forms considered. Descriptive statistics and model results for the data used for investigating thin overlay effectiveness are given in Tables 7-1 and 7-2, respectively.

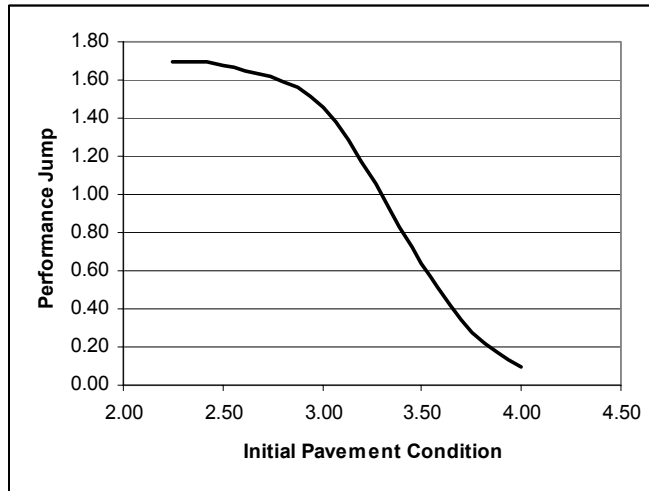
Table 7-1: Descriptive Statistics for Thin Overlay Effectiveness Model

	Variables	
	Initial Pavement Condition (PSI)	Performance Jump (PSI)
Minimum	1.98	0.35
Maximum	3.10	1.96
Mean	3.10	0.87
Standard Deviation	0.52	0.45
Coefficient of Variation	17%	52%

Table 7-2: Model for Thin Overlay Effectiveness

Coefficient	Estimate	t-statistic
A	71.63509	14.3258
B	42.0087	2.2541
C	7.74E-06	1.3548
D	97.1797	6.9445
R ²		0.41

Preliminary descriptive analysis of the data suggests that the level of maintenance expenditure per lane-mile is not an influential factor in the magnitude of performance jump offered by this treatment. As such, the only explanatory variable used is pavement condition at time of treatment. This means that within the range of thicknesses typical of thin overlays, increased expenditure obviously does not result in increased performance jump at the specified level of significance, all else being constant. Rather, an increase in performance jump is attributable to the condition of the pavement before the overlay treatment; the lower the condition of the pavement, the higher the jump in performance. Because this treatment is always administered on a contract basis, available cost data represents total costs of such treatment as well as other associated externalities such as mobilization, contractors profit, utility relocation, and other costs that are not directly related to pavement repair but could not be separated from pavement costs for modeling purposes due to lack of further information. In this respect, differences in treatment costs, even across pavements that received the same overlay thickness are attributable to non-pavement factors such as contractor's profit, length of section treated (economies of scale), as well as pavement factors such as surface preparation and condition of the pavement at time of treatment. The shape of the curve for thin overlay effectiveness is S-shaped, as shown in Figure 7-1, signifying that the relative change in performance jump per unit change in pavement condition changes with level of initial pavement condition.



$$PJ = \frac{71.63}{42.01 + (10^{-5.11} * 97.17^{PSI})}$$

Figure 7-1: Fitted Model for Thin Overlay Treatment Effectiveness

The curve starts with a “slow” phase, indicating that the difference in performance jump is relatively little when pavements is in poor condition. This suggests that a pavement in very poor condition benefits relatively little from overlay treatment. The fitted curve also indicates that for pavements in fair condition, the difference in the performance jump is substantial for a small difference in pavement condition. At the third phase of the effectiveness curve, a small difference in pavement condition yields little incremental benefit, as there is relatively “little room for improvement” for pavements in that condition.

The findings for thin HMAC overlay effectiveness appear to be consistent with those of previous research. A study in Mississippi found that surface treated pavements experience 20–40% jump in condition (in terms of PCR) after treatment, and that lower condition of the pavement at time of treatment was associated with higher jumps in pavement condition [Rajagopal and George, 1991]. An earlier study in Indiana found overlay performance jump to be related solely to thickness of the overlay [Colucci-Rios and Sinha, 1985]. Because the present study only considers thin overlays, which have a very little range of thickness application, overlay thickness was not considered as a factor. Indeed, for a vast majority of pavements that received this treatment in 1995 fiscal year, the overlay thickness was 1.5 inches. Pavement

condition at time of treatment was probably not considered in the earlier study because the focus of the study was on rehabilitation (thick overlays), which is typically applied to pavements in poor condition (rather than to pavements with a wide range of conditions) and generally yield fairly uniform and large jumps in performance.

The questionnaire survey, the results of which are provided in Chapter 4, showed that Indiana's sub-districts and districts administer thin HMAC overlay treatments as a preventive maintenance treatment. The survey results indicated thin overlays are associated with significant benefits in terms of increase in pavement condition and extended service life. Pavements in the state that have benefited significantly from thin overlays include I-465 in Marion county in 1993. Possible future enhancements to the thin HMAC effectiveness model include the use of more years of data, a broader set of explanatory variables such as milling status (or depth), and possibly, thickness of the thin overlay.

7.1.2 Micro-surfacing Effectiveness

Micro-surfacing involves laying of a bituminous mixture over the entire surface of a pavement. No rolling of the laid material is required, as the mixture includes a hardening additive. The thickness of the laid material is typically up to 50 mm. This treatment is relatively new in Indiana, and there are very few pavement sections that have received this treatment either in-house or on contract basis. Therefore no models could be developed for this treatment, and an assessment of the cost-effectiveness of this treatment can only be made from the descriptive statistics of the data as shown in Table 7-3.

Table 7-3: Descriptive Statistics for Micro-surfacing Effectiveness Modeling

	Micro-surfacing Cost (\$/lane-mile)	Pavement Condition before Treatment (PSI units)	Performance Jump (PSI units)
Maximum	26,393.31	3.42	1.05
Minimum	18,427.68	2.58	0.40
Mean	21,629.10	2.92	0.76
Standard Deviation	3,654.49	0.31	0.25
Coefficient of Variation	16.90%	10.44%	33.14%

Obviously there is some short-term benefit associated with micro-surfacing treatment. However lack of a model precludes estimation of the level of effectiveness for pavements in a certain condition and at a certain level of investment for this treatment. Therefore, an average value of 0.76 PSI (see Table 7-3) units may be used as the general benefit or effectiveness (performance jump) whenever micro-surfacing treatment is applied. Figures 7-2 and 7-3 present the observed values of effectiveness of micro-surfacing treatments plotted against initial pavement condition, and treatment cost, respectively. The data for these points came from treatments carried out at various sections of State Road 46, Interstate 465, and US 231 between the period 1995-1996.

The benefits of micro-surfacing treatment have been documented in literature. Some researchers have mentioned that this treatment addresses cracks and rutting, both of which have a direct bearing on PSI, thus implying that there is a performance jump associated with micro-surfacing [Dwight Hixon and Ooten, 1993]. Others have indicated the benefits of this maintenance treatment in the long-term [Raza, 1994].

The questionnaire survey conducted as part of this research (Chapter 4) showed that micro-surfacing is not a common treatment, probably because of its novelty. However, sub-districts that have applied this treatment perceived a 3-year extension in pavement life, which is confirmatory of the significant performance jump associated with this treatment.

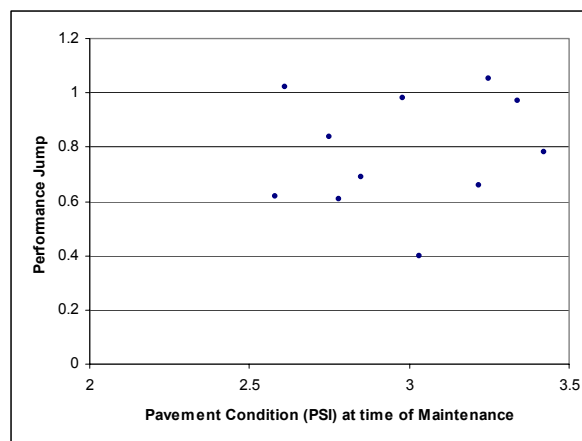


Figure 7-2: Effectiveness of Micro-surfacing Treatments by Initial Pavement Condition

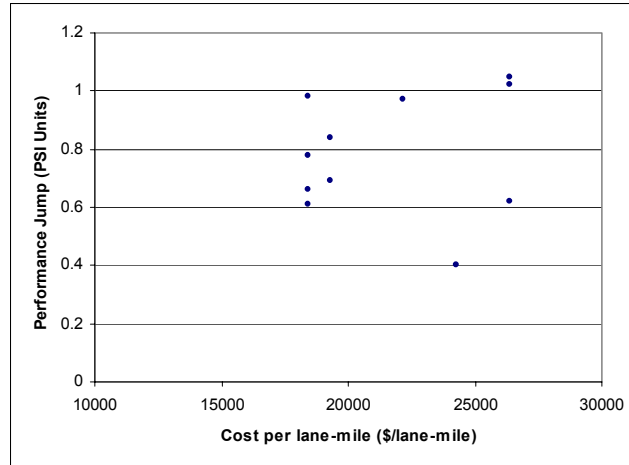


Figure 7-3: Effectiveness of Micro-surfacing Treatments by Treatment Cost

7.1.3 Seal Costing Effectiveness

A thin coat of binder and aggregates is typically spread over low-volume non-Interstate flexible pavements with a view to keeping such pavements in motorable condition and consequently deferring the need for major preventive maintenance (thin overlay) or rehabilitation. Such treatments are known to heal surface cracks and raveled surfaces and are therefore expected to have a direct impact on PSI. In recent times in Indiana, there have been a few instances where seal coating has been carried out on contract, however the bulk of such treatments are done in-house by INDOT sub-districts. Seal coating typically accounts for about 10% of the annual force-account pavement maintenance budget, and is used far more widely than sand sealing.

Using annual condition and costs for pavement sections that were seal coated in 1995 fiscal year, models were developed to estimate the effectiveness of such treatment in terms of the immediate jump in PSI due to the treatment. Table 7-4 presents the descriptive statistics of relevant variables considered in modeling the effectiveness of chip sealing.

7.1.3.1 Computation of Seal Coating Effectiveness Values

Using annual condition data for pavement sections that received seal coating in the 1995 and 1996 fiscal years, the effectiveness of this maintenance treatment, in terms of performance jump and

deterioration rate reduction, was determined for each pavement section. Only pavements that received seal coating and little or no other treatment were selected for the study. Pavement performance data was available as roughness values (IRI), which were then converted into PSI values using established IRI-PSI relationships (Al-Omari and Darter 1994, Guten et al. 1994).

Table 7-4: Locations and Data for Pavement Sections that Received Seal Coating Treatment

SEGMENT ID	ROAD	START	END	FUNCCLSS	REGION	PAVE		IPC	PJ	DRR	AGG. TYPE
						TYPE					
2007	SR13	35.7	36	Major Collector	Central	OVR		3.29	0.21	3.30	CHIP
2008	SR13	36	37	Major Collector	Central	OVR		2.56	0.63	2.20	CHIP
2009	SR13	37	38	Major Collector	Central	OVR		3.32	0.17	3.29	CHIP
2010	SR13	38	39	Major Collector	Central	OVR		2.86	0.18	2.78	CHIP
2011	SR13	39	40	Major Collector	Central	OVR		2.90	0.20	2.92	CHIP
2015	SR13	43	44	Major Collector	Central	OVR		3.50	0.32	3.39	CHIP
2016	SR13	44	45.01	Major Collector	Central	OVR		3.50	0.21	3.40	CHIP
2017	SR13	45.01	46	Major Collector	Central	OVR		3.56	0.13	3.54	CHIP
2018	SR13	46	47	Major Collector	Central	OVR		3.46	0.36	3.31	CHIP
2019	SR13	47	48	Major Collector	Central	ACP		3.74	0.25	3.68	CHIP
2020	SR13	48	49	Major Collector	Central	ACP		3.83	0.26	3.74	CHIP
2023	SR13	51	52	Major Collector	Central	ACP		3.47	0.09	3.54	CHIP
2024	SR13	52	53	Major Collector	Northern	ACP		3.51	0.24	3.51	CHIP
2025	SR13	53	54	Major Collector	Northern	ACP		3.69	0.15	3.72	CHIP
2026	SR13	54	55	Major Collector	Northern	ACP		3.53	0.09	3.56	CHIP
2027	SR13	55	55.99	Major Collector	Northern	ACP		3.56	0.10	3.54	CHIP
2028	SR13	55.99	57.05	Major Collector	Northern	ACP		3.49	0.03	3.62	CHIP
2029	SR13	57.05	58.04	Major Collector	Northern	ACP		3.46	0.06	3.44	CHIP
2055	SR13	83	84	Major Collector	Northern	OVR		3.77	0.21	3.72	SAND
2056	SR13	84	85	Major Collector	Northern	OVR		3.65	0.01	3.66	SAND
2057	SR13	85	86	Major Collector	Northern	OVR		3.72	0.22	3.66	SAND
2058	SR13	86	87	Major Collector	Northern	OVR		3.54	0.14	3.51	SAND
2060	SR13	88	89	Major Collector	Northern	OVR		3.56	0.10	3.48	SAND
2061	SR13	89	90	Major Collector	Northern	OVR		3.69	0.13	3.66	SAND
2062	SR13	90	91	Major Collector	Northern	OVR		3.72	0.17	3.66	SAND
2063	SR13	91	92	Major Collector	Northern	OVR		3.66	0.03	3.68	SAND
2064	SR13	92	93	Major Collector	Northern	OVR		3.74	0.18	3.68	SAND
3977	SR63	71	72	Principal Arterial	Central	ACP		3.28	0.05	3.32	CHIP
3979	SR63	73	74	Principal Arterial	Central	ACP		2.98	0.45	2.65	CHIP
3980	SR63	74	75	Principal Arterial	Central	ACP		2.90	0.29	2.60	CHIP
3981	SR63	75	76	Principal Arterial	Central	ACP		3.15	0.13	3.08	CHIP
4543	US231	219	220	Major Collector	Central	OVR		3.80	0.22	4.02	CHIP
4544	US231	220	221	Major Collector	Central	OVR		3.57	0.46	3.72	CHIP
4548	US231	224	225	Major Collector	Central	OVR		3.09	0.34	3.32	CHIP
4550	US231	226	227	Major Collector	Central	OVR		2.63	0.77	2.82	CHIP

7.1.3.2 Statistical Test of Significance of Seal Coating Effectiveness Values

The statistical significance of the estimated Performance Jump (PJ) and deterioration rate reduction (DRR) values for seal coating were tested at a 95% level of confidence. This was done to investigate whether the effectiveness of seal coating treatments received by the pavement sections are significantly greater than zero. As the PJ and DRR values are derived from PSI values (which are, in turn, average values of pavement condition over a stretch of highway pavement), the distribution of the PJ and DRR values can be considered as sampling distributions of means. Therefore, the formulated hypothesis for Performance Jump was therefore as follows:

$H_0: \mu_{PJ} \leq 0$ (the seal coating treatments were not effective)

$H_1: \mu_{PJ} > 0$ (the seal coating treatments were effective)

This is a 1-sided hypothesis test with the “rejection region” in the upper tail. Therefore, the critical value of the test statistic is $Z_\alpha = Z_{0.05} = 1.645$. The calculated value of the test statistic is given by:

$$Z^* = (\mu_{PJ} - 0) / (\sigma / \sqrt{n})$$

Where σ is the standard deviation, and n is the sample size.

For the Performance Jump measure, computation of the test statistic gave 5.88, which exceeds the critical value of 1.645, and therefore falls in the rejection region. By rejecting the null hypothesis, it is averred that the seal coating treatments received by the pavement sections yielded performance jumps that were significantly greater than zero, and were therefore effective (from the perspective of performance jump) at a 95% level of confidence. A similar hypothesis test was carried out for the Deterioration Rate Reduction measure. The computation of the test statistic gave 36.48, which far exceeds the critical value of 1.645, and therefore falls in the rejection region, implying that seal coating treatments yielded significant reductions in the rates of pavement deterioration.

From the tests of significance for the pavement sections under study, it is seen that:

- the seal coating treatment is effective, regardless of whether DRR or PJ is used to assess effectiveness,

- all else being the same, seal coating effectiveness appears to be more perceptible when viewed within the context of the Deterioration Reduction Rate, compared to the Performance Jump.

In the next section, seal coating effectiveness models are developed as a function of treatment and pavement attributes, using various linear and non-linear functional forms.

7.1.3.3 Seal Coating Effectiveness Models

Preliminary scatter plots of seal coating effectiveness were drawn in a bid to unveil any glaring trends in such effectiveness over the given ranges of explanatory variables. Besides initial pavement condition, other explanatory variables exhibited relatively little variation with respect to changes in response variable (seal coating effectiveness). Linear, intrinsically linear, and non-linear functional forms were investigated for developing the seal coating effectiveness model. A discussion of the modeling efforts for each functional form and measure of effectiveness is provided below.

Linear Functional Forms for Performance Jump upon Seal Coating

After several trials with a variety of linear and intrinsically linear mathematical forms, it was found that seal coating effectiveness can be explained using a relationship of the following general form (Equation 1):

$$PJ = A_0 + \sum_{j=1}^M (A_j X_j) \dots \dots \dots (101)$$

- where PJ = Performance Jump (in PSI units)
- A_0 = constant term
- A_j = coefficient of term X_j
- X_j = explanatory variable j
- M = number of significant variables

The model results are presented in Table 7-5.

The model results showed that the only significant variable influencing Performance Jump due to seal coating, is the initial pavement condition. The result suggests that the higher the initial pavement condition, the lower the Performance Jump. This finding seems to be intuitive when it is considered against the realization that there is a ceiling to which accrued levels of pavement condition may attain (the maximum level is 5 PSI units).

Table 7-5: Summary of Developed Models

MOE	Model Types	Model Structure	Symbol	Estimate	t-statistic	Adjusted R ²
Performance Jump (PJ)	Linear	$PJ = A + B*IPC$	A	1.158	4.851	0.310
			B	-0.275	-3.962	
	Intrinsically Linear	$PJ = A*\exp(IPC-B)^C$	A	1.360	2.332	0.792
			B	1.800	5.082	
			C	1.408	5.950	
Deterioration Rate Reduction (DRR)	Linear	$DRR = A*ROUTE_TYPE + B*IPC$	A	-0.159	-2.462	0.901
			B	0.998	15.630	
	Intrinsically Linear	$DRR = \exp(A*IPC)$	A	0.335	16.146	0.876
	Non Linear	$DRR = \exp[1/\{C + A*(B^{IPC})\}]$	A	-7.024	-0.06	0.800
			B	1.037	1.96	
			C	8.794	0.08	

The variable ROUTE_TYPE takes a value of 1 if Principal Arterial, but is 0 if Otherwise (Major Collector).
 IPC- Initial Pavement Condition.
 MOE- Measure of Effectiveness (of seal coating treatment).

The farther the condition of a pavement from this maximum, the greater the potential jump to reach that level, and the closer the condition of a pavement to this maximum, the smaller the potential jump needed to reach that level. It must be added that this finding is applicable to the range of condition values of the pavements under study. Variables that were considered but turned out to be insignificant at 95% level of confidence include aggregate type (coarse vs. fine), work source (sub-district that carried out the work), year of treatment, functional class of road, and type of pavement (AC-over-PCC overlay versus full-depth AC). Intrinsically linear models for Performance Jump were investigated using various Bob-Cox

transformations of the response variable, but such efforts failed to yield encouraging results and were therefore abandoned.

Linear Functional Forms for Deterioration Rate Reduction upon Seal Coating

After several trials with a variety of linear and intrinsically linear mathematical forms, it was found that seal coating effectiveness, in terms of deterioration rate reduction, can be explained using relationships of the general forms shown as Equations 2 and 3. The general form for Equation 2 is as follows:

$$DRR = A_0 + \sum_{j=1}^M (A_j X_j) \dots \dots \dots (102)$$

Where DRR = deterioration rate reduction (in PSI units per year)

A_0 = constant term

A_j = coefficient of term X_j

X_j = explanatory variable j

M = number of significant variables

The results for the model based on Equation 102 are presented in Table 7-5.

The estimated linear DRR model (Table 7-5) suggests that the reduction in the rate of pavement deterioration due to seal coating treatment is a function of route type and initial condition of the pavement. For flexible pavements in the “Major Collector” functional class, a greater reduction in their deterioration rates in response to seal coating were observed, compared to flexible pavements on the principal arterial system. This implies that seal coating is more effective on major collector pavements than on principal arterials. Principal arterial roads are generally associated with greater levels of traffic than major collectors. Recognizing that higher traffic volume has an adverse effect on the stability of laid aggregates (Mouaket et al. 1992; Shuler 1998) and consequently on the effectiveness of the treatment, it seems quite intuitive that seal coating on high volume roads such as principal arterials yield lower deterioration reduction rates than on relatively lower volume roads (major collectors).

Table 7-6: Results of the Breusch-Pagan and White Tests for Heteroscedasticity

Model MOE and Type → Results ↓		Performance Jump (PJ) Residuals Models		Deterioration Rate Reduction (DRR) Residuals Models		
		Linear PJ Model	Non-Linear PJ Model	Linear DRR Model	Log-Linear DRR Model	Non-linear DRR Model
Estimated Coefficients of the Heteroscedasticity Equation	<i>a</i>	-1.9312	-0.9417	-0.7712	-0.7712	-2.892
	<i>b</i>	7.7176	4.3341	3.7504	3.7504	11.004
R Square		0.1762	0.0185	0.0156	0.0156	0.0156
Regression Sum of Squares (RSS)		14.259	3.414	2.289	2.289	73.204
Calculated Value of Test Statistic (Breusch-Pagan)		7.125	1.707	1.145	1.145	36.602
Calculated Value of Test Statistic (White)		3.524	0.369	0.312	0.312	5.680
Critical Value of Test Statistic (at 95% confidence)		3.84	3.84	3.84	3.84	3.84
Conclusion (Breusch-Pagan Test)		HT	HM	HM	HM	HT
Conclusion (White Test)		HM	HM	HM	HM	HT
Final Conclusion on Heteroscedasticity		Inconclusive	HM	HM	HM	HT

*HT- Heteroscedastic**HM- Homoscedastic*

Indeed, in many states, seal coating of Interstates and other high volume pavements is precluded as a matter of policy. The model also suggests that higher initial condition of flexible pavements is generally associated with higher effectiveness (reduction in the rate of deterioration) due to seal coating, all else being equal. This corroborates findings from previous studies that preventive maintenance applied before the onset of advanced deterioration is more effective than when it is applied at later stages rate (Hanna 1993; O'Brien 1996; Syed et al. 1998), "effective" in this context meaning a reduction in deterioration rate.

Intrinsically linear models for deterioration rate reduction were investigated using various Bob-Cox transformations of the response variable ($Y \rightarrow Y^\lambda$). The most encouraging model from such transformations was that where $\lambda = 0$, i.e., $Y \rightarrow \log_e Y$. The general form for the selected intrinsically linear equation is as follows:

$$DRR = e^{A_0 + \sum_{j=1}^M (A_j X_j)} \dots\dots\dots(103)$$

Where symbols have their usual meanings.

The results for the model based on Equation 103 are presented below in Table 7-5.

When log *DRR* is used as a response variable instead of *DRR*, the variable representing functional class was found insignificant at 95% confidence. Also, the constant term was insignificant, and the only remaining explanatory variable was the initial pavement condition. As it was in the linear *DRR* model, the sign of the IPC variable in the Log *DRR* model infers that higher reductions in the rate of deterioration are associated with higher levels of initial pavement condition. Other explanatory variables that were investigated, but were found to be statistically insignificant include aggregate type: coarse aggregate (chip sealing) versus fine aggregates (sand sealing), sub-district that carried out the work, year of treatment, functional class of road, cost of the treatment, and auxiliary work, if any, that was carried out on the pavement surface prior to the treatment.

Figure 7-4 illustrates the trend of seal coating effectiveness (expressed as performance jump and reduction in the rate of pavement deterioration) relative to initial pavement condition, for the linear, and intrinsically linear model forms.

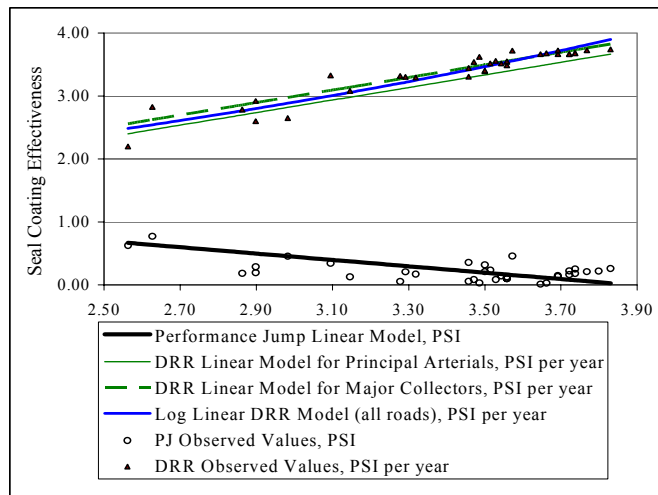


Figure 7-4: Linear and Log-linear Seal Coating Effectiveness (DRR and PJ) Models

As regards the effect of initial pavement condition on seal coating effectiveness, it is interesting to observe that diametrically contrasting directions of impact of this factor were observed for the two measures of effectiveness (Figure 7-4): seal coating a pavement with higher initial condition is associated with a lower performance jump, but is associated with a greater reduction in the deterioration rate, compared to a pavement in relatively lower initial condition. In other words, the effectiveness of seal coating is greater for pavements in relatively poor condition, from the perspective of performance jump, but is smaller for such pavements from the perspective of deterioration rate reduction. This is obviously because a pavement in relatively poor condition has a greater potential (ceiling) to reach a certain maximum condition (5.0 PSI) than a pavement in relatively good condition. However, even though such a pavement (in poor condition) may accrue a higher jump in performance, it obviously cannot and does not sustain this benefit with the same tenacity as a pavement in good condition, all else being equal, and therefore has a lower reduction in its deterioration rate.

Intrinsically Non-linear Model for Deterioration Rate Reduction upon Seal Coating

After investigating several intrinsically non-linear functional forms, the following model (Equation 104) was selected as most representative of DRR due to seal coating:

$$DRR = EXP[1/(C + A * B^{IPC})] \dots\dots\dots (104)$$

Where

DRR = Reduction in the rate of deterioration of a flexible pavement section due to seal coating treatment, in PSI units per year,

IPC = Initial pavement condition, i.e., condition of pavement at time of maintenance, in PSI units,

A, *B*, and *C* are constants.

The above model form provided the closest fit to the available data. The model results are presented in Table 7-5, and illustrated in Figure 7-5.

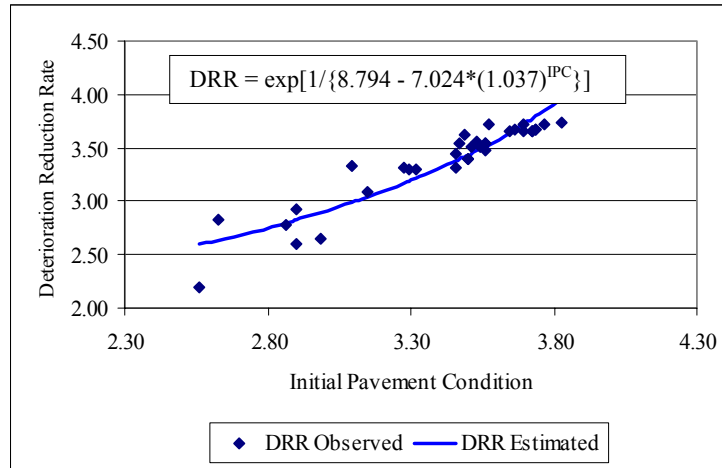


Figure 7-5: Non-linear Seal Coating Effectiveness (*DRR*) Model

The model results show that increasing initial pavement condition is associated with increasing *DRR*. This result is consistent with the linear *DRR* model developed earlier in the present study, and supports the rationale behind the application of preventive maintenance treatments before the onset of significant deterioration.

Intrinsically Non-linear Model for Performance Jump upon Seal Coating

After trying several intrinsically non-linear functional forms, the following model (Equation 105) was adjudged the most representative of seal coating effectiveness (performance jump) trends with respect to initial pavement condition:

$$PJ = A * \text{EXP}[-(IPC - B)^C] \dots\dots\dots(105)$$

Where *PJ* = Performance jump experienced by pavement section upon seal coating, in PSI units,
IPC = Initial condition of pavement (i.e., at time of maintenance) in PSI units,
A, *B*, and *C* are constants.

The above model form was chosen because compared to other intrinsically non-linear model forms considered it provided the closest fit to the available data, and also because it facilitated engineering interpretation of the model form. The model results are presented in Table 7-5.

No identifiable pattern was revealed when seal coating effectiveness was considered against the cost of chip sealing per lane-mile, or the cost per lane-mile per unit level of initial pavement condition. In other words, the data suggested that increases in chip sealing costs per lane-mile from one pavement section to another, does not significantly increase Performance Jump. Increases in seal coating cost per lane-mile are typically attributable to changes in costs of material and labor, which may vary by region and source of work (chip sealing is more expensive if carried out by-contract). Also, it may be argued that higher costs of chip sealing may be due to the extent of preparatory works prior to this treatment; therefore treatment costs are expected to significantly influence increase in pavement condition, all else being equal. However, it is worth mentioning that cost records for seal coating, if it is carried out in-house (which is often the case), exclude cost of surface preparatory works, as the latter are reported separately. Where carried out by contract, seal coating costs include surface preparatory works. However, the number of pavement sections that received seal coating by contract are very few. For these two reasons, pavements that received seal coating treatments by contract were excluded from the modeling process. Figure 7-6 shows the observed and fitted values of the seal coating effectiveness model. The curve represented by the range of initial condition of pavements that have received this treatment is shown as a bold continuous line, while that represented by the range of initial pavement conditions not covered in the observations (but are useful for model interpretation) are shown as a dashed line.

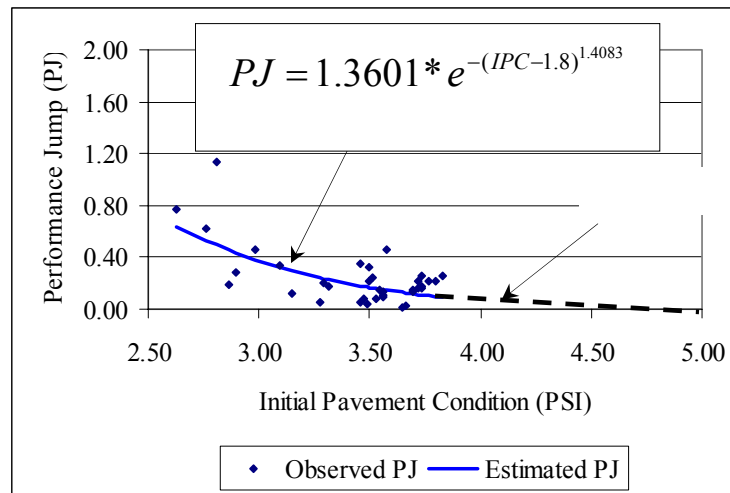


Figure 7-6: Non-Linear Seal Coating Effectiveness (PJ) Model

The curve in Figure 7-6 provides inferences that are generally similar to that obtained for the linear PJ model: pavements in relatively good initial condition are associated with lower Performance Jumps upon seal coating, while those in relatively poor initial condition have higher jumps in performance. While Performance Jumps of up to 1.13 PSI were observed from the present dataset, the developed model suggests that for the range of initial pavement conditions given, the expected maximum theoretical performance jump upon seal coating is 0.63. As all pavement sections studied had initial PSI values above 2.56, the model does not provide an indication of seal coating effectiveness for pavements whose initial condition levels are lower than this value. Also, the model suggests that as the initial pavement condition approaches 5 PSI (maximum level of pavement condition), the performance jump approaches zero. This implies that for pavements in very good-to-excellent condition, the benefits of seal coating, in terms of performance jump, is likely to be negligible, and any such exercise may not likely be cost-effective. This lends credence to the belief that preventive maintenance that is applied too early in the life of a pavement is wasteful and not cost-effective (Geoffroy 1996).

Discussion for Linear and Non-linear PJ and DRR Models

The findings of this study are similar to past research efforts on seal coating effectiveness. Using pavement condition rating (PCR) as the unit of performance jump measurement, a study in Mississippi found that pavements treated as such experience a 19-44% jump in performance after treatment (Rajagopal and George 1991). That study also found that the lower the condition of the pavement before treatment, the higher the performance jump, which appear consistent with the findings of the present study. A major finding of the Supplemental Maintenance Effectiveness Research Program (SMERP) carried out in Texas in 1997, was that the condition of a pavement was a major determinant of the effectiveness of chip sealing treatment that the pavement receives when such effectiveness is considered over a period of time (Syed et al. 1998). This seems consistent with the DRR model results in the present study.

Other seal coating effectiveness studies focused on long-term effectiveness evaluation, and provided evidence of the benefits of seal coating over the entire pavement life cycle (Young et al. 1986; Mouaket and Sinha 1990), which can be considered a direct consequence of short-term effectiveness of this

treatment. Also, responses from a questionnaire survey of INDOT sub-districts (Labi 2001) were found to be consistent with the findings of the present study and similar past studies. The survey showed that seal coating treatments are associated with appreciable increases in pavement condition, extension in service life for both full-depth AC and AC-over-PCC overlay pavements, and a decrease in the level of pavement corrective maintenance subsequent to application of this treatment. Table 7-5 provides a summary of the linear and non-linear performance jump and deterioration rate reduction models that were developed in the present study.

Upon close visual examination of the DRR and PJ models, it appears that as IPC increases, the deviation (error term) between the observed and estimated values of effectiveness decreases. This could be indicative of a serious statistical problem known as heteroscedasticity. If uncorrected, this could compromise the predictive efficacy of the models. It was therefore found necessary to carry out validation of the developed models to ascertain the predictive capability of the models, and also to carry out requisite econometric tests to identify any presence of heteroscedasticity.

Validation of Seal Coating Effectiveness Models

Most of the 35 pavement sections studied received seal coating treatment at lanes in both directions. The estimated seal coating effectiveness models utilized data from the eastbound and northbound lanes, while data from a different set of pavement sections (the corresponding westbound and southbound lanes that received such treatment) were used for validation. Validation was essentially carried out by estimating seal coating effectiveness from the developed models and comparing the estimated values to the observed effectiveness values at those sections. The root mean square errors (RMSE) of each data point were then computed to provide an insight into how well the models estimate the observed seal coating effectiveness. The validation formula used (Equation 106) is as follows:

$$RMSE = \frac{\sum_{i=1}^{n^*} (Y_i - y_i)^2}{n^*} \dots\dots\dots(106)$$

Where: Y_i = observed value of response variable in the i th validation case

y_i = predicted value for the i th validation case based on the model building data set

n^* is the number of cases in the validation dataset

The calculated RMSE for the developed models ranged from 0.02 to 0.08, indicating that the performance of the developed models in predicting seal coating effectiveness as a function of the selected explanatory variables, are satisfactory.

Tests for Heteroscedasticity

One of the basic assumptions associated with statistical models is that the error term is homoscedastic, that is, it has constant variance. In other words, the distribution of the error term with respect to any explanatory variable should not follow a definite increasing or decreasing pattern. If this assumption is violated, the resulting model is said to suffer from heteroscedasticity. In fact, in modeling maintenance effectiveness using a cross section of pavement sections that received maintenance, significant levels of heteroscedasticity may be encountered because the error terms associated with pavements in relatively good initial condition may be different from the error terms of those in relatively poor condition. In other words, it seems plausible to expect that the estimated maintenance effectiveness when the pavement is in relatively good condition would be close to the true observed value, compared to the case for pavements in relatively poor condition. Whenever heteroscedasticity is present in a model, ordinary least squares estimation places more weight on the observations with large error variances than those with small error variances (Pindyck and Rubinfeld 1991). Such imbalanced weighting occurs because the sum-of-squared residuals associated with the large variance error terms (likely from the pavements with low initial condition), are significantly greater than the sum-of-squared residuals associated with the small variance error terms (likely from the pavements with high initial condition). A consequence of this implicit weighting is that parameters estimated using ordinary least squares are inefficient, even though they may be unbiased and consistent. In other words the estimated variances that are obtained are not the minimum variances that are desired. Another consequence of heteroscedasticity is that the estimated variances of the estimated parameters will be biased estimators of the true variance of the estimated parameters. Therefore,

if a model containing such biased estimates of the parameters variances is used to predict maintenance effectiveness, the resulting statistical tests and confidence levels would be incorrect.

Quantitative Tests for Heteroscedasticity

The Breusch-Pagan and White Tests were used to test for heteroscedasticity. The Breusch-Pagan tests involves determining the residuals (error terms) between the observed and estimated values of maintenance effectiveness, and calculating the variance of the residuals. The residuals are then normalized by division by their variance. Then, assuming any present heteroscedasticity is linear, the normalized residuals are regressed on X (initial pavement condition) to obtain a relationship of the form:

$$\frac{\hat{\varepsilon}_i^2}{\hat{\sigma}^2} = a * X_i + b$$

Where ε = error term, or deviation of the estimated values of effectiveness from the observed value, for the i th observation

$$\hat{\sigma}^2 = \frac{\sum \hat{\varepsilon}_i^2}{N}$$

N = number of observations (pavement sections)

a, b are constants

From the above estimation (Equation 107), the regression sum of squares is calculated. If the calculated value of the test statistic ($RSS/2$) exceeds its critical value (value of the chi-square distribution with 1 degree of freedom, given a certain significance level), then the null hypothesis of homoscedasticity is rejected and it is concluded that the model suffers from heteroscedasticity. Application of the Breusch-Pagan procedure to the developed models yielded the results presented in Table 7-6.

The White test involves calculating the product of the number twenty (20) and the R-square of the regression model for the normalized residuals. If this test statistic exceeds its critical value (value of the chi-square distribution with 1 degree of freedom at a given level of significance), then the null hypothesis

of homoscedasticity is rejected and it is concluded that the model suffers from heteroscedasticity. Results of the White tests confirmed that the *DRR* nonlinear model is heteroscedastic, while the linear and log-linear *DRR* models were found to be homoscedastic. On the other hand, the non-linear *PJ* model was found to be homoscedastic. These conclusions were reached at a 95% confidence level. It is therefore recommended that for estimating *DRR*, the linear or log-linear model can be used. For *PJ* estimation, the non-linear model developed in the present study can be used to estimate such effectiveness of seal coating on highway pavements. The case for the linear performance jump model was inconclusive.

In the context of seal coating, the treatment is intended to correct extensive cracking, spalling, shallow surface failures, loss of skid resistance, and raveling, among others. Ideally, the measure of pavement performance that should therefore be an index that directly captures the extent and severity of such defects. PSI may not suffice for this purpose, as it is more directly associated with ride quality. A more appropriate index would be the Pavement Condition Rating (PCR). Given the lack of PCR data at the time of study, PSI was used. Therefore the use of PSI was implicit with the assumption that the surface defects that seal coating addresses are ultimately manifested in ride quality. In future studies, collection and utilization of PCR data may likely yield better models than those obtained using PSI.

The present study found that seal coating affords such pavements a jump in pavement condition of between 0.08 and 0.63 PSI units, with an average of 0.23 PSI units. From the perspective of short-term deterioration trends, it was found that this treatment reduces the rate of pavement deterioration by a level that is between 2.52 and 4.04 PSI units per year, with an average of 3.38 PSI units per year. As these values are very general in nature, better estimates of effectiveness for a specific pavement of known condition at the time of such treatment can be found using the models developed in the present study. In two of the models (one linear, and the other non-linear), Performance Jump was used as a measure of effectiveness, while in the other three models (linear, log-linear, and non-linear) the rate of deterioration reduction was used. As much as possible, the selected model forms were those that best fit the given data while (in some cases) facilitating engineering interpretation of the variability of seal coating effectiveness in relation to various level of initial pavement condition.

Past research on seal coating effectiveness has generally indicated that both short and long-term benefits are associated with this treatment. Regarding the relationship between short-term effectiveness and initial pavement condition, results of past studies have been equivocal: some studies found that lower levels of initial pavement condition are associated with lower effectiveness after seal coating treatment, while results of other studies were to the contrary. The present study, in developing seal coating effectiveness models, explains the findings of both schools of thought. It was found that all else being equal, pavements in relatively poor condition were associated with higher performance jumps but lower reductions in their rates of deterioration. This implies that there are greater benefits (effectiveness) of seal coating on relatively good pavements compared to relatively poor pavement when considered over an extended period of time, but lesser benefits when considered the very instant the treatment is applied. In the study, traffic levels, properties of the subgrade material, and pavement layers were not explicitly considered, but were surrogated by the use of pavement functional class. While this may seem somewhat restrictive, such factors are expected to have no impact on the immediate jump in pavement performance. On the other hand, the effect of functional class on deterioration rate reduction was found to be significant, suggesting that traffic and subgrade potentially affect the level of seal coating effectiveness after a period of time, rather than instantaneously. From validation tests, it was found that the linear or log-linear model developed in the study can be used to estimate the reduction in the rate of pavement deterioration upon seal coating treatment. Also, to estimate the instantaneous jump in pavement condition due to seal coating, the non-linear performance jump model was found most suitable. The study duly accommodated that fact that correctly specified relative timing between the application of seal coating and performance monitoring (conduction of deterioration measurements) for a given year is crucial in the computation of short-term effectiveness of maintenance. With the developed seal coating effectiveness models, operators of maintenance and pavement management systems can update existing pavement performance curves to reflect the application of such maintenance treatments. Also, with the models developed in the present study, life-cycle cost and benefit analyses of various alternative M&R strategies (arrays of treatment types and respective timings) that include seal coating can be carried out for purposes of highway pavement asset management in the long-term. The benefits of seal coating in the short-term, as demonstrated in the present

study, translate to increased pavement longevity. Therefore, the study results are important to agencies that are considering the use of seal coats as an emergency or stop gap maintenance treatment to hold a poor pavement in acceptable condition until funds are available for more extensive work. Finally, future studies on seal coating effectiveness should strive to obtain requisite data that would enable utilization of a measure of pavement performance that more directly captures the benefits of such treatment.

Responses from the questionnaire survey of INDOT sub-districts are quite consistent with the above findings. The survey showed that chip sealing treatments are associated with approximately six years extension in service life for both full-depth AC and AC-over-PCC overlay pavements. Typical examples of pavement sections in Indiana that are associated with significant performance jumps after chip sealing are various sections on SR-3 in Allen County that received this treatment in 1995.

7.1.4 Crack Sealing Effectiveness Model

Crack Sealing involves the placement of sealing material into surface cracks has the purpose of protecting the underlying pavement materials from wetting and subsequent strength loss and pumping. Accounting for 15–25% of the total annual force account for pavement maintenance, crack sealing is a very common preventive maintenance activity whose cost-effectiveness has come into question in recent years [Shober, 1994]. Using annual road condition and maintenance records for pavements that received only this treatment in the 1995 fiscal year, models for the reduction in the rate of pavement deterioration due to crack sealing was estimated. The 75 sections studied were urban and rural Interstate, US Road, and State Road pavements located at various geographical regions of the state, from the relatively cold and dry north, to the relatively warm and wet south. Crack sealing costs were expressed in 1995 dollars. Descriptive statistics of data used for crump rubber sealing treatment effectiveness are shown as Table 7-7 below. It is seen that crack sealing was generally effective in reducing the rate of deterioration (average of 0.7 PSI units per year, but was not always effective in doing so (increase in deterioration rates of as much as 0.17 PSI units per year).

Table 7-7: Descriptive Statistics for Crack Sealing Effectiveness

	Maintenance Expenditure (\$1000s per ln-mi)	Initial Pavement Condition (PSI)	Annual ESALS (10 ⁶)	Annual Number of Wet Days	Deterioration Rate Reduction (PSI/Year)
Mean	318.76	3.393	0.870	116.881	0.177
Minimum	23.50	1.650	0.020	106	-0.180
Maximum	1471.80	4.190	2.840	128	0.700
Standard Deviation	281.431	0.476	0.922	7.462	0.173

After several trials with a variety of mathematical forms for the crack sealing DRR models, it was found that such effectiveness was best explained by the following functional relationship:

$$1/DRR = A_0 + \sum_{i=1}^N (A_i X_i)$$

where

DRR = Deterioration Rate Reduction, in PSI units per year, upon crumb rubber sealing

A_0 = constant term

A_i = coefficient of term X_i

X_i = explanatory variable i

Table 7-8 present the model results.

Table 7-8: Model for Effectiveness of Crack Sealing

Variable	Coefficient Estimate	t-statistic	Remarks
Constant Term	-12.08	-2.18	
Functional Class	-12.99	-8.09	1 - Interstate 0 - Non-Interstate
Traffic Loading	6.61	7.75	In millions of ESALs
INIT	1.47	2.20	Initial Pavement Condition
WETDYS	0.09	2.33	Number of wet days per year
Response Variable		1/DRR	
R^2		0.59	

From the model results, it is seen that non-Interstate pavement sections that receive crack sealing exhibit lower reduction in their deterioration rates compared to Interstate pavement sections. This finding seems to be counter-intuitive: non-Interstate pavement sections, by “virtue” of their lower design and

construction standards, are more vulnerable to the effects of water ingress through surface cracks, and therefore stand to gain more from crack sealing compared to Interstate pavements. In other words, when non-Interstate pavements are denied deserving crack sealing treatment, they are likely to deteriorate faster than Interstate pavements in the same situation. Indeed, the results for crumb rubber sealing attest to this supposition. However, the finding that non-Interstate pavement sections that receive crack sealing exhibit lower reduction in their deterioration rates compared to Interstate pavement sections, may be explained by the fact that non-Interstate pavements are not built to standard that sustain such benefits to a greater degree, compared to Interstate pavements. The model also showed that more heavily loaded pavement sections show lower reduction in deterioration rates upon crack sealing, compared to relatively lightly loaded pavements. This means that crack sealing seems to be more effective on lightly loaded pavements, all other factors being constant. Again, this finding seems counter-intuitive, as one may expect heavily loaded pavements to be more vulnerable to the effects of not sealing cracks, and therefore would show greater reduction in their deterioration rates. It seems therefore, that the vulnerability of pavements, though a salient consideration, is outweighed by the debilitating effects of heavy loading on crack sealed pavements. Furthermore, the model results showed that the higher the overall condition of the pavement before sealing, the lower the reduction in the rate of deterioration, upon crack sealing. This is suggestive of an effectiveness “cap”: as pavements get better and better, the effectiveness of maintenance treatments reaches a natural maximum, and extra effectiveness cannot be obtained beyond a certain point. Finally, the results showed that all else being equal, the greater the precipitation, the lower the reduction in the rate of deterioration. Again, this seems to be counter-intuitive: pavements in areas of higher precipitation are more likely to have more water reach their subgrades, and are therefore more susceptible to wetting and subsequent weakening of their subgrade soils. Such pavements should therefore stand to benefit more from crack sealing treatments, compared to pavements in less wet areas. The fact that a result contrary to the above position was found is probably because for pavement in areas of high precipitation, the debilitating effects of sustained precipitation offset the benefits derived from crack sealing.

The results show that crack sealing is generally effective, but may not be effective in some cases. The results also show that it may be possible to estimate crack sealing effectiveness as a function of some

pavement and treatment attributes. The results seem to be generally consistent with the results of a study carried out for Indiana pavement routine maintenance [Sinha et al., 1988]. That earlier study expressed maintenance effectiveness as the change in pavement condition in the year following maintenance, and found that crack sealing was generally effective. The response variable in that study was a form of a Deterioration Reduction Level (*DRL*) response variable as explained in Chapter 5, where effectiveness was considered over a 1-year period. Therefore, it is not surprising that time-related variables such as traffic loading and regional (climate) factors were found to be significant in that study. It must be pointed out that the use of the *DRR* and *DRL* response variables for maintenance effectiveness may be accompanied by a serious limitation: over a 1-year period after maintenance, effectiveness of maintenance treatments (especially short-lived ones) may diminish to pre-maintenance levels and it may be erroneously inferred that the treatment is not effective. Furthermore, the model functional form utilized by previous studies did not allow for interpretation that could directly indicate the cap on maintenance effectiveness and the impact of zero maintenance.

The study results for crack sealing also generally seem to be in agreement with the results of the questionnaire survey (Chapter 4), even though the sub-districts' perceptions of short-term impacts crack sealing effectiveness appear to be somewhat higher than expected (Chapter 4). Some respondents to the survey indicated that the benefit of crack sealing depends on the condition of the pavement before application of the treatment, which is consistent with the model results. The short-term effectiveness of crack sealing, in terms of performance jump, has been vivid on many pavement sections such as US-31 in Tipton County, Indiana, in 1994.

The above results represent the behavior of the pavement system when traditional materials are used for crack sealing treatment. The effectiveness of crack sealing using crumb rubber was investigated in the subsequent section.

7.1.5 Effectiveness Model for Crack Sealing using Crumb Rubber

The method and purpose of crump rubber sealing is essentially similar to those of crack sealing with the exception that crump rubber is used in place of the traditional sealant material. Using annual road

condition and maintenance records for pavements that received only this treatment in the 1995 fiscal year, models for the reduction in the rate of pavement deterioration due to crumb rubber sealing was estimated. The 23 sections were rural Interstate and US Road pavements on the National Highway System, and were located at various geographical regions of the state, from the relatively cold and dry north, to the relatively warm and wet south.

Costs were expressed in terms of 1995 dollars. The above model form was selected from a variety of alternative model forms that were investigated on the basis of two criteria: goodness of fit to the observed data, and ability to provide engineering interpretation of the functional form. Descriptive statistics of data used for crumb rubber sealing treatment effectiveness are shown as Table 7-9 below.

Table 7-9: Descriptive Statistics for Crumb Rubber Sealing Effectiveness

	Maintenance Expenditure (\$1000s per ln-mi)	Initial Pavement Condition (PSI)	Annual ESALS (10 ⁶)	Annual Precipitation	Subgrade Quality	Deterioration Rate Reduction (PSI/Year)
Mean	918.94	2.989	2.742	42.733	14.900	0.318
Minimum	53.96	1.980	0.172	40.630	3.940	0.010
Maximum	2446.58	3.570	1.317	45.560	45.640	1.390
Standard Deviation	812.33	0.419	1.093	2.238	10.800	0.293
Coefficient of Variation	0.88	0.140	0.829	0.052	0.725	0.923

The descriptive statistics are shown in Table 7-9 shows that the deterioration rate reduction offered by crumb rubber sealing is much higher than that using the traditional material, which seems to justify the higher unit accomplishment cost of crumb rubber sealing. It is interesting to note that crumb rubber sealing specifically received rave reviews during the questionnaire survey of the sub districts and districts (Chapter 4). After several trials with a variety of mathematical forms for the crumb sealing DRR models, it was found that such effectiveness was best explained by the following functional relationship:

$$DRR = EXP[A_0 + \sum_{i=1}^N (A_i X_i)]$$

where DRR = Deterioration Rate Reduction, in PSI units per year

A_0 = constant term

A_i = coefficient of term X_i

X_i = explanatory variable i

The model results are presented in Table 7-10.

Table 7-10: Model for Effectiveness of Crumb Rubber Sealing

Variable	Coefficient Estimate	t-statistic	Remarks
Constant Term	-123.21	-8.42	
Traffic Loading	2.7785	8.05	Annual ESALS in millions
Precipitation	2.6347	8.31	Annual Precipitation
Subgrade Quality	-0.0202	-2.07	Function of %fines and plasticity index
Crack Sealing Effort	62.282	8.25	Expenditure (\$1000s) per lane mile
Response Variable		LN (DRR)	
R^2		0.75	

The model results show that sections that receive crumb rubber sealing exhibit greater reduction in their deterioration rates when traffic volume is higher. In other words, sections with light traffic stand to lose less if they are denied such treatment, compared to sections with heavy traffic. This suggests that traffic loading is an important consideration in the effectiveness evaluation of crack sealing in the long term. The model also showed a positive effect of precipitation on *DRR*. Pavement sections at areas with high precipitation are more vulnerable to greater amounts of water ingress through their surface cracks and consequent weakening of the subgrade. Such pavements stand to gain more from having their cracks sealed, compared with pavements at areas of low precipitation, all else being equal. It has been established from past research that subgrades characterized by low plasticity and low percentage of fines, such as gravels and coarse sands, lose relatively little or no strength upon wetting, while subgrades with high plasticity and a large fraction of fines lose much strength when they are wet. The variable “subgrade quality” and “subgrade vulnerability” were coined in the present study as simple functions of plasticity index and percent fines to represent the integrity and susceptibility, respectively, of the subgrade. The concept of subgrade vulnerability is similar to that of plasticity modulus used in some developing countries such as Ghana (Larbi-Yeboah, 1973). The model results showed that the lower the subgrade quality (i.e., higher vulnerability), the greater the reduction in the rate of deterioration, all else being equal. This means that pavement sections with vulnerable subgrades stand to gain more from crump rubber sealing compared

to pavement sections with good subgrades. Finally, the results showed that all else being equal, the greater the crump rubber sealing, effort, the greater the reduction in the rate of deterioration. This is consistent with expectation. The short-term cost-effectiveness, rather than just effectiveness, of alternative crack sealing treatments (traditional sealing versus crump rubber) could be investigated in a future study.

7.1.6 Effectiveness Model for Bump Grinding

The Field Operations Manual of INDOT's Operations Support Division describes this activity as "grinding or planning of bituminous surfaces to remove bumps, ripples and heaved joints. A model for the effectiveness of bump grinding (expressed as a jump in pavement surface condition/performance) was estimated using annual road condition and maintenance records for 25 pavements that received only this treatment in the 1995 fiscal year. The pavement sections were predominantly Interstate and US Roads.

Costs were expressed in terms of 1995 dollars. Descriptive statistics of data considered for investigating the effectiveness of bump grinding are shown as Table 7-11 below.

Table 7-11: Descriptive Statistics for Bump Grinding Effectiveness

	Maintenance Expenditure (\$100s per ln-mi)	Initial Pavement Condition (PSI)	Performance Jump (PSI)
Mean	0.876	3.023	0.202
Minimum	0.170	1.996	0.010
Maximum	1.717	3.471	0.599
Standard Deviation	0.719	0.323	0.146
Coefficient of Variation	0.820	0.107	0.725

The descriptive statistics are shown in Table 33 shows that performance jumps offered by joint grinding can be significant, as much as 0.2 PSI units. Joint grinding is typically done at a sub-district level.

It was found that the effectiveness of joint grinding was best explained by the following functional linear relationship:

$$PJ = A_0 + \sum_{i=1}^N (A_i X_i)$$

where PJ = Performance Jump, in PSI units

A_0 = constant term

A_i = coefficient of term X_i

X_i = explanatory variable i

The model results are shown as Table 7-12.

Table 7-12: Model for Bump Grinding Effectiveness

Variable	Coefficient Estimate	t-statistic	Remarks
Constant Term	0.087	2.23	
Bump Grinding Effort	0.132	3.97	(\$100's)
Response Variable		Performance Jump	
R^2		0.40	

The model results show that sections that receive bump grinding exhibit greater jumps in performance when the expended effort is greater, all else being equal.

7.2 Average Annual Pavement Maintenance Expenditure (AAMEX) Models

Pavement average annual maintenance expenditure (*AAMEX*) models estimate the level of maintenance that a pavement section is expected to receive over a period of time, given the attributes of the pavement, such as type, location, functional class, etc. *AAMEX* models may be considered short-term models because they provide expected expenditures that a pavement is expected to receive in 1-year. *AAMEX* models are needed for the present study because they enable the imputation of annual maintenance expenditure data for pavement sections lacking such data. More importantly, such models can be used for maintenance budgeting purposes. Expenditures are expressed in terms of constant dollar, as all expenditures were brought to their 1995 values in order to avoid errors due to inflation. Also, expenditures are given in terms of dollars per lane-mile, as lane-widths do not vary significantly with functional class.

A literature review of past pavement maintenance expenditures models is provided in Chapter 3, while a discussion of the methods used in *AAMEX* modeling, including selection of model form, response and explanatory variables, is provided in Chapter 5. The subsequent section provides the results of the

descriptive analysis based on the values in Table 34, and discusses the patterns that are revealed by such analysis. This is later followed by modeling of pavement annual maintenance expenditure using values for each pavement section and in each year. The sections below discuss the results of *AAMEX* modeling for the three main pavement surface types: Full-depth asphaltic concrete, rigid (PCC), and overlay (AC-over-PCC). Maintenance expenditure values used in the modeling covered all pavement maintenance work regardless of work source (by contract or in-house), application cycle (periodic and routine), or treatment role (preventive and corrective).

After several trials with a variety of mathematical forms for the *AAMEX* models, it was found that the average annual pavement maintenance expenditure levels were best explained by the relationship of the general form:

$$AAMEX = EXP[A_0 + \sum_{i=1}^N (A_i X_i)]$$

where

AAMEX = Average Annual maintenance expenditure per lane-mile, in 1995 constant dollar

A_0 = constant term

A_i = coefficient of term X_i

X_i = explanatory variable i

N = number of significant variables

The model results for each of the three major pavement surface types are presented in the next section.

7.2.1 *AAMEX* Modeling for Full-depth Asphaltic (FDA) Concrete Pavements

Model development was carried out to estimate the expected annual maintenance expenditure on full-depth asphaltic concrete pavements, in terms of 1995 dollars. Five hundred and seventeen FDA sections of various ages that received maintenance between 1991 and 1997 fiscal years were used for the

modeling process. Coefficient estimates and validation statistics for the model for the above pavement type are shown in Table 7-13:

Table 7-13: *AAMEX* Model Results for Full-depth Asphaltic Concrete Pavements ($R^2 = 0.34$)

Predictor	Coefficient Estimate	t-statistic	Meaning of Predictor Symbol
Constant	4.7089	14.4816	Constant term
<i>INT CLSS</i>	0.7926	2.0801	1 if Interstate 0 if Otherwise
<i>AGE</i>	0.0265	2.6324	Years since last rehabilitation
<i>WSL</i>	1.5780	3.2416	Weather Severity Level

The model results for full-depth asphaltic concrete pavements showed that functional class, weather severity and age are significant predictors of annual maintenance expenditures for that pavement type. It was found that all else being equal, FDA pavements on the Interstate Road system have higher annual maintenance expenditure compared to US Roads or State Roads. It is worth noting that there are relatively very few existing full-depth asphalt Interstate highways (I-265 in Lawrenceburg and I-64 in Warren County).

The sign of the t-statistic for *INT CLSS* indicates that full-depth asphaltic concrete pavements in Interstate pavements are associated with more maintenance than their counterparts on U.S. and State Roads, all other factors remaining the same. Also, it is seen that full-depth asphaltic concrete pavements in areas of more severe weather have more maintenance expenditures. This is obviously because weather severity is generally higher at colder regions. The lower temperatures associated with the northern parts of the state are particularly unfavorable to asphaltic concrete pavements as such conditions foster the development of transverse cracking on such pavements, an ominous precursor to further accelerated pavement distress. The deleterious effect of colder weather on AC pavements has been observed by many researchers that have carried out work in cold climates [Chong and Phang, 1988; Quin-Lin, 1988; Chong, 1990; Joseph, 1992]. These adverse effects of colder weather in northern Indiana obviously outweigh the benefits of relatively less rutting associated with pavements in colder regions.

Finally, the model estimates show that higher ages of full-depth asphaltic concrete pavement, result in higher levels of annual pavement maintenance expenditure. This is intuitive, as higher ages are associated with higher levels of accumulated traffic loading as well as greater levels of accumulated exposure to the vagaries of the weather.

Variables that were found statistically insignificant included a dummy factor to represent area class. This implies that the annual maintenance expenditure for full-depth asphaltic concrete pavements at rural areas is statistically the same as those in urban areas, all else being equal. A plot of the estimated average annual maintenance expenditure for full-depth asphalt pavements is provided as Figure 7-7.

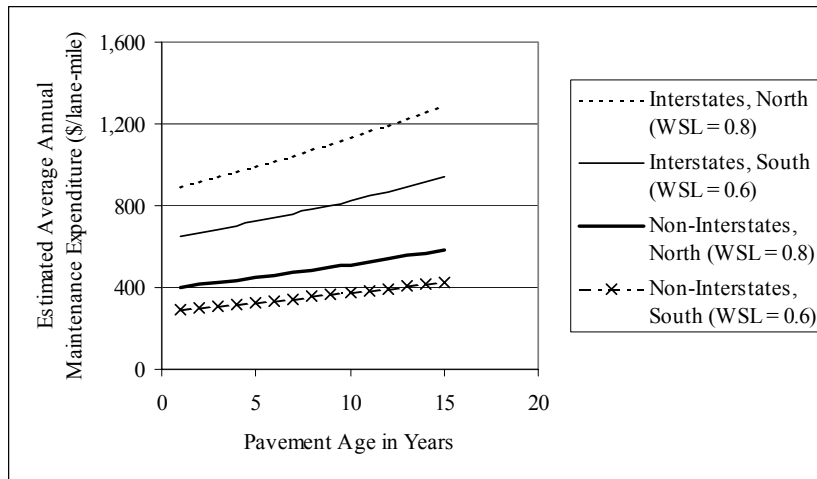


Figure 7-7: Fitted Values for *AAMEX* Model, Full-Depth Asphalt Pavements

7.2.2 AAMEX Modeling for AC-over-PCC Overlay Pavements

Model development was carried out to estimate the expected annual maintenance expenditure on AC-over-PCC overlay pavements, in terms of 1995 US dollars. Eleven hundred and seventy-two overlay sections that received some or no maintenance between 1991 and 1999 fiscal years were used for the modeling process. Coefficient estimates and validation statistics for the model for the above pavement type are shown in Table 7-14:

Table 7-14: *AAMEX* Model Results for Overlay Pavements ($R^2 = 0.24$)

Predictor	Coefficient Estimate	t-statistic	Meaning of Predictor Symbol
Constant	5.8515	35.1424	Constant term
<i>INT CLSS</i>	-0.4275	-4.5193	1 if Interstate 0 if Otherwise
<i>AREA CLSS</i>	-0.2252	-2.9627	1 if Rural 0 if Urban
<i>SOUTH F</i>	-0.3278	-4.4908	1 if Pavement in located in South 0 if Otherwise
<i>TRAD F</i>	0.2403	1.7624	1 if Traditional 0 if Crack-and-Seat or rubblized
<i>AGE</i>	0.0364	4.3831	Number of Years since Last Rehabilitation

Contrary to the finding for full-depth asphaltic concrete pavements, FDA maintenance expenditure is lower for Interstates than it is for overlay non-Interstates, all else being equal. This suggests that overlay Interstates are better equipped (by way of design and construction features) to withstand the agent of pavement deterioration (loading and weather) compared to overlay non-Interstates, while FDA Interstates are relatively less equipped to handle such effects compared to FDA non-Interstates. This finding is interesting especially considering that overlay Interstate pavements typically carry far heavier loads than their non-interstate counterparts.

The variable representing area class (*AREA CLSS*) was found significant in the annual maintenance expenditure model for overlay pavements. The negative t-statistic for this variable indicates that all else being equal, rural overlay pavements are associated with lower levels of pavement maintenance expenditure, compared to their urban counterparts. This appears to be a reasonable finding as rural pavements are associated with higher operating speeds, consequently less time of contact between the traffic load and the pavement surface, and therefore less pavement damage, all other factors being equal. Secondly, maintenance of pavements in urban areas is typically associated with higher costs as urban roadwork problems such as utility relocation are encountered.

The coefficient estimates for *SOUTH F*, a variable that represents the climatic region in which a pavement is located, showed that overlay pavements in the south are less expensive maintain than their central or northern counterparts, all other factors being equal. This is consistent with results of previous research as well as the analysis of pavement maintenance trends from a spatial perspective (Chapter 2), and is explained by the relatively mild climatic conditions in the southern part of the state, undulating terrain that fosters quick surface run-off from the pavement surface.

The model results showed that traditional overlay pavements are more expensive to maintain than rubblized pavements. Rubblization of rigid concrete slabs is a relatively new process in Indiana. It involves crushing of the existing concrete pavement to small pieces (prior to the AC overlay). This is done to prevent the overlying flexible layer from manifesting distresses that are rooted in the underlying concrete slab, a major problem typically encountered on AC overlays on untreated concrete [Kilareski and Bionda, 1997; Jayawickrama and Lytton, 1987]. Furthermore, rubblization provides for the asphaltic concrete layer a high-strength non-plastic yet relatively porous new base layer (features that are critical for pavement longevity in regions prone to extended freeze, freeze-thaw and high moisture regimes). If annual pavement maintenance expenditure levels are any measure of pavement longevity, then indications from this model are that rubblization of concrete pavements serves its intended purpose of increasing pavement life. In this study, rubblized pavements were considered as AC-over-PCC overlay pavements. However, the correct characterization of such pavements (i.e., whether overlay or full-depth asphalts) could be a subject of future investigation. Finally, higher ages of the pavement are associated with higher maintenance expenditure, which is quite intuitive.

A second model developed to estimate overlay pavement average annual maintenance expenditure utilized the weather severity indices of the pavements sections, in lieu of their regional locations (i.e., north/central/south). The model results for this specification are shown in Table 7-15.

Table 7-15: *AAMEX* Model Results for Overlay Pavements using Weather Severity Levels ($R^2 = 0.26$)

Predictor	Coefficient Estimate	t-statistic	Meaning of Predictor Symbol
Constant	4.4753	14.1506	Constant term
<i>INT CLSS</i>	-0.3663	-3.8719	1 if Interstate 0 if Otherwise
<i>AREA CLSS</i>	-0.2230	-2.9334	1 if Rural 0 if Urban
<i>WSL</i>	1.7261	-4.7938	Weather Severity Level of County in which Pavement is Located
<i>TRAD F</i>	0.2701	1.9807	1 if Traditional 0 if Crack-and-Seat or rubblized
<i>AGE</i>	0.0361	4.3516	Number of Years since Last Rehabilitation

The signs and magnitudes of the variables representing route type (*INT CLSS*), area class (*AREA CLSS*), type of overlay, i.e., traditional versus non-traditional (*TRAD F*), and pavement age (*AGE*) obtained in this specification were similar to those obtained for the earlier specification (Table 7-15). The coefficient estimates for the weather severity level variable, *WSL*, indicate that a higher weather severity is associated with higher maintenance costs, all else being equal. This is intuitive, as many distresses encountered on overlay pavements can be attributed to weather effects, as explained in Chapters 3 and 5.

Figures 7-8 and 7-9 present graphs of the fitted values for the estimated model using the latter specification, i.e., weather severity level variable to represent differences in weather effects, for Interstate and non-Interstate overlays, respectively.

Figure 7-8: Fitted Values for *AAMEX* Model, Interstate Overlay Pavements

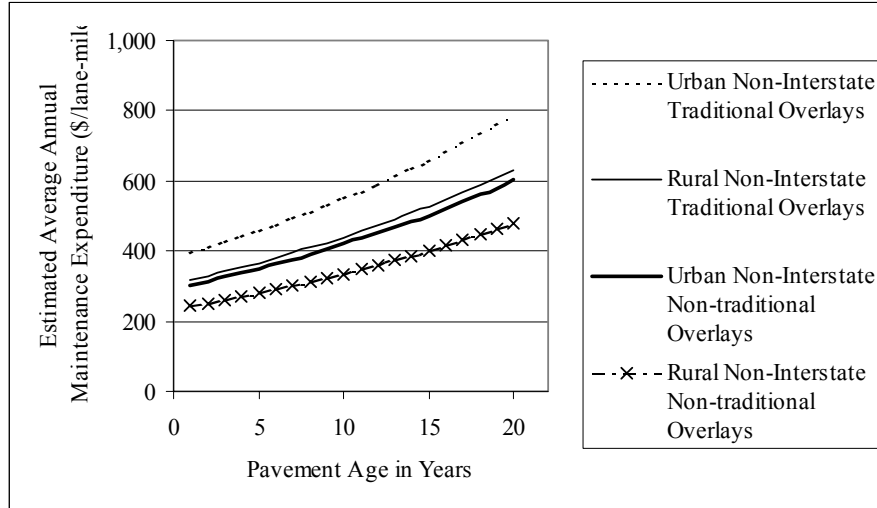


Figure 7-9: Fitted Values for AAMEX Model, Non-Interstate Overlay Pavements

7.2.2 AAMEX Modeling for Rigid Pavements

Model development was carried out to estimate the expected annual maintenance expenditure on jointed and continuous concrete pavements, in terms of 1995 US dollars. One hundred and ten rigid sections that received some or no maintenance between 1991 and 1999 fiscal years were used for the modeling process. Coefficient estimates and validation statistics for the model for the above pavement type are shown in Table 7-16.

Table 7-16: AAMEX Model Results for Rigid Pavements ($R^2 = 0.31$)

Predictor	Coefficient Estimate	t-statistic	Meaning of Predictor Symbol
Constant	5.1100	23.8412	Constant Term
<i>INT CLSS</i>	1.1012	4.3816	1 if Interstate pavement 0 if otherwise
<i>SR CLSS</i>	1.2012	4.7439	1 if State Road pavement 0 if otherwise
<i>AREA CLSS</i>	-0.6428	-3.5056	1 if Rural pavement 0 if otherwise
<i>CONF</i>	-1.0251	-2.8324	1 if CRC pavement 0 if otherwise
<i>AGE</i>	0.0459	3.9313	Pavement age in years

The coefficient estimates for the route type variables (*INT CLSS* and *SR CLSS*) showed that rigid pavements on state roads had the highest levels of pavement maintenance expenditure, followed by Interstates, while US Roads had the lowest expenditure, all other factors remaining the same. This suggests that rigid Interstate pavements in the state have design and construction features that equip them to withstand the effects of weather and traffic to a greater degree than such features on rigid state roads, but to a lesser degree than such features on rigid US Roads. The negative sign of t-statistic for area class (*AREA CLSS*) indicates that all else being equal, rigid pavements in rural areas are associated with lower levels of pavement maintenance expenditure, compared to their urban counterparts. The reasons for this have been explained in the model results for overlay pavement in the previous section. Also, the variable representing continuity of the rigid pavement concrete slab (*CONT F*) was significant, indicating that continuously reinforced concrete (CRC) pavements require less maintenance than their jointed counterparts, all other factors remaining the same. This is obviously because CRC pavements lack joints, and are therefore free from joint-related distresses and repairs that are quite common on jointed concrete pavements (JRC and JPC) in the state of Indiana. Figure 7-10 presents a graph of the fitted values for the model for estimating the average annual pavement maintenance expenditure of rigid Interstate pavements.



Figure 7-10: Fitted Values for *AAMEX* Model, Rigid Interstate Pavements

7.3 Chapter Summary

In this chapter, it was found that there are significant benefits associated with maintenance treatments, and that such short-term benefits generally involve an increase in pavement condition or a decrease in the rate of deterioration. For most treatments, a greater benefit is obtained for a larger effort expended in the maintenance treatment, at a given level of pavement condition. Also, a greater benefit is accrued for pavement in poor condition compared to those in fair condition, at a given level of maintenance. Also, annual pavement maintenance expenditure models were developed in this chapter as functions of pavement age, functional class, surface type and other pavement attributes. It was found that pavement expenditure is generally higher for pavements in the northern region compared to those in the southern region, and is also generally highest for state roads and lowest for Interstate roads. Also it is generally less expensive to maintain rigid than flexible (overly and full-depth asphalt) pavements.

CHAPTER 8: RESULTS OF LONG-TERM EFFECTIVENESS EVALUATION

8.1 Introduction

As seen in Chapter 7, maintenance treatments are associated with a short-term impact on pavement condition, either in the form of an immediate increase in pavement condition or slowed rate of deterioration. This information is not only useful to operators of Maintenance Management Systems, but is also a vital input to evaluation of maintenance effectiveness over the entire life cycle of the pavement. The present chapter discusses the results of the long-term evaluation of maintenance cost-effectiveness, which was conducted separately for each pavement family.

The first step in this aspect of the study was the development of performance models as a function of pavement type, climate, loading, maintenance, and other factors. The zero-maintenance curve was then determined by assigning the maintenance term a zero value. Strategies were formulated in a manner to ensure that a sufficient range of maintenance scenarios, consistent with the state of practice or the state of art (as found from the questionnaire survey and literature review, respectively) was represented.

For a given strategy, the effectiveness was measured as the extra benefit offered by that strategy (in terms of increased area under the performance curve) relative to the zero-maintenance curve. As each maintenance activity provides a certain jump in performance, results from the short-term effectiveness modeling (Chapter 7) were used to obtain the incremental gain. This was done for the entire life cycle of the pavement under a given strategy. An important assumption made is that the benefits of different treatments carried out at a given pavement section in a given year are independent of each other, so the total benefit in that year is simply a sum of the benefits of the individual treatments. Also, salvage values were assumed to be zero for all possible strategies.

The cost of each strategy was computed by summing up the agency cost (total cost) of the individual treatments as well as the user costs (expected delay and safety costs due to maintenance work zones). All costs were in constant 1995 dollars.

After determining the incremental benefits and incremental costs of each strategy relative to the zero-maintenance strategy, the cost-effectiveness index of that strategy was computed as the incremental cost-benefit ratio. It is worth noting that corrective maintenance (such as shallow and deep patching) will be, by default, part of each strategy, irrespective of the preventive maintenance composition of the strategy. Therefore, trade-off models were used to estimate the level of corrective maintenance every three years in response to preventive maintenance treatments in the preceding three-year period. The strategy with the highest value of cost-effectiveness was adjudged the optimal strategy for the pavement category in question. The entire procedure was repeated for each pavement family. The results of evaluation of maintenance in the long-term are provided below.

This chapter presents results of the evaluation of long-term maintenance effectiveness in the following sequence:

- Performance modeling for each pavement family
- Cost modeling for each maintenance treatment
- Formulation of strategies for each pavement family
- Determination of costs (using cost models) and benefits (using performance models), and subsequent computation of cost-effectiveness, for each of the several strategies formulated for each pavement family.

8.2 Performance Modeling

Pavement performance analysis was carried out as a preliminary step to long-term maintenance effectiveness evaluation. This was done to describe past performance and to predict future performance for each category of pavements, with an ultimate view to deriving the zero maintenance curve against which

incremental benefits and costs of all other strategies would be ultimately measured. The response variable used was PSI, which was estimated using the PSI-IRI roughness relationship derived for pavements in Indiana [Gulen et al., 1994].

The examination of temporal and spatial trends of pavement-related characteristics (Chapter 2) provided a background for the selection explanatory variables for the performance modeling. Explanatory variables considered included time-dependent stress and strength variables, and the time-independent stress and strength variables. Such variables were climatic attributes, pavement type, pavement loading, maintenance history, and design and construction features. Climatic attributes were expressed as a level of weather severity (which is a function of the levels of precipitation, freeze-index, and freeze-thaw cycles at a pavement location relative to the worst of such conditions in the state). The moment of the weather severity level experienced by a pavement in its lifetime was used as the climate variable for the performance modeling. Pavement types considered were: rigid, full-depth asphalt, and AC-over-PCC overlays. Pavement loading was measured in terms of the total moment of (rather than cumulative) ESAL values. ESAL values were computed using interpolations of the temporal distribution of ESAL factors developed for Indiana's pavements in 1980 and 2000 [Gulen et al., 2000], as discussed in Chapter 6. Road functional class was used as a surrogate to represent the contribution of pavement structural integrity, subgrade quality and other design and construction features.

Non-linear regression techniques were used to estimate the deterministic pavement performance models. Model validation and evaluation included as assessment of the coefficient of determination of the resulting models, and the root-mean-square values of the actual and estimated responses when the model was used to estimate the responses (PSI) for a section of the dataset that was excluded from the modeling process.

The performance models that were obtained for each family of pavements were of the following general functional form (chosen for closeness of fit to data and for intuitive engineering interpretation of the resulting curve):

$$PSI = A - \text{EXP}(P + Q*LOAD + R*WEATH + S*MAINT) \dots\dots\dots (107)$$

where,

PSI = Present Serviceability Index of the rigid pavement section in a given year,

LOAD = A measure of the load (in ESALS) experienced by the rigid pavement over its lifetime, up to the given year, (=Load Moment^{0.1})

WEATH = A measure of the weather effects (precipitation, freeze index, freeze-thaw cycles) experienced by the rigid pavement over its lifetime, up to the given year. (=Weather Moment^{0.1})

MAINT = A measure of the level of maintenance received by a lane-mile of the rigid pavement over its lifetime, up to the given year, (=Maintenance Moment^{0.1}), and

A, *P*, *Q*, *R*, and *S* are coefficients.

This model form was selected over several other mathematical forms because it provided relatively closest fit to the existing data points, and also because it enables intuitive engineering interpretation of the features of the resulting curve.

8.2.1 Rigid Interstate Pavements

Using 72 1-mile pavement sections in the state, the performance curve obtained for rigid interstate pavements was obtained as shown in Table 8-1.

Table 8-1: Performance Model Results for Rigid Interstates ($R^2 = 0.42$)

Coefficient	Estimate	t-statistic
<i>A</i>	4.3985	12.3415
<i>P</i>	-9.2518	-3.1213
<i>Q</i>	2.5648	1.9936
<i>R</i>	3.1254	2.2215
<i>S</i>	-0.2539	-1.5110

The signs and magnitudes of the t-statistics appear consistent with expectation. The constant term, *A*, is significant, and has a value of 4.3985. Also, the constant *P* has a value of -9.2518. This implies that just after construction of rigid pavements (i.e., at zero age), the moments of all factors equal zero, and

consequently PSI takes the following value: $4.3985 - e^{-9.2518 + 0} = 4.3984$ units. This extrapolated PSI value which represents the typical surface condition of rigid Interstate pavements immediately after construction, indicates that such pavements typically do not attain a perfect 5.0 PSI after construction as assumed or implied in most studies. In the current situation, road surface condition measurements are carried out just after rehabilitation or reconstruction, as part of the pavement warranty system that has been adopted by INDOT. With this practice, post-construction pavement condition data (which are preferred to extrapolated values) will be made available.

The t-statistics for the coefficients of the load moment (*LOAD*) and weather moment (*WEATH*), *Q* and *R*, respectively, are positive and significant. This means that all else being equal, the higher the load moment experienced by the pavement up to the year under consideration, the poorer the pavement condition. A similar explanation is offered for weather effects. The maintenance variable has a negative t-statistic, indicating that higher values of maintenance experienced in the previous life of a pavement, all else being equal, leads to better pavement condition. These results are consistent with expectation.

8.2.1.1 Boundary Condition Performance Curves for Rigid Interstates

Boundary condition curves represent the shape of the performance curve when any one or two of the three dynamic pavement deterioration factors assume a value of zero. These are as follows: zero-maintenance curve, zero-load curve, zero-weather (vacuum) curve, load-only curve, and the weather-only curve. With the models developed in this chapter, boundary condition performance curves can be determined. However, the present study investigates only the zero-maintenance curve, because only that boundary condition is germane to the objectives of the study. The zero-maintenance curve served as the base case against the cost-effectiveness of each formulated maintenance strategy was determined.

If maintenance is zero, the developed performance model becomes:

$$PSI = 4.3985 - \text{EXP}(-9.2518 + 2.5648*LOAD + 3.1254*WEATH)$$

8.2.2 Rigid Non-Interstate Pavements

Using 55 1-mile rigid non-Interstate pavement sections in the state, the performance curve obtained for such pavements was as follows (Table 8-2):

Table 8-2: Model Results for Rigid Non-Interstates ($R^2 = 0.33$)

Variable/Constant	Coefficient Estimate	t-statistic
<i>A</i>	4.0200	4.3125
<i>P</i>	-9.0780	-2.9317
<i>Q</i>	2.8094	3.1442
<i>R</i>	3.3973	2.5353
<i>S</i>	-0.0972	-1.9883

The model obtained was generally similar to that for rigid Interstate pavements. The constant term is positive and has a significant t-statistic. Also, the t-statistic for the load and weather term are positive and large, indicating those higher levels of weather and loading each have an influential but adverse impact on pavement condition. Also, the maintenance term is significant and negative, suggesting that maintenance plays a significant role in reducing pavement deterioration, for pavements in this category.

The model for non-Interstate rigid pavements bears slight differences compared to that for rigid Interstates. The constant term, *A*, is less than for rigid Interstates. Also the absolute value of the “*P*” constant is less for non-Interstate rigid pavements. This means that that the initial (post-construction) pavement condition is higher for rigid Interstate pavements than it is for rigid non-Interstate pavements. This is probably evidential of the fact that higher standards of surface finish are typically specified for higher-class pavements. Also, it is seen that the coefficients for the load and weather term are higher for Interstate rigid pavements than for non-Interstate rigid pavements. This is reflective of the relatively higher impact of these factors on non-Interstate pavements compared to Interstate pavements, all else being equal. A likely explanation is that all else being equal, rigid Interstate pavements have better design and construction features than rigid non-Interstates, and are therefore relatively less vulnerable to the debilitating effects of weather and traffic. It is noticed that the absolute value of the coefficient estimate of the maintenance term for rigid non-Interstate pavements is about a third that for rigid Interstates.

8.2.2.1 Boundary Condition Performance Curves for Rigid Non-Interstates.

The zero-maintenance curve for rigid non-Interstates is determined by equating the maintenance term to zero as follows:

$$PSI = 4.020 - \text{EXP}(-9.0781 + 2.8094*LOAD + 3.3973*WEATH)$$

8.2.3 Overlay Interstate Pavements

Constituting approximately 80% of all Interstate pavements in the state, overlay (AC-over-PCC) pavements are an important category of pavements in Indiana. The dataset for modeling the performance of such pavements excluded rubblized and crack-and-seat overlay pavements. This is because of their relatively limited mileage, and more importantly because of the relatively young ages of such non-traditional overlay types would very likely introduce statistical bias in the modeling process. Therefore only pavements with traditional overlays (where no mechanical treatment was applied to the existing pavement prior to overlay) were used for the modeling. The resulting model is shown in Table 8-3.

Table 8-3: Model Results for Overlay Interstates ($R^2 = 0.23$)

Variable/Constant	Coefficient Estimate	t-statistic
<i>A</i>	4.5001	9.3614
<i>P</i>	-9.1058	-3.1334
<i>LOAD</i>	2.7662	2.6218
<i>WEATH</i>	3.3536	3.9391
<i>MAINT</i>	-0.1427	-1.8215

The modeling results were quite intuitive. Higher values of weather experienced in the lifetime of an overlay pavement translate to lower pavement condition, all else being equal, as reflected in the positive sign of the t-statistic. Furthermore, overlay Interstate pavements that have suffered higher levels of traffic loading have lower pavement condition, all other factors remaining the same. Also, lower PSI values are associated with higher levels of maintenance effects suffered by a pavement in its lifetime, all other factors being equal. The condition of overlay Interstate pavements just after reconstruction or resurfacing is given as $4.54 - e^{-9.1058} = 4.53$. This shows that overlay Interstate pavements rarely start off with a perfect 5.0 PSI,

yet have an initial condition that exceeds that of their rigid counterparts. The coefficient for the maintenance variable is negative, indicating that higher levels of maintenance received in the past life of pavements of this type results in better pavement condition (i.e., higher PSI values) all other factors remaining constant.

8.2.3.1 Boundary Condition Performance Curves for Overlay Interstates.

If maintenance is zero, the developed model becomes:

$$PSI = 4.020 - \text{EXP}(-9.0781 + 2.8094*LOAD + 3.3973*WEATH)$$

8.2.4 Overlay Non-Interstate Pavements

Overlay non-Interstate pavements in Indiana typically traditional overlays found on US Roads and State Roads. These pavements comprise approximately 30% of all pavements in the state. The performance curve obtained for overlay non-Interstate pavements was as follows (Table 8-4).

Table 8-4: Model Results for Overlay Non-Interstates ($R^2 = 0.39$)

Variable/Constant	Coefficient Estimate	t-statistic
<i>A</i>	4.0231	13.1354
<i>P</i>	-9.0428	-2.3541
<i>LOAD</i>	2.8585	3.1658
<i>WEATH</i>	3.4528	1.9352
<i>MAINT</i>	-0.0389	-2.0652

The above model results were generally similar to that obtained for overlay Interstate pavements. The interpretation of the signs and magnitudes of the t-statistics of the various variables are same as those as for overlay Interstate pavements. The constant terms *A* and *P* are positive and negative respectively, implying that for zero values of the dynamic variables, (i.e., at zero age), pavement condition is close to 4.02, which is significantly less than that for overlay Interstate pavements. An obvious reason for this is that surface finish tolerances are more stringent for Interstates, which are considered roads of a higher class. The coefficients of the load and weather variables, for the overlay non-Interstate pavements are higher than it they are for their Interstate counterparts. This finding is inferential of the greater effect of

traffic loading and weather on non-Interstate overlay pavements compared to Interstates, which is in turn obviously due to the relatively superior static factors (design, and construction features, sub grade quality, etc) of the non- Interstate pavements. Furthermore, it is observed that the absolute value of the maintenance coefficient is lower for non-Interstate pavements than for Interstate pavements. This is suggestive of the lower cost-effectiveness of pavement maintenance for non-Interstate overlays relative to their Interstate counterparts. In other words, the returns (PSI reduction) yielded by each maintenance dollar invested is higher for Interstates than that is for non-Interstates, a finding which is counter-intuitive given the generally lower pavement condition of non-Interstates (and have higher performance jumps). However, this may be explained by the fact that Interstate pavement are built to higher standards of quality and are therefore more receptive to the prophylactic effects of maintenance treatments. Results from the questionnaire survey (Chapter 4) seem to support this explanation. In that survey, pavement thickness, subgrade quality, and drainage were cited as some of the most influential factors that influence than ability of a pavement to sustain performance after maintenance, and these are qualities in which overlay Interstate pavements are significantly superior, compared to non-Interstate overlays.

8.2.4.1 Boundary Condition Performance Curves for Overlay Non-Interstates.

If maintenance is zero, the model shown in Table 41 becomes:

$$PSI = 4.0231 - \text{EXP}(-9.0428 + 2.8585*LOAD + 3.4528*WEATH)$$

8.2.5 Full-depth Asphaltic Concrete Pavements

Full-depth AC pavements constitute over 60% of all pavements in the state. Most of such pavements are on non-Interstate highways. The only full-depth asphaltic concrete pavements on Interstate system, I-64 in the southern Indiana, and I-265 in south-western Indiana, were excluded from the model development for this pavement family due to their relatively little contribution to the overall mileage of pavement in this family. Model development for the performance trend of full-depth asphaltic concrete pavements yielded the following function:

$$PSI = 3.85 - \text{EXP}(-8.9674 + 2.9561*LOAD + 3.5885*WEATH - 0.0176*MAINT) \dots\dots\dots (111)$$

$$(8.2354) \quad (-1.6524) \quad (2.2141) \quad (2.1541) \quad (-1.3655) \quad R^2 = 0.28$$

Where symbols have their usual meanings. The t-statistics are shown in parenthesis.

In general, the signs of the various constants and variables were similar to those of the rigid and overlay pavements performance models. The model results showed that increased loading or weather moment, all else being equal, leads to lower PSI values (i.e., lower pavement condition). Also, it was found that with a negative t-statistic of the maintenance variable, higher levels of maintenance leads to higher PSI values, all else being equal. The constants *A* and *P* were found as 3.85 and -8.9674 respectively, indicating that new pavements in this category assume values of $3.85 - e^{-8.9674} = 3.84$ units. This rather low starting PSI is probably a result of relatively low quality control and pavement smoothness specifications for road functional classes that are dominated by such pavement types. Previous research shows that pavements with poor initial smoothness tend to have faster rates of deterioration, all else being constant [Smith et al., 1996]. In this respect, it can be inferred that the typically low starting PSI value of a full-depth asphalt pavement can be considered as one of factors that contribute to the relatively short life span observed for such pavements.

8.2.5.1 *Boundary Condition Performance Curves for Full-Depth Asphalt Pavements*

If maintenance is zero, the model shown is Equation 112 becomes:

$$PSI = 3.85 - \text{EXP}(-8.9674 + 2.9561*LOAD + 3.5885*WEATH)$$

8.3 Development of Maintenance Unit Accomplishment Cost Models

Long-term maintenance policies typically involve strategies that are simply a “collection” of one or more maintenance treatment types carried out at various points in time on a given pavement. The costs of the treatments provide a means to determine the cost aspect of cost-effectiveness analyses. Maintenance treatment unit accomplishment cost (UAC) models typically express the cost of a treatment in terms of dollars per unit output (tons, lane-miles, linear miles, etc). For a given maintenance treatment, the variation in unit accomplishment costs are typically due to variations in pavement attributes (such as location, condition, etc) on one hand, and treatment attributes such as type (alternative material or process), work source (in-house or by-contract) on the other hand. Using treatment levels and annualized cost data for various maintenance treatments received by pavements within the study period, models were developed to estimate the unit costs of various treatments. Details are provided below. All costs indicated are in constant 1995 dollars. The source of the data is annual reports generated by INDOT’s maintenance management system.

8.3.1 Crack Sealing

Sealing of cracks is described as the placement of specialized materials either above or into pavement surface cracks with the aim of preventing intrusion of surface moisture and incompressible matter into the cracks [McGhee, 1996; INDOT, 1998]. This treatment is typically carried out in-house by INDOT (i.e., by the sub-districts) on a force account basis on a recurring cycle of length 1-4 years. Crack sealing unit costs reported by INDOT are per lane-mile rather than the number of cracks or the volume of material used for sealing. Consequently, this rate does not consider the severity of the cracking problem that the treatment addresses. In other words, the cost per lane-mile of crack sealing on a pavement section with extensive and severe cracking will be very different from that on another with infrequent cracks. This probably explains why there is so much variation in crack sealing unit costs (coefficient of variation of 117%, see [Table 8-6](#)). The mean unit cost of crack sealing is \$444.19 per lane-mile, but the large variation associated with this statistic renders it inappropriate for use as a reliable predictor of such unit costs. If

crack sealing is to be related to pavement condition, then it is useful to develop a model that expresses the unit cost of crack sealing as a function of the cracking index. However, this data is not always known. Therefore in the present study, crack sealing unit cost was modeled directly as a function of the factors that influence crack development. The model obtained and t-statistics are as follows:

$$UNIT_COST = \underset{(42.91)}{439.96} - \underset{(-2.94)}{56.1 * S_FACTOR} \dots\dots\dots (112)$$

where $UNIT_COST$ = cost of crack sealing in \$1995 dollars per lane-mile treated
 S_FACTOR = 1 if treatment is carried out in the south, 0 if otherwise

The adjusted R^2 is 0.39. The estimated model shows that the unit cost of crack sealing is lower for pavement in the southern part of the state ($S_FACTOR = 1$). This is obviously because southern pavements suffer less incidence of cracking compared to their northern or central counterparts, all other factors being equal, but could also be explained by the relatively lower prices of aggregates and labor in the southern part of the state.

8.3.2 Crumb Rubber Sealing

Sealing of cracks using crumb rubber (a blend of waste tires and asphaltic cement) has a similar purpose to that using traditional sealing materials, but utilizes different equipment. The unit cost of crumb rubber sealing is approximately twice that of the traditional treatment. At INDOT, unit costs for crumb rubber sealing are expressed in lane-miles, a unit which renders such rates subject to marked variation for reasons stated above. The average cost of crumb rubber sealing in Indiana is \$714.21 per lane-mile, with a coefficient of variation of approximately 27% (Table 8-6). A model was developed for this treatment type yielded the following coefficients estimates:

$$UNIT_COST = \underset{(31.61)}{795.29} - \underset{(-6.64)}{360.77 * S_FACTOR} \dots\dots\dots (113)$$

where $UNIT_COST$ = cost of crumb rubber sealing in \$1995 dollars per lane-mile
 S_FACTOR = 1 if treatment is carried out in the south, 0 if otherwise

The adjusted R^2 is 0.62. The model results indicate that like crack sealing, the unit cost of this treatment type is less in the south than it is in the upper regions of the state.

8.3.3 Premix Leveling

Indiana’s sub-districts typically fill local pavement depressions with a blend of asphaltic cement and coarse aggregate. This treatment, known as premix leveling, is measured in tons of material used. From Table 8-6, the average cost of premix leveling is \$70.54 per ton. This cost includes equipment use. The use of a grader typically lowers the unit cost, while the use of rollers increases the unit costs but is associated with a more durable treatment [Feighan et al., 1985]. The coefficient of variation is 15%. The model developed for premix leveling unit costs yielded the following estimates:

$$UNIT_COST = \frac{62.14}{(26.40)} - \frac{14.71}{(4.83)} * F_CLASS \dots\dots\dots (114)$$

where $UNIT_COST$ = cost of premix leveling in \$1995 dollars per ton
 F_CLASS = 1 if treatment is carried out on an Interstate pavement, 0 if otherwise

The adjusted R^2 was 0.44. The model results showed the only influencing factor is the functional class of the pavement section. The sign of the t-statistic for this variable indicates that all else being equal, the unit cost of premix leveling is higher for Interstate pavements than it is on non-Interstate pavements, all else equal. This is probably because work on Interstate pavements are carried out to higher standards, and therefore extra time and care is taken to ensure that the finished surface is of highest quality, which translates to higher unit costs.

8.3.4 Joint Bump Repair

Shoving of asphaltic concrete mixes on flexible pavements and faulting of joints on rigid pavements lead to surface irregularities that are typically addressed by the sub-districts by mechanical grinding of the distressed spot down to appropriate level. This preventive maintenance treatment helps retard the rate of deterioration and therefore defers the conduction of corrective maintenance treatments at affected locations. The mean unit cost of joint/bump repair is \$73.01 per bump. Because the unit cost of this treatment is measured per number of joints/bump locations ground, differences in unit costs may vary from one location to another depending on the dimensions of the problem (a higher bump would take more equipment and labor time to grind). This explains why there is so much variation in the unit rates for this treatment (coefficient of variation = 55%, from Table 8-6). Using data from pavement sections that received of joint/bump repair within the study period, estimation of the cost of this treatment was modeled as follows:

$$UNIT_COST = 41.6 + 15.0 * N_FACTOR + 21.1 * SOUTH_F + 20.0 * F_CLASS \dots\dots\dots(115)$$

(12.81)
(3.52)
(4.89)
(5.51)

- where *UNIT_COST* = cost of joint bump repair in \$1995 dollars per bump treated
- N_FACTOR* = 1 if treatment is carried out in the north, 0 if otherwise
- S_FACTOR* = 1 if treatment is carried out in the south, 0 if otherwise
- F_CLASS* = 1 if treatment is carried out on an Interstate pavement, 0 if otherwise

The adjusted coefficient of determination was 0.63. The model showed that the unit costs of joint bump repair was higher in location of weather extremes, either the relatively warmer south and north compared to the central part of the state. The only common weather feature that is common to the north and south regions is the fact that they each have a higher combined (rain and snow) precipitation than the central part of the state. This suggests that precipitation may be a major factor that leads to joint bump distress. Also, higher functional classes are associated with higher unit costs of joint bump repair, all else

being equal. Obviously, for a given level of severity, it is expected that field crews take care to ensure a better-finished product on Interstate highways.

8.3.5 Underdrain Maintenance

The cost of maintaining underdrains (which involves inspection and cleaning of such drains using specialized equipment) is measured by the number of such structures. Obviously, the more severe the problem (i.e., the greater the amount of debris in the pipes), the longer the equipment and labor time (and hence, cost) that will be involved in this treatment. Using data at sections that receive this treatment between 1995 and 1996, and average unit costs of \$5.81, and a coefficient of variation of 24.6 % was obtained. The cost model for this treatment was obtained as follows:

$$UNIT_COST = 5.95 + 1.25 * N_FACTOR - 1.36 * F_CLASS \quad \dots\dots\dots(116)$$

(16.32)
(2.92)
(3.22)

where

UNIT_COST = cost of underdrain maintenance in 1995 dollars per drain

N_FACTOR = 1 if treatment is carried out in the north, 0 if otherwise

F_CLASS = 1 if treatment is carried out on an Interstate pavement, 0 if otherwise

The adjusted R^2 obtained was 0.50. The model results show that northern pavements are associated with higher costs of maintenance of this type. This finding seems contrary to expectation: Unlike those in the sandy North, pavements in the south are underlain by finer materials that are more likely to percolate through the underdrain pipe perforations with greater ease, either from below borne by rising groundwater or capillary moisture, or from above through the ingress of surface run-off through pavement cracks or distressed joints. However, it may be argued that the southern part of the state has a rolling topography that facilitates surface runoff into the side ditches, and therefore lessens the ingress of silt-bearing water into any pavement surface cracks. Also a greater part of the south is underlain by a porous bed of limestone [Fenelon et al., 1994], which enhances sub-surface drainage below the pavement layers.

With such topographical and geological features, the adverse effect of a generally finer subgrade (in the south) on underdrain performance, is obviated. Consequently, southern pavements' underdrain pipes get clogged less and therefore require less effort (unit cost) of treatment. The negative sign of the t-statistic for functional class indicates that underdrain maintenance is less expensive on Interstates than it is on non-Interstate highways. As stated previously, the unit cost of underdrain maintenance depends on the time taken to carry out this treatment, which in turn depends on the volume of material that needs to be flushed out of the underdrain. A lower unit cost for Interstate underdrain maintenance implies that these drainage structures on such higher class highways get clogged less often, which probably attests to the fact that such highways are built to higher standard, e.g., higher embankments, better grades to facilitate quick surface runoff, etc.

8.3.6 Seal Coating

The surface of low-volume non-Interstate flexible pavements in Indiana typically receive seal coating, a “moderate” preventive maintenance treatment, in the middle years of their service lives in order to fill light cracks, improve skid resistance, and to rejuvenate the pavement. Seal coating involves mechanical spreading of asphaltic cement followed by fine or coarse aggregate. Also, seal coating is typically carried out in-house by the sub-districts, and to a lesser extent, by contract. Unlike localized treatments such as crack sealing and shallow patching, seal coating covers the entire surface of the pavement, and is therefore appropriately measured in terms of lane-miles. Using 46 sections that received this treatment between within the study period, an average unit cost of \$4799.69/lane-mile was obtained, with a coefficient of variation of 158.8%. The high value of the variation is due to the fact that seal coating unit costs are expected to vary by size of aggregate used (chips vs. sand), and by the source of work (in-house vs. by-contract). In this respect, the use of mean unit costs is not recommended. The development of a model to estimate the unit costs of seal coating yielded the following estimates:

$$UNIT_COST = 3848.10 - 3170.56 * AREA_F + 744.61 * AGG_SIZE + 17554 * WRK_SRCE \dots(117)$$

(14.21)
(1.82)
(3.52)
(9.35)

Where $UNIT_COST$ = cost of seal coating in \$1995 dollars per lane-mile

$AREA_F = 1$ if pavement is in a rural area, 0 if urban

$AGG_SIZE = 1$ if chips are used, 0 if sand is used

$WRK_SRCE = 1$ if work was done by contract, 0 if done in-house

With an adjusted R^2 of 0.63, this model does a fairly good job of estimating the unit costs of seal coating as a function of attributes of both pavement and treatment. The negative sign of the coefficient for the $AREA_F$ variable indicates that all else being equal, it is less expensive to carry out seal coating in rural areas than it is in urban areas, probably due to the extra effort associated with traffic control, utilities and other features peculiar to pavements in the urban areas. Also, the use of chips ($AGG_SIZE = 1$) is indicative of higher unit costs all else being equal. This is due to the higher price of chips. The most influential variable in the seal coating unit cost model is the source of work. The very large and positive value of the t-statistic for this variable's coefficient confirms that the unit cost of seal coating is significantly higher when it is carried out by contract. This is expected because the unit cost of seal coating by contract, for each section was calculated as the total contract sum divided by the amount of lane-miles covered. Such contract sums include any surface preparatory and auxiliary work and the contractor's profit, while in the case of in-house work seal coating costs are reported separately from such externalities.

Finally, it is significant to note that the condition of the pavement at time of seal coating was found to be insignificant in the unit cost model for this treatment. This is intuitive for two reasons: Seal coating is carried out over the entire surface, with no extra material or effort expended on localized distressed areas. In other words similar levels (and therefore, cost) of seal coating is expected on a lane-mile of pavement section in poor condition as on one in fair condition), all other factors being equal. Secondly, the cost of seal coating is far in excess of the cost of surface preparatory works, so the extra effort expended to prepare a poor pavement for seal coating may be insignificant in relation to the total cost of this treatment.

8.3.7 Thin HMAC Overlays

Thin overlays, which involve the laying of a relatively thin (up to 1.5”) layer of gap-, open- or dense-graded Asphaltic Concrete mix, are carried out only by contract in Indiana for pavement sections that have experienced 10-20 years of service. Thin overlay costs are measured by lane-miles and material thickness, which together represent the volume (or tonnage) of material used. The cost of each ton of HMAC mix is fairly constant. For this reason, the costs of thin overlays are expected to be fairly *stable* regardless of pavement location, type, and other attributes. However, a close examination of thin overlay contracts carried out within the study period reveals the contrary. With a minimum and maximum of \$32,000 and \$118,000 respectively, and a coefficient of variation of 37%, it is obvious that the costs of overlay treatments per lane-mile are influenced by other factors beside the basic unit cost of HMAC mix which is generally constant. Thin overlay contracts include contractor mobilization sums, surface preparatory work (such as milling), and other expenses. The model developed for thin overlay costs yielded the following coefficient estimates (Table 8-5):

Table 8-5: Model Estimates for Unit Cost of Thin HMAC Overlay Treatment
(Response Variable is LogUNIT_COST) $R^2 = 0.65$

Variable	Coefficient	t-statistic	Meaning of Variable Names
Constant	4.6133	66.19	
<i>AREA_F</i>	-0.1515	-4.26	=1 if rural =0 if urban
<i>LENGTH</i>	-0.0113	-2.84	Length of section that received overlay (miles)
<i>ADDED_THK</i>	0.1737	3.72	Thickness of the overlay, in inches
<i>INT_F</i>	-0.0921	-1.75	=1 if Interstate =0 if non-Interstate
<i>MILL_DEPTH</i>	0.0574	2.52	Depth of milling prior to overlay, in inches

The estimated model showed that the unit costs of thin overlay treatments are higher in urban areas than it is in rural areas, all else being equal. This is obviously because of the problems typically faced by construction in urban areas (such as utility relocation), which translate to higher costs of contracts in such areas. Such extraneous costs do not affect the price (costs) of HMAC mix, but affects the overall contract

sum for laying such material. Also, the model showed that shorter sections of pavement that received thin overlays were significantly more expensive, in unit costs, compared to longer sections. This is expected because of economies of scale: all contracts consist of two components, a fixed sum (such as mobilization) and a variable sum that depend on the amount of work (such road length). For shorter stretches, the fixed sum tends to increase contract sum in a manner that is proportionately higher than for long stretches.

The thickness of the HMAC overlay was also found to be a significant factor that affects the unit cost of thin overlays, which in this study, is measured per lane-mile. Obviously, thicker overlays require more material, labor and equipment time, and lead to higher unit costs, all else being equal. The negative value of the t-statistic for Interstate indicates that all other factors being equal, the unit cost of thin overlays on Interstates highways are lower for Interstates than it for non-Interstates. In view of the fact extra care is taken to ensure a better finish on Interstates pavements, such a finding may seem contrary to expectation. However, it is worth noting that surfaces of Interstates pavements are generally not allowed to deteriorate to a point where extensive surface preparatory works are required prior to overlay, and are therefore associated with lower overall unit costs of thin overlays. A variable was specified to explore the possible difference in the unit costs on US Roads and State roads, but was found to be insignificant, implying that the unit costs of such treatments are statistically same for these two road classes, all else being equal.

Finally, the depth of milling was found to be significant: the deeper the milling prior to an overlay, the higher the overall cost of the overlay, all other factors remaining constant, because of the higher contractual rates associated with deeper milling.

8.3.8 Micro-surfacing

This treatment, which involves the laying of a mixture of asphalt emulsion, mineral filler, aggregate, and water, was started in the state of Indiana only in the mid 1990's. No rolling of this material is required. Using data from 11 sections that received this treatment within the study period, descriptive statistics of the unit cost of this treatment type are provided in [Table 8-6](#). The average cost of micro-surfacing is \$27,434 per lane-mile, with a coefficient of variation of 29%. The number of sections was too

few to permit model development. In future, after more sections receive this treatment, it may be appropriate to develop a model that would estimate the unit costs in terms of pavement attributes.

8.3.9 Shallow Patching

A corrective maintenance treatment used to address patch deterioration and pothole development on both rigid and flexible pavements, shallow patching is typically carried out in-house by INDOT’s sub-districts. This is the most common activity carried out by the sub-districts, accounting for over 30% of the overall pavement maintenance expenditure. From Table 8-6, the unit cost of shallow patching is \$302.54 per ton, with a considerably large coefficient of variation of 26%. A shallow patching unit cost model was developed, and the following coefficient estimates were obtained:

$$UNIT_COST = 252.33 - 39.80 * NORTH_F - 30.11 * SOUTH_F + 140.34 * F_CLASS \dots\dots\dots(118)$$

(20.45)
(-2.65)
(-1.93)
(11.17)

where

- UNIT_COST* = cost of shallow patching in \$1995 dollars per ton of material
- NORTH_F* = 1 if treatment is carried out in the north, 0 if otherwise
- SOUTH_F* = 1 if treatment is carried out in the south, 0 if otherwise
- F_CLASS* = 1 if treatment is carried out on an Interstate pavement, 0 if otherwise

An R² of 0.85 was obtained. The modeling results indicated that the unit cost of shallow patching was highest for pavements in the central region, followed by those in the northern region, and lowest for the south. The unit costs of shallow patching, even though it is expressed in terms of the volume of material, includes equipment as well as labor costs. For a given level of patching distress, a higher unit cost of shallow patching may be due to a greater level of effort or time expended for the repair activity. Where work culture is less efficient in terms of resource use, or, where extra care is taken to ensure a better product, the unit costs of shallow patching is expected to be higher. The “pavement type” variable was

found to be insignificant at 10% significance level, implying that the unit costs of shallow patching are statistically same for this treatment on either rigid or flexible pavements.

8.3.10 Deep Patching

Deep patching is a maintenance treatment used to correct pavement structural distress caused by base failure, blowup, or settlement. The mean unit cost for deep patching is \$227.46 per ton, with a coefficient of variation of 40% (Table 8-6). The model developed for this treatment type was found as:

$$UNIT_COST = 179.70 - 65.99 * SOUTH_F + 135.65 * F_CLASS \dots\dots\dots (119)$$

(11.39) (-2.69) (6.36)

where

$UNIT_COST$ = cost of deep patching in \$1995 dollars per ton of material

$SOUTH_F$ = 1 if treatment is carried out in the south, 0 if otherwise

F_CLASS = 1 if treatment is carried out on an Interstate pavement, 0 if otherwise.

The R^2 obtained was 0.72. The model results showed that pavements in the southern part of the state had lower unit costs of deep patching, compared to those in the central or north. This is probably explained by differences in institutional practices. Also, like shallow patching, deep patching has higher unit costs for Interstates than for non-Interstates.

Table 8-6 presents a summary of costs associated with various maintenance treatments.

Table 8-6: Summary Statistics of Unit Costs of Various Maintenance Treatments (\$1995 Values)

	Treatment Type	Units	Mean	Maximum	Minimum	Standard Deviation	Coefficient of Variation
“Minor” Preventive Maintenance	Crack Sealing	Lane-miles	444.19	2446.58	20.16	630.55	117.51%
	Crumb Rubber Sealing	Lane-miles	714.21	1041.65	396.96	192.64	26.97%
	Joint/Bump Repair	Number	73.00	307.15	20.64	40.61	55.61%
	Undrain Maintenance	Number	5.81	8.89	3.60	1.54	24.54%
“Moderate” Preventive Maintenance	Seal Coating	Lane-miles	4799.69	25,624.82	216.82	7622.61	158.81%
“Major” Preventive Maintenance	Micro-surfacing	Lane-miles	27,434	37,755	18,427	7,933	28.92%
	Thin HMA Overlay	Lane-miles	61,664	118,349	30,710	22,935	37.19%
Corrective Maintenance	Shallow Patching	Tons	302.54	424.22	169.21	78.63	26.11%
	Deep Patching	Tons	227.46	397.81	124.11	90.11	39.63%
	Premix Leveling	Tons	70.54	88.40	52.59	10.89	15.63%

8.4 Formulation of Strategies for Each Pavement Family

As indicated in Chapters 3 and 5, a thorough cost-effectiveness evaluation of long-term maintenance is best carried out on the basis of strategies, not just treatments. A strategies consist of none, one, or multiple preventive maintenance treatments, each applied at its unique criterion that may be expressed in terms of frequency of usage (e.g., every 3 years) or condition triggers, e.g., anytime PSI fall below 3.5 units. Unlike preventive maintenance, corrective maintenance is generally not programmable, and therefore cannot be an option in each strategy. Therefore, each overall maintenance “scenario” consists of a strategy (preventive maintenance treatments), and by default, corrective maintenance treatments that are carried out periodically but whose level are functions of the amount of preventive maintenance treatments administered at a previous period. Lower levels of total preventive maintenance, over a given period of time (say, three years), translate to higher levels of total corrective maintenance within that time period, and vice versa. Given a certain combination of preventive maintenance treatments in a strategy, the 3-year cost of preventive maintenance was computed, and the corresponding total cost of corrective maintenance was determined using an approximate trade-off relation developed for 3-year corrective maintenance and 3-year preventive maintenance.

Tables 8-7 to 8-12 provide details (application criteria) of each strategy that was formulated for each pavement family. For each pavement family, strategies were formulated to reflect average geographical condition, therefore the effect of weather was not considered, but this could be addressed in a future study Each strategy consists of preventive maintenance activities that are typically carried out to retard deterioration or to prevent imminent deterioration, such as crack sealing or thin overlays, and also include maintenance activities that are of the preventive “remedial type (see Figure 1 in Chapter 1) such as undersealing and fault grinding, and corrective maintenance activities such as shallow and deep patching. Such maintenance activities are termed “default” activities as they are carried out to address distresses that are generally bound to occur, regardless of strategy. It is assumed that such “default” treatments are carried out on cycles of 3-year duration.

Table 8-7: Formulated Strategies for Rigid Interstate Pavements

Strategy #	Overall Maintenance Scenario			Remarks	
	Details of Strategy (Preventive Maintenance Elements)				Default Actions: Corrective Maintenance Elements (As needed, but 3-year Intervals is assumed)
	Crack Sealing	Joint Sealing	Underdrain Maintenance		
0	-	-	-	-	No maintenance.
1	-	-	-	Shallow Patching, Deep Patching Premix Leveling, Fault Grinding, Undersealing, Stitching	No preventive maintenance.
2	FA: 6 Yrs FT: 6 Yrs	FA: 6 Yrs FT: 6 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
3	FA: 5 Yrs FT: 5 Yrs	FA: 5 Yrs FT: 5 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
4	FA: 4 Yrs FT: 4 Yrs	FA: 4 Yrs FT: 4 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
5	FA: 3 Yrs FT: 3 Yrs	FA: 3 Yrs FT: 3 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
6	FA: 2 Yrs FT: 2 Yrs	FA: 2 Yrs FT: 2 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
7	FA: 1 Yrs FT: 1 Yrs	FA: 1 Yrs FT: 1 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
8	FA: 7 Yrs FT: 7 Yrs	FA: 7 Yrs FT: 7 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
9	FA: 8 Yrs FT: 8 Yrs	FA: 8 Yrs FT: 8 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
10	FA: 9 Yrs FT: 9 Yrs	FA: 9 Yrs FT: 9 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
11	FA:10 Yrs FT: 10 Yrs	FA: 10 Yrs FT: 10 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
12	FA:11 Yrs FT: 11 Yrs	FA: 11 Yrs FT: 11 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	

FA: Age of first application. FT: Frequency thereafter (after first application).
PM-Preventive Maintenance.

Table 8-8: Formulated Strategies for Rigid Non-Interstate Pavements

Strategy #	Overall Maintenance Scenario			Remarks	
	Details of Strategy (Preventive Maintenance Elements)				Default Actions: Corrective Maintenance Elements (As needed, but 3-year Intervals is assumed)
	Crack Sealing	Joint Sealing	Underdrain Maintenance		
0	-	-	-	-	No maintenance.
1	-	-	-	Shallow Patching, Deep Patching Premix Leveling, Fault Grinding, Undersealing, Stitching	No preventive maintenance.
2	FA: 6 Yrs FT: 6 Yrs	FA: 6 Yrs FT: 6 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
3	FA: 5 Yrs FT: 5 Yrs	FA: 5 Yrs FT: 5 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
4	FA: 4 Yrs FT: 4 Yrs	FA: 4 Yrs FT: 4 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
5	FA: 3 Yrs FT: 3 Yrs	FA: 3 Yrs FT: 3 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
6	FA: 2 Yrs FT: 2 Yrs	FA: 2 Yrs FT: 2 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
7	FA: 1 Yrs FT: 1 Yrs	FA: 1 Yrs FT: 1 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
8	FA: 7 Yrs FT: 7 Yrs	FA: 7 Yrs FT: 7 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
9	FA: 8 Yrs FT: 8 Yrs	FA: 8 Yrs FT: 8 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
10	FA: 9 Yrs FT: 9 Yrs	FA: 9 Yrs FT: 9 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
11	FA:10 Yrs FT: 10 Yrs	FA: 10 Yrs FT: 10 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	
12	FA:11 Yrs FT: 11 Yrs	FA: 11 Yrs FT: 11 Yrs	FA: 1 Yr FT: 1 Yr	Same as above	

FA: Age of first application. FT: Frequency thereafter (after first application).
PM-Preventive Maintenance

Table 8-9: Formulated Strategies for Overlay Interstate Pavements

Strategy #	Overall Maintenance Scenario				
	Details of Strategy (Preventive Maintenance Elements)				Default Actions: Corrective Maintenance Elements (As needed, but 3-year Intervals is assumed)
	Thin HMA Overlay	Micro-surfacing	Crack Sealing	Underdrain Maintenance	
0	-	-	-	-	-
1	-	-	-	-	Shallow Patching, Deep Patching, Premix Leveling Bump Planing
2	-	-	FA: 6 Yrs FT: 6 Yrs	Every Year	Same as above
3	-	-	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
4	-	FA: 6 Yrs FT: 6 Yrs	-	Every Year	Same as above
5	-	FA: 6 Yrs FT: 6 Yrs	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
6	FA: 6 Yrs FT: 6 Yrs	-	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
7	-	FA: 3 Yrs FT: 3 Yrs	-	Every Year	Same as above
8	-	FA: 4 Yrs FT: 4 Yrs	-	Every Year	Same as above
9	-	FA: 8 Yrs FT: 8 Yrs	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
10	FA: 9 Yrs FT: 9 Yrs	-	6 years after rehab. or micro-surf.	3 years after thin overlay	Same as above
11	FA: 8 Yrs FT: 6 Yrs	-	3 years after rehab. or thin overlay.	Every year	Same as above
12	FA: 5 Yrs FT: 5 Yrs	-	3 years after rehab. or micro-surf.	1 years after micro-surf.	Same as above

FA: Age of first application. FT: Frequency thereafter (after first application).

PM-Preventive Maintenance

Micro-surf- Micro-surfacing. Rehab.- Rehabilitation

Table 8-10: Formulated Strategies for High Volume Overlay Non-Interstate Pavements (AADT >2500)

Strategy #	Overall Maintenance Scenario				
	Details of Strategy (Preventive Maintenance Elements)				Default Actions: Corrective Maintenance Elements (As needed, but 3-year Intervals is assumed)
	Thin HMA Overlay	Micro-surfacing	Crack Sealing	Underdrain Maintenance	
0	-	-	-	-	-
1	-	-	-	-	Shallow Patching, Deep Patching, Premix Leveling Bump Planing
2	-	-	FA: 6 Yrs FT: 6 Yrs	Every Year	Same as above
3	-	-	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
4	-	FA: 6 Yrs FT: 6 Yrs	-	Every Year	Same as above
5	-	FA: 6 Yrs FT: 6 Yrs	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
6	FA: 6 Yrs FT: 6 Yrs	-	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
7	-	FA: 3 Yrs FT: 3 Yrs	-	Every Year	Same as above
8	-	FA: 4 Yrs FT: 4 Yrs	-	Every Year	Same as above
9	-	FA: 8 Yrs FT: 8 Yrs	FA: 3 Yrs FT: 3 Yrs	Every Year	Same as above
10	FA: 9 Yrs FT: 9 Yrs	-	6 years after rehab. or micro-surf.	3 years after micro-surf. or chip sealing	Same as above
11	FA: 8 Yrs FT: 8 Yrs	-	6 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
12	FA: 5 Yrs FT: 5 Yrs	-	3 years after rehab. or micro-surf.	1 years after micro-surf. or chip sealing	Same as above

FA: Age of first application. FT: Frequency thereafter (after first application).

PM-Preventive Maintenance

Micro-surf- Micro-surfacing. Rehab.- Rehabilitation

Table 8-11: Formulated Strategies for Low Volume Overlay Non-Interstate Pavements (AADT <2500)

Strategy #	Overall Maintenance Scenario				
	Details of Strategy (Preventive Maintenance Elements)				Default Actions: Corrective Maintenance Elements (As needed, but 3-year Intervals is assumed)
	Thin HMA Overlay	Micro-surfacing	Chip Sealing	Crack Sealing	
0	-	-	-	-	-
1	-	-	-	-	Shallow Patching, Deep Patching, Premix Leveling Bump Planing
2	-	-	-	FA: 6 Yrs FT: 3 Yrs	Same as above
3	-	-	-	FA: 3 Yrs FT: 3 Yrs	Same as above
4	-	-	FA: 6 Yrs FT: 6 Yrs	-	Same as above
5	-	-	FA: 6 Yrs FT: 6 Yrs	FA: 3 Yrs FT: 3 Yrs	Same as above
6	FA: 6 Yrs FT: 6 Yrs	-	-	FA: 3 Yrs FT: 3 Yrs	Same as above
7	-	FA: 5 Yrs FT: 5 Yrs	2 years after micro-surf.	1 year after chip sealing	Same as above
8	-	FA: 8 Yrs FT: 8 Yrs	4 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
9	FA: 8 Yrs FT: 8 Yrs	-	4 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
10	FA: 9 Yrs FT: 9 Yrs	-	6 years after rehab. or micro-surf.	3 years after micro-surf. or chip sealing	Same as above
11	FA: 8 Yrs FT: 8 Yrs	-	6 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
12	FA: 5 Yrs FT: 5 Yrs	-	3 years after rehab. or micro-surf.	1 years after micro-surf. or chip sealing	Same as above

FA: Age of first application. FT: Frequency thereafter (after first application).

PM-Preventive Maintenance

Micro-surf- Micro-surfacing. Rehab.- Rehabilitation

Table 8-12: Formulated Strategies for Full-depth Asphalt Pavements (Non-Interstates Only)

Strategy #	Overall Maintenance Scenario				
	Details of Strategy (Preventive Maintenance Elements)				Default Actions: Corrective Maintenance Elements (As needed, but 3-year Intervals is assumed)
	Thin HMA Overlay	Micro-surfacing	Chip Sealing	Crack Sealing	
0	-	-	-	-	-
1	-	-	-	-	Shallow Patching, Deep Patching, Premix Leveling Bump Planing
2	-	-	-	FA: 6 Yrs FT: 3 Yrs	Same as above
3	-	-	-	FA: 3 Yrs FT: 3 Yrs	Same as above
4	-	-	FA: 6 Yrs FT: 6 Yrs	-	Same as above
5	-	-	FA: 6 Yrs FT: 6 Yrs	FA: 3 Yrs FT: 3 Yrs	Same as above
6	FA: 6 Yrs FT: 6 Yrs	-	-	FA: 3 Yrs FT: 3 Yrs	Same as above
7	-	FA: 5 Yrs FT: 5 Yrs	2 years after rehab. or micro-surf.	1 year after micro-surf. or chip sealing	Same as above
8	-	FA: 8 Yrs FT: 8 Yrs	4 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
9	FA: 8 Yrs FT: 8 Yrs	-	4 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
10	FA: 9 Yrs FT: 9 Yrs	-	6 years after rehab. or micro-surf.	3 years after micro-surf. or chip sealing	Same as above
11	FA: 8 Yrs FT: 8 Yrs	-	6 years after rehab. or micro-surf.	2 years after micro-surf. or chip sealing	Same as above
12	FA: 5 Yrs FT: 5 Yrs	-	3 years after rehab. or micro-surf.	1 years after micro-surf. or chip sealing	Same as above

FA: Age of first application. FT: Frequency thereafter (after first application).

PM-Preventive Maintenance

Micro-surf- Micro-surfacing. Rehab.- Rehabilitation

8.5 Cost-effectiveness Analyses for Each Strategy and Pavement Family

As explained in Chapter 5, an important assumption made in the present study is that the costs of different treatments carried out in the same year are independent of each other, so the total cost is simply the arithmetic sum of the cost of the individual treatments. The basic cost of each maintenance treatment type is obtained using the cost models developed in Section 8.3, while details of methods and data used in calculating the delay and safety costs are presented in Chapter 6 and Appendix H, respectively. All dollar amounts are expressed in 1995 values, and expenditures were brought to their present worth (at Year 0) to account for the fact that different strategies have treatments carried out different points in time, and that the time value of money needs to be considered due to opportunity cost of maintenance investments at any point in time.

The life cycle effectiveness (benefits) of each strategy was represented by the increase in the area of the pavement performance curve due to the various jumps in the curve at points indicating application of maintenance treatments. The cost-effectiveness of each strategy was then computed as the incremental cost-effectiveness ratio of that strategy relative to that of the zero-maintenance strategy.

In the section below, the cost-effectiveness of each of several maintenance strategies formulated for each pavement family, are discussed. This includes a presentation of the following values that are associated with each strategy: life cycle costs of preventive and corrective maintenance, the effectiveness of each strategy in terms of PSI-years, and the cost-effectiveness.

8.5.1 Rigid Interstates

Approximately 20% of all Interstate pavements in the state had a rigid surface at a typical year within the study period. The percentage of rigid Interstate pavements declined between the years 1990 and 1998, but increased slightly after 1998. Most rigid Interstate pavements are jointed: some are plain, while others are reinforced. There are very few continuously reinforced concrete pavements left in the state. [Table 8-13](#) provides the costs and benefits associated with various strategies formulated for maintenance of rigid Interstate pavements. The details of each strategy are provided in the preceding section.

Table 8-13: Cost-effectiveness for Various Strategies, Rigid Interstates

	Life Cycle Preventive Maintenance (\$)	Life Cycle Corrective Maintenance (\$)	Total Life Cycle Maintenance (\$)	Effectiveness (Incremental Area under Performance Curve) in PSI-Yrs	Cost-effectiveness (PSI-Years per \$10⁶)
0	\$0.00	\$0.00	\$0.00	0.00	1.00
1	\$0.00	\$41,000	\$41,000	0.21	5.16
2	\$25,000	\$30,900	\$55,900	0.31	5.54
3	\$37,800	\$23,300	\$61,100	0.47	7.69
4	\$48,800	\$17,600	\$66,400	0.41	6.13
5	\$79,900	\$4,200	\$84,100	0.67	7.91
6	\$107,700	\$4,400	\$112,200	0.83	7.36
7	\$225,200	\$4,200	\$229,400	1.79	7.78
8	\$44,900	\$4,800	\$49,700	0.40	7.99
9	\$32,500	\$14,100	\$46,700	0.39	8.32
10	\$19,200	\$28,200	\$47,400	0.31	6.53
11	\$15,100	\$31,400	\$46,500	0.37	7.99
12	\$14,900	\$31,100	\$46,000	0.37	8.08

Figure 84 shows the cost-effectiveness of each maintenance strategy for this family of pavements. From Table 8-13 and Figure 8-1, it is clear that the most cost-effective of the strategies considered for rigid Interstates is Strategy 9, the details of which are as follows:

- Cracks and joint sealing 8 years after reconstruction, and every 8 years thereafter,
- Underdrain maintenance every year,
- Default (typically corrective) activities to be carried out as needed:
 - Shallow Patching, Deep Patching, Premix Leveling, Fault Grinding, Undersealing, and Stitching.

This strategy is associated with a present worth total life cycle maintenance expenditure of \$47,000 per lane mile, in terms of 1995 dollars. Of this amount, \$33,000 is allotted to preventive maintenance treatments, such as crack and joint sealing, and underdrain maintenance, while \$14,000 is for the corrective maintenance activities that will be used to address inevitable pavement distresses such as patch deterioration, base failures, faulting, and development of voids under the concrete slab.

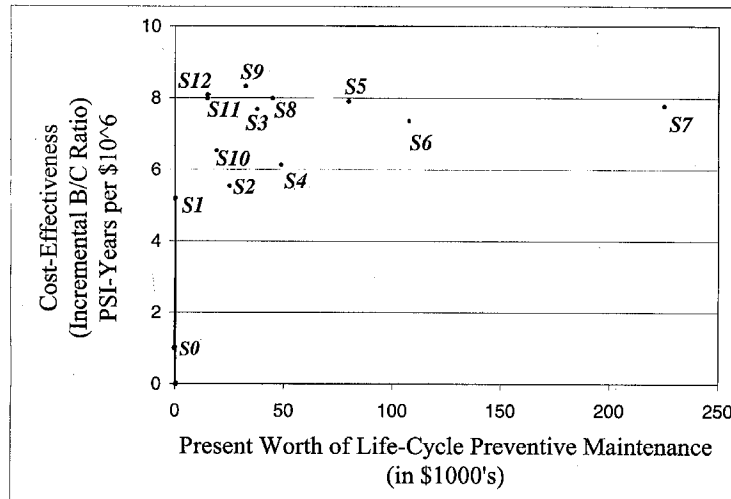


Figure 8-1: Cost-effectiveness of each Maintenance Strategy, Rigid Interstates

8.5.2 Rigid Non-Interstates

There are relatively few rigid non-Interstate pavements in the state, but these play a vital role in the overall transportation network. Examples include State Road 37 in Monroe and Johnson Counties, which link Indiana University's Bloomington Campus to Indianapolis, and State Road 3 By-Pass in Muncie, Delaware County. Table 8-14 provides the costs and benefits associated with various strategies that were formulated to examine the impact (in terms of cost-effectiveness) of preventive maintenance types and levels for this family of pavements. The details of each strategy are provided in the preceding section.

Table 8-14: Cost-effectiveness of Various Strategies, Rigid Non-Interstates

	Life Cycle Preventive Maintenance (\$)	Life Cycle Corrective Maintenance (\$)	Total Life Cycle Maintenance (\$)	Effectiveness (Incremental Area under Performance Curve) in PSI -Yrs	Cost-effectiveness (PSI-Yrs per 10 ⁶ constant dollar)
0	\$0.00	\$0.00	\$0.00	0.00	1.00
1	\$0	\$41,000	\$41,000	0.21	5.16
2	\$11,300	\$29,000	\$40,300	0.31	7.69
3	\$18,200	\$23,200	\$41,400	0.47	11.34
4	\$23,700	\$17,800	\$41,500	0.41	9.84
5	\$39,200	\$4,500	\$43,700	0.67	15.26
6	\$52,400	\$4,700	\$57,100	0.83	14.46
7	\$109,500	\$4,200	\$113,800	1.79	15.68
8	\$17,600	\$6,700	\$24,300	0.41	16.73
9	\$12,300	\$14,800	\$27,200	0.39	14.49
10	\$7,700	\$29,800	\$37,600	0.31	8.28
11	\$6,100	\$33,000	\$39,100	0.37	9.51
12	\$6,100	\$33,000	\$39,100	0.37	9.50

Figure 8-2 shows the cost-effectiveness of each maintenance strategy for this family of pavements. It is apparent from this figure that of the strategies considered for rigid non-Interstate, the most cost-effective is Strategy 8, the details of which are as follows:

- Cracks and joint sealing 7 years after reconstruction, and every 7 years thereafter,
- Underdrain maintenance every year,
- Default (typically corrective) activities to be carried out as needed:
 - Shallow Patching, Deep Patching, Premix Leveling, Fault Grinding, Undersealing, and Stitching.

This strategy is associated with a total life cycle maintenance expenditure (present worth) of \$25,000 per lane-mile, in terms of 1995 dollars. Of this amount, \$18,000 is allotted to preventive maintenance treatments such as crack and joint maintenance, and underdrain cleaning and inspection, while \$7,000 is for the corrective maintenance activities that will be used to address pavement distresses such as patch deterioration, base failures, faulting, and sub-surface void development.

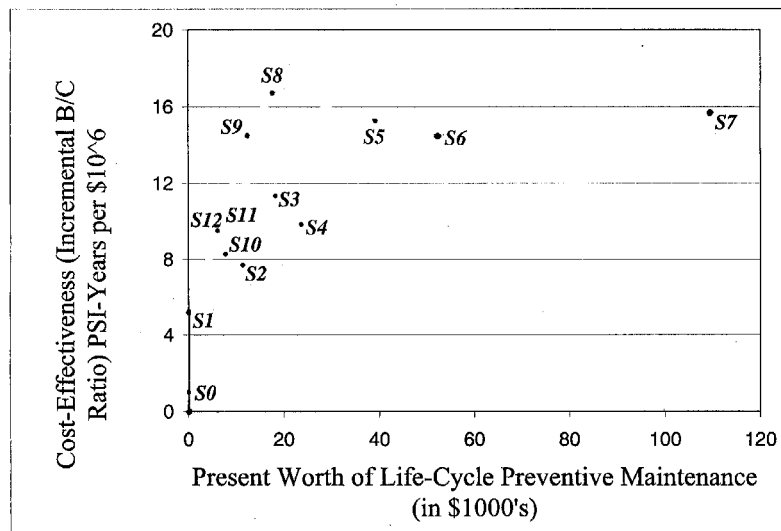


Figure 8-2: Cost-effectiveness of each Maintenance Strategy, Rigid Non-Interstates

8.5.3 AC-over-PCC Composite Interstate Pavements

AC-over-PCC composite pavements are probably the most important family of pavements in the state, as they consist of over 80% of the entire Interstate system. As indicated in Chapter 2, the past decade was characterized by an upsurge in both mileage and percentage of asphaltic concrete AC-over-PCC composite pavements. While traditional AC-over-PCC composites (where no mechanical treatment of the existing slab is carried out) is most common, there have been attempts at rubblization or crack-and-sealing the existing concrete before applying the AC AC-over-PCC composite. Table 8-15 provides the costs and benefits associated with various strategies that were formulated for maintenance of AC-over-PCC composite Interstate pavements. The details of each strategy are provided in the preceding section.

Table 8-15: Cost-effectiveness of Various Strategies, AC-over-PCC Composite Interstate Pavements

	Life Cycle Preventive Maintenance (\$)	Life Cycle Corrective Maintenance (\$)	Total Life Cycle Maintenance (\$)	Effectiveness (Incremental Area under Performance Curve) in PSI-Yrs	Cost- effectiveness (PSI-Yrs per 10 ⁶ constant dollar)
0	\$0.00	\$0.00	\$0.00	\$0.00	1.00
1	\$0.00	\$41,000	\$41,000	0.35	5.16
2	\$4,700	\$28,000	\$32,800	0.38	6.80
3	\$15,100	\$6,800	\$22,000	0.41	11.14
4	\$4,700	\$28,000	\$32,800	2.43	27.51
5	\$15,100	\$6,800	\$22,000	2.46	41.95
6	\$71,300	\$6,000	\$77,300	3.76	24.86
7	\$91,600	\$5,700	\$84,600	7.16	50.23
8	\$43,500	\$4,700	\$48,300	4.73	47.37
9	\$12,500	\$15,100	\$27,700	1.42	35.54
10	\$64,600	\$5,400	\$70,100	4.14	37.11
11	\$79,000	\$5,000	\$84,000	7.06	52.23
12	\$130,800	\$4,700	\$135,500	9.01	39.17

Table 8-15 indicates that of the strategies considered, the most cost-effective strategy for AC-over-PCC composite Interstates was Strategy 11, the details of which are as follows:

- Thin AC-over-PCC composite 8 years after reconstruction/ rehabilitation and every six years thereafter,
- Underdrain maintenance every year after reconstruction/resurfacing/thin AC-over-PCC composite,
- Default (typically corrective) activities to be carried out on a three-year cycle:
 - Shallow Patching, Deep Patching, Joint/Bump Repair.

This strategy is associated with a total life cycle maintenance expenditure (present worth) of \$84,000 per lane mile, in terms of 1995 dollars. The strategy specifies that \$79,000 of this amount should be allotted to preventive maintenance treatments, while \$5,000 should be used for corrective maintenance activities to address inevitable pavement distresses such as patch deterioration and bumps and other surface defects typically associated with AC-over-PCC AC-over-PCC composite pavements.

8.5.4 High-volume Non-Interstate AC-over-PCC Composite Pavements

AC-over-PCC composite non-Interstate pavements are categorized as high volume or low volume for the present study because certain treatments typically vary by functional class. For example, seal coating is typically not applied to high volume roads due to the hazards posed by flying chips. Many high volume non-Interstate highways are U.S. roads and State Roads on the National Highway System, and are principal arterials, major arterials, or other freeways and expressways. [Table 8-16](#) provides the costs and benefits associated with various maintenance strategies that were formulated for such pavements. The details of each strategy are provided in the preceding section. All dollar amounts are expressed in 1995 values.

Table 8-16: Cost-effectiveness of Various Strategies, Non-Interstate High-volume AC-over-PCC Composite Pavements,

	Life Cycle Preventive Maintenance (\$)	Life Cycle Corrective Maintenance (\$)	Total Life Cycle Maintenance (\$)	Effectiveness (Incremental Area under Performance Curve) in PSI -Yrs	Cost-effectiveness (PSI-Yrs per 10⁶ constant dollar)
0	\$0.00	\$0.00	\$0.00	0.00	1.00
1	\$0.00	\$41,000	\$41,000	0.28	5.16
2	\$1,900	\$28,000	\$30,000	0.32	7.48
3	\$6,300	\$11,000	\$17,000	0.36	15.40
4	\$54,100	\$33,000	\$87,200	2.36	14.33
5	\$62,900	\$10,800	\$73,700	2.39	17.43
6	\$66,400	\$8,700	\$75,200	3.69	25.74
7	\$173,100	\$4,200	\$177,300	7.06	26.53
8	\$96,200	\$6,500	\$102,700	3.55	23.79
9	\$39,900	\$14,600	\$54,500	1.36	24.53
10	\$103,300	\$6,300	\$109,600	3.04	26.95
11	\$118,600	\$6,900	\$125,500	4.20	24.65
12	\$199,300	\$4,200	\$203,600	7.18	29.49

The cost-effectiveness of each maintenance strategy for this family of pavements is illustrated as Figure 8-3. From Table 8-16 and Figure 8-3, it is obvious that the most cost-effective of the strategies considered for high volume AC-over-PCC composite non-Interstates is Strategy 12, the details of which are as follows:

- Thin AC-over-PCC composite every 5 years after resurfacing,
- Crack sealing every 3 years after reconstruction/resurfacing/thin overlay,
- Underdrain maintenance every year,
- Default (typically corrective) activities to be carried out on a three-year cycle:

- Shallow Patching, Deep Patching, Joint/Bump Repair.

This strategy is associated with a present worth total life cycle maintenance expenditure of \$204,000 per lane-mile, in terms of 1995 dollars. Of this amount, it is estimated that \$200,000 be allotted to preventive maintenance treatments, while \$4,000 should be used for corrective maintenance activities such as patching and bump repair.

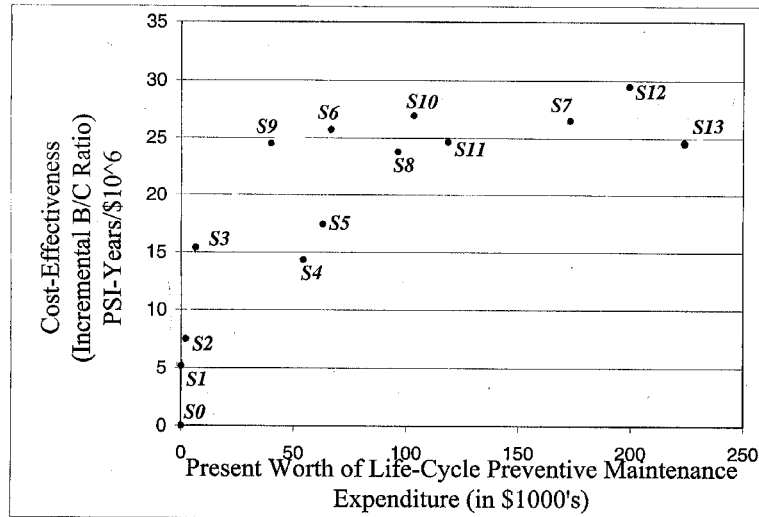


Figure 8-3: Cost-effectiveness of Each Maintenance Strategy, High-volume Non-Interstate AC-over-PCC Composite Pavements

8.5.5 Non-Interstate Low-volume AC-over-PCC Composite Pavements

Low-volume Non-Interstate AC-over-PCC composite pavements are typically State Roads and US Roads that are either minor arterials or collectors. Because of their relatively low daily traffic volumes (less than 2,500 ADT), the use of chip seals is typically considered as a viable preventive maintenance treatment for this category of pavements. The costs and benefits associated with various strategies that may be used for maintenance of such pavements are provided as [Table 8-17](#). The details of each strategy are provided in the preceding section.

Table 8-17: Cost-effectiveness for Various Strategies, Low-volume Non-Interstate AC-over-PCC Composite Pavements

	Life Cycle Preventive Maintenance (\$)	Life Cycle Corrective Maintenance (\$)	Total Life Cycle Maintenance (\$)	Effectiveness (Incremental Area under Performance Curve) in PSI -Yrs	Cost-effectiveness (PSI-Yrs per 10⁶ constant dollar)
0	\$0.00	\$0.00	\$0.00	\$0.00	1.00
1	\$0.00	\$26,000	\$26,000	0.21	8.14
2	\$1,600	\$14,600	\$16,300	0.26	16.02
3	\$2,400	\$9,200	\$11,700	0.29	24.36
4	\$7,300	\$18,400	\$25,700	0.90	34.98
5	\$9,900	\$8,000	\$17,900	0.95	52.90
6	\$64,990	\$9,300	\$74,300	1.95	26.22
7	\$44,200	\$3,000	\$47,300	4.01	84.86
8	\$29,700	\$4,700	\$34,500	2.10	60.93
9	\$8,100	\$11,600	\$19,700	0.99	50.37
10	\$42,300	\$3,900	\$46,200	2.61	56.55
11	\$48,700	\$4,300	\$53,100	2.75	51.86
12	\$91,900	\$2,600	\$94,500	5.31	56.14

The relative cost-effectiveness of the various maintenance strategies formulated for this family of pavements is shown as [Figure 8-5](#). [This figure and Table 8-5](#) indicate that the most cost-effective of the strategies considered for low volume Non-Interstate AC-over-PCC composite pavements is Strategy 7, the details of which are as follows:

- Micro-surfacing every 8 years after resurfacing or reconstruction,
- Chip sealing 4 years after micro-surfacing,
- Crack sealing 2 years after micro-surfacing,

- Default (typically corrective) activities to be carried out as needed:
 - Shallow Patching, Deep Patching, Joint/Bump Repair.

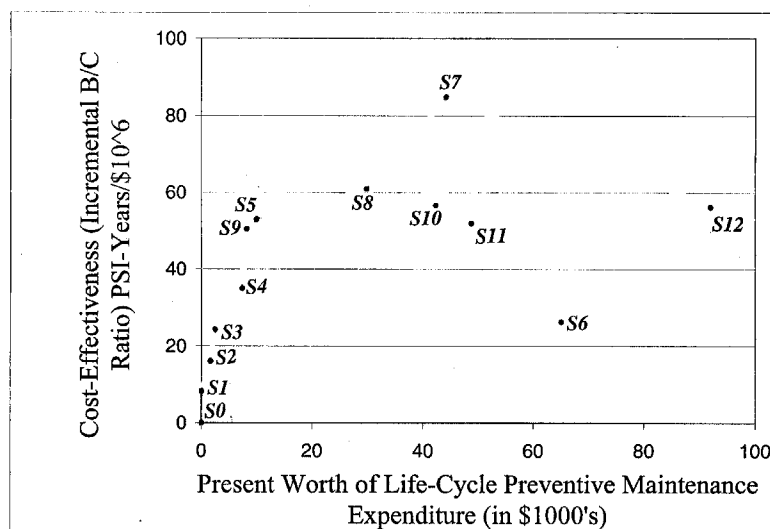


Figure 8-4: Cost-effectiveness of each Maintenance Strategy, Low-volume AC-over-PCC composite Non-Interstates

The most cost-effective strategy for low-volume AC-over-PCC composite non-Interstates pavements is associated with a total life cycle maintenance expenditure (present worth) of \$47,000 per lane mile, in terms of 1995 dollars. The strategy specifies that \$43,000 of this amount should be allotted to preventive maintenance treatments, i.e., crack sealing and chip sealing, while \$3,000 should be used for corrective maintenance activities to address pavement distresses such as patch deterioration, base failure, and bumps, for each lane-mile of pavement.

8.5.6 Full-depth Asphaltic Concrete Non-Interstates

The most common type of pavements in the state, full-depth asphaltic concrete pavements constitute over 60% of the entire state road network (and 30-40% of the state highway network). While there are very few full-depth asphaltic concrete pavements on the Interstate or the National Highway

System, the percentage of full-depth asphalt pavements in the state has remained relatively steady over the years. Most of such pavements belong to the Minor Collectors functional class and other classes that are associated with relatively low traffic volumes and relatively inferior design and construction standards. [Table 8-18](#) provides the costs and benefits associated with various strategies that may be used for maintenance of such pavements. The details of each strategy are provided in the preceding section.

[Table 8-18](#): Cost-effectiveness for Various Strategies, Full-depth Asphalt Non-Interstates

	Life Cycle Corrective Maintenance (\$)	Life Cycle Corrective Maintenance (\$)	Total Life Cycle Maintenance (\$)	Effectiveness (Incremental Area under Performance Curve) in PSI-Yrs	Cost-effectiveness (PSI-Yrs per 10⁶ constant dollar)
0	\$0.00	\$0.00	\$0.00	0.00	1.00
1	\$0.00	\$21,100	\$21,100	0.21	10.03
2	\$1,800	\$12,000	\$13,800	0.25	18.42
3	\$2,500	\$6,600	\$9,000	0.28	30.35
4	\$2,600	\$15,300	\$17,900	0.90	50.40
5	\$4,300	\$5,600	\$9,800	0.95	96.04
6	\$34,100	\$5,190	\$39,300	1.94	49.41
7	\$39,200	\$2,900	\$42,100	4.01	96.27
8	\$20,600	\$3,800	\$24,400	2.10	85.96
9	\$4,400	\$10,800	\$15,300	0.99	65.02
10	\$31,500	\$3,680	\$35,200	2.61	74.17
11	\$34,800	\$3,700	\$38,600	2.75	71.30
12	\$103,900	\$4,300	\$108,200	5.31	49.07

Figure 8-6 illustrates the various cost-effectiveness of the maintenance strategies formulated for this family of pavements. [Table 8-18](#) and Figure 8-6 show that of the strategies considered, the most cost-effective for full-depth asphaltic concrete pavements is Strategy 7, the details of which are as follows:

- Chip sealing every 6 years after resurfacing or reconstruction,
- Crack sealing every 3 years after reconstruction, resurfacing, or chip sealing,
- Default (typically corrective) activities to be carried out as needed:
 - Shallow Patching, Deep Patching, Joint/Bump Repair.

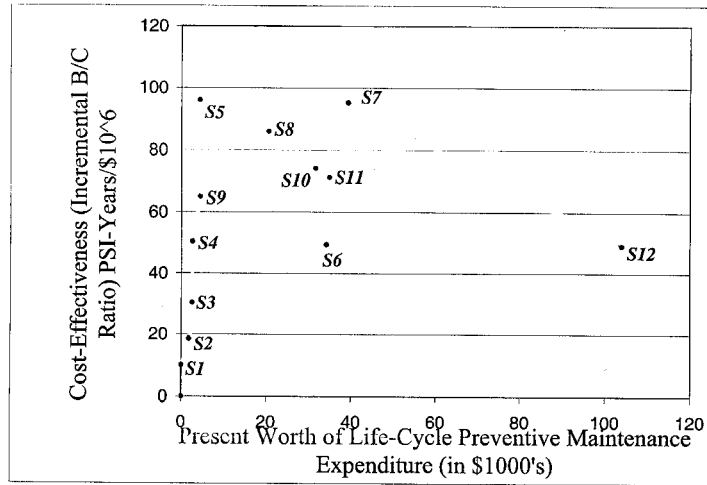


Figure 8-5: Cost-effectiveness of Each Maintenance Strategy, Full-depth Asphalt Non-Interstates

The “optimal” strategy for full-depth asphaltic concrete pavements is associated with a present worth value of a total life cycle maintenance expenditure of \$42,000 per lane mile, in terms of 1995 dollars. The strategy specifies that \$39,000 of this amount should be allotted to preventive maintenance treatments, i.e., crack sealing and chip sealing, while \$3,000 should be used for corrective maintenance activities to address pavement distresses such as patch deterioration, base failure, and bumps, for each lane-mile of pavement. It can be seen that unlike rigid or overlay pavements, the “optimal” strategy for full-depth asphalt pavements is associated with lower fraction of preventive maintenance compared to corrective maintenance, signifying that the role of preventive maintenance is relatively more crucial for rigid and overlay pavements compared to full-depth asphalts.

8.6 Life-cycle Cost-effectiveness and Preventive Maintenance Models for Each Pavement Family

8.6.1 LCC Effectiveness and Preventive Maintenance Models

Using data points generated by the estimated costs and benefits of each strategy in the previous section, models were developed for each pavement family to reflect the relationship between levels of preventive maintenance effort (costs per lane-mile in constant dollar) and corresponding cost effectiveness, over pavement life-cycle. Non-linear statistical techniques were used to estimate equations from which the cost-effectiveness of any level of preventive maintenance can be estimated.

The developed models suggest that cost-effectiveness of preventive maintenance generally increases with increasing preventive maintenance effort, up to a certain maximum, after which it declines with increasing effort. The rate of post-optimum decline of cost-effectiveness is generally slower than the rate of its pre-optimum increase. It was found that the curves generally fit the following functional form:

$$Y = b * X - a(X^c) \dots\dots\dots (120)$$

Where

a, b, c are constants that control the shape of the cost effectiveness curve,

Y is the cost effectiveness of preventive maintenance,

X is the preventive maintenance effort per lane mile, expressed in dollar value.

The estimated model coefficients for each pavement family are shown in Table 8-19.

A plot of the developed cost-effectiveness models, using fitted values, is provided as Figure 8-7. The results generally show that the optimal level of preventive maintenance effort (expenditure) and the corresponding maximum cost effectiveness can be determined theoretically from the developed curves. Similarly, the impacts (cost-effectiveness) of any given level of preventive maintenance effort can be determined.

Table 8-19: Estimated Coefficients of Cost Effectiveness Models

Pavement Family	Model Coefficients			Adjusted R ²	Description of Pavement Family
	<i>a</i> (*10 ⁵)	<i>b</i> (*10 ⁵)	<i>c</i>		
I	0.799909	0.800011	1.000115	0.21	Rigid (PCC) Interstates
II	2.216722	2.216876	1.000136	0.44	Rigid (PCC) Non Interstates
III	5.012173	5.012574	1.000124	0.51	Overlay (PCC-over-AC) Interstates
IV	1.229855	1.230115	1.000150	0.70	Overlay (PCC-over-AC) High-volume Non Interstates
V	2.298091	2.297424	1.001043	0.94	Overlay (PCC-over-AC) Low-volume Non Interstates
VI	20.218605	20.219048	1.000097	0.22	Full-depth AC Non Interstates

It is seen that for rigid Interstate pavements for instance, a maximum cost effectiveness of approximately 10 PSI-Years per \$million is attained at a preventive maintenance effort of \$1,200 per lane mile, over the life cycle of such pavements. At the other extreme end of the spectrum is the case for full-depth asphaltic concrete pavements, for which a remarkably high cost-effectiveness (90 PSI-Years per \$million) is attained at relatively low effort (\$400/lane-mile) over its life-cycle. The figure therefore suggests that rigid Interstate pavements are least resilient to changes in preventive maintenance, while full-depth AC and low-volume overlay non-Interstate pavements are least resilient (most sensitive to an increase or decrease in preventive maintenance effort).

This might seem to imply that the latter class of pavements currently generally bears heavier loads or/and are more susceptible to weather effects (by virtue of their material, design and construction standards) than they were designed for, compared to the former, and therefore are more “receptive” to the benefits offered by preventive maintenance.

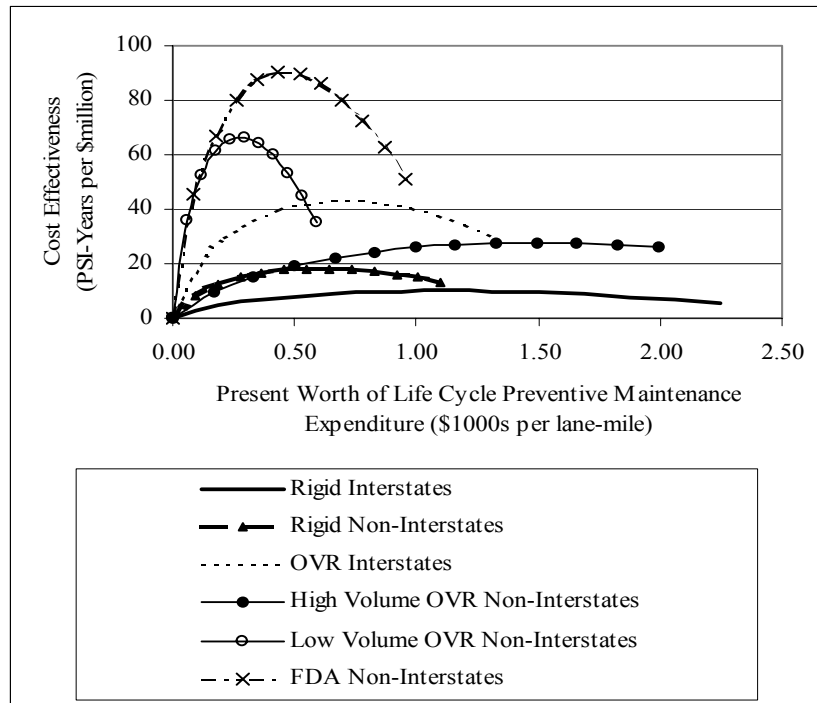


Figure 8-6: Comparison of Preventive Maintenance Cost Effectiveness Trends

In furtherance to the observed increase in optimum level of preventive maintenance and a decrease in maximum cost-effectiveness as one moves from full-depth AC non Interstate pavements to rigid Interstate pavements (with the case for overlay pavements lying in between these two extremes), there appears to be implications regarding relative efficacy of preventive maintenance activities among various pavement families. Such maintenance efficacy appear directly linked to the relative pavement resilience that is discussed above. The results seem to imply that it is more cost-effective to carry out preventive maintenance on full-depth AC pavements and least cost-effective to carry out such maintenance on rigid pavements. In other words, for a given level of investment, the returns seem to be higher for full-depth AC pavements. Such lower resilience of AC pavements to maintenance to suggest that the ratio of traffic loading to pavement structural capacity (and weather effects to pavement material resilience to weather) are higher for full-depth AC pavements than they are for rigid pavements in Indiana. In other words, rate of AC pavement deterioration due to effects of load and weather seem to be reduced to a greater extent upon the

application of maintenance, compared to rigid pavements. A plausible reason could be the fact that typical preventive maintenance treatments on flexible pavements offer greater increase in pavement condition and longevity than typical treatments do for rigid pavements. Indeed, the range of typical treatments on AC pavements ranges from local repairs such as crack sealing to rehabilitation-like treatments such as thin HMA overlays. On the other hand, typical preventive maintenance treatments on rigid pavements are relatively low-level treatments such as joint sealing and crack sealing.

Preventive maintenance treatments on rigid pavement treatments such as under-sealing, diamond grinding, and stitching were not considered in the study due to lack of data. It is quite possible that inclusion of these treatments in rigid pavement strategies may yield a result that indicates higher levels of maximum cost effectiveness than that observed from the above results.

8.6.2 Marginal Effects of Life-Cycle Preventive Maintenance

Having obtained the trade-off function between life-cycle preventive maintenance effort and cost effectiveness of such efforts for each pavement family, the present study proceeded to investigate the marginal effects of life-cycle preventive maintenance cost-effectiveness to the effort (expenditure) invested in such activities. The concept of elasticity was used in this study, as shown below:

Elasticity (E) = % change in response variable / %change in explanatory variable

$$E_x = \frac{x * f'(x)}{f(x)} \dots\dots\dots(121)$$

Where

$f'(x)$ = derivative of the response variable $f(X)$ or Y , with respect to the explanatory variable (x).

The value of elasticity depends on the value of X . The functional form for preventive maintenance cost-effectiveness model is shown as Equation (3). The elasticity is given as follows:

$$\frac{\partial y / y}{\partial x / x} = \frac{x * f'(x)}{f(x)} = \frac{bx - acx^c}{bx - ax^c} \dots\dots\dots(122)$$

All symbols have their usual meanings.

The above expression makes it possible to determine the impact of a unit change in life-cycle preventive maintenance effort (expenditure per lane-mile) on the cost effectiveness of such efforts, at any existing level of expenditure. Figure 8-8 shows the marginal effects models for preventive maintenance cost effectiveness, by pavement family.

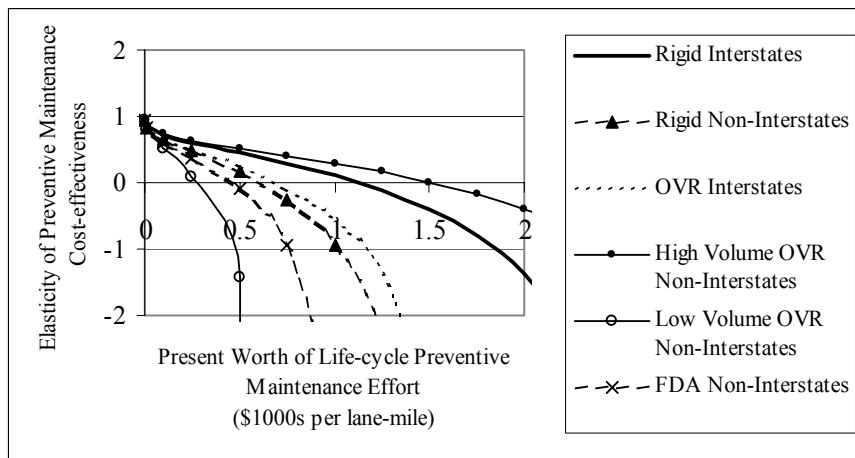


Figure 8-7: Elasticity of Preventive Maintenance Cost Effectiveness With Respect to Effort

A marked contrast in levels of elasticities is also observed between rigid Interstates (least sensitive) and full-depth AC (most sensitive). Also, rigid Interstates appear to be more sensitive to changes in preventive maintenance effort, compared to overlay Interstates. The general trend seems to be that rigid pavements have lower sensitivities to changes in preventive maintenance efforts compared to flexible pavements, particularly full-depth AC pavements. Also, it is generally seen that Interstate pavements are less sensitive than their non-Interstate counterparts. These patterns may be attributable to the differences in the ratio (or a function thereof) of material quality, and design and construction standards on one hand, to

their respective traffic and weather “experiences” on the other hand. It is suggested that existing differences in the ratio (or a function thereof) of such “strength” and “stress” attributes obviously differ between pavement surface types and also between functional classes.

8.6.3 Recommendations: Field Experiment for Life-Cycle Evaluation of Preventive Maintenance Cost Effectiveness

In order to shed more light on the cost-effectiveness evaluation of preventive maintenance strategies and not just treatments, field experiments could be carried out. This would involve implementation of several alternative maintenance treatment types and timings for each pre-defined pavement family. An advantage of this experiment would be the acquisition of directly observed data on effectiveness and cost, rather than resorting to the use of such data estimated from cost models and effectiveness models, as done in the present study. This effort would require careful monitoring of pavement condition over time, jumps in pavement condition in response to maintenance treatments, and costs of all preventive *and* corrective maintenance treatments associated with a given strategy. If a large number of strategies are implemented for each pavement family, it may be possible to have several data points from which more reliable statistical functions can be developed to explain the trade-off between preventive maintenance effort and its cost-effectiveness. Probably more importantly, such an experiment would result in the determination of a single optimal strategy (the best preventive maintenance treatment types and timings) associated with each pavement family.

8.6.4 Discussion

The models generally show that increasing levels of preventive maintenance is associated with increasing cost effectiveness, but only up to a point, after which increasing preventive maintenance leads to decreasing cost-effectiveness. Interstate pavements generally exhibited relatively low maximum cost-effectiveness achieved at relatively high unit preventive maintenance expenditure, compared to non-Interstate pavements. Also, rigid pavements were generally associated with low maximum cost-effectiveness that corresponds to relatively high unit preventive maintenance expenditure, compared to

their flexible counterparts. The marginal effects models showed that Interstate pavements and rigid pavements are generally associated with greater resilience (less sensitivity) to preventive maintenance, compared to non-Interstate and flexible pavements, respectively. Such lower resilience (higher sensitivities) of AC and Non-Interstate pavements to preventive maintenance, relative to their rigid and Interstate counterparts, respectively, seems to suggest that the ratio of traffic and weather experiences to pavement material and design quality or structural capacity are generally higher for full-depth AC pavements than they are for rigid pavements, and also higher for Non-Interstates compared to Interstates in Indiana. The models developed in the present study provide a guide for pavement managers to determine optimum funding levels for pavement preventive maintenance, and also to assess the impacts of any shortfall in preventive maintenance funding. Future studies in this direction could adopt an experimental field approach, and can help identify specific optimal preventive strategies (treatment types and timings) for each pavement family, such that overall cost effectiveness is maximized. As many highway agencies strive to establish or enhance their existing pavement management databases to include maintenance costs and effectiveness, studies similar to the present one can be carried out with local data to address the issues posed in the present study.

8.7 Chapter Summary

This chapter discussed results of the evaluation of maintenance over entire pavement life. This was done by developing performance curves, formulating strategies, determining the cost and benefits of each strategy. Effectiveness was measured as the area under the performance curve (the shape of which depends on the strategy in question). The cost of each strategy was found by adding up the costs of individual maintenance treatment that comprise the strategy. The cost-effectiveness of the strategy was found as the incremental benefit/cost ratio relative to the zero maintenance strategy.

For each pavement family, the “optimal” strategy was the one that yielded the maximum cost-effectiveness. It was found that these “optimal” strategies were generally different from those strategies currently being used by most sub-districts and districts in the state, with a few exceptions. For rigid

Interstates, the “optimal” strategy involved joint and crack (re)sealing every 7 years, and underdrain maintenance every year. The most cost-effective strategy for rigid non-Interstates was similar to that of their Interstates counterparts, the only difference being crack and joint sealing a 7, rather than 8 years intervals. For overlay Interstates, the most cost-effective strategy was found to be the application of micro-surfacing treatment every 3 years, and underdrain maintenance every year. In the case of overlay non-Interstates with high volumes, the application of a thin HMAC overlay every 5 years, crack sealing every third year after resurfacing or thin overlay, and underdrain maintenance every year, was found to yield the highest cost-effectiveness. A regimen of micro-surfacing treatment every 8 years, chip sealing 4 years after micro-surfacing, and crack sealing two years after micro-surfacing, was found to be the “optimal “ strategy for low-volume overlay non-Interstates. For full-depth asphaltic concrete pavements, the strategy with the highest cost-effectiveness involves application of a chip seal every 6 years after resurfacing and every 6 years thereafter, and crack sealing every 3 years after resurfacing and every 3 years thereafter. All these preventive maintenance strategies were associated with default corrective maintenance treatments whose costs and impacts were considered in the analysis.

**CHAPTER 9: TRADE-OFFS BETWEEN CAPITAL INVESTMENT FREQUENCY
AND LIFE-CYCLE MAINTENANCE EXPENDITURE**

9.1 Introduction

Findings from the literature review (Chapter 3) and the questionnaire survey (Chapter 4) suggest that a trade-off relationship could exist between levels of life-cycle preventive maintenance and the frequency of capital investments. It may be hypothesized that higher levels of life cycle preventive maintenance could translate to (i) less frequent capital investments, and/or (ii) lower levels of capital investments. Preliminary examination of data showed that while the first hypothesis seems to be a reasonable assumption worth investigation, there was really no basis for the second. In other words, different amounts of life-cycle preventive maintenance efforts (in terms of dollars per lane-mile) for pavements of similar characteristics seemed to have little or no relationship with the cost of subsequent resurfacing. It can be argued that little preventive maintenance could translate to more extensive and severe surface defects, and consequently, higher levels of surface preparatory works prior to the overlay. However, failure of the preliminary data analysis to reveal that trend suggests that the volume (cost) of preparatory works is typically so little compared to the volume of the main overlay activity, as such the cost of overlaying a fair pavement is similar to the cost of overlaying a poor pavement for which prior surface repairs was necessary, all other factors remaining constant. This aspect of the study therefore focused only on the relationships between levels of preventive maintenance and the frequency of capital investment (expressed as a the intervals between resurfacing).

Findings from the literature review and questionnaire survey also suggest that trade-off relationships probably exist between levels of preventive maintenance in a given period of time, and levels of corrective maintenance at a subsequent period. This chapter investigates such trade-off relationships and

presents mathematical models to represent such relationships, from which marginal effects of preventive maintenance efforts on frequency of capital investment and on subsequent corrective maintenance can be derived.

9.2 Rehabilitation Interval and Maintenance Expenditure Trade-off Modeling

9.2.1 Descriptive Statistics for Rehabilitation Interval vs. Maintenance Expenditure Modeling

As a prelude to modeling of rehabilitation interval as a function of average annual maintenance expenditure and other variables, a descriptive analysis of the major variables was carried out for all pavement families under investigation (Figures 9-1 to 9-3). Average annual maintenance expenditures are given in constant dollars (1995 values) per lane-mile. Average annual pavement loading is expressed in millions of equivalent single axle loads (ESALs). The analysis for full-depth asphaltic concrete considers only non-Interstate pavements as currently there are relatively very few Interstate full-depth asphalt pavements.

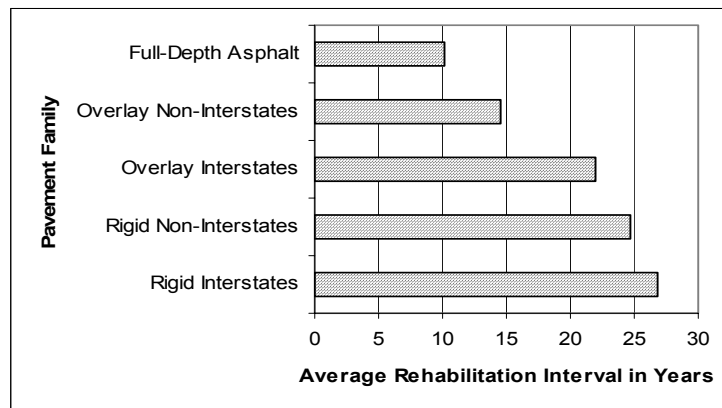


Figure 9-1: Average Rehabilitation Interval for Various Pavement Families

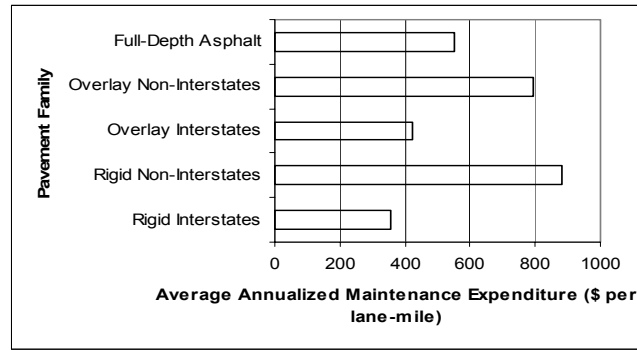


Figure 9-2: Average Annual Maintenance Expenditure for Various Pavement Families

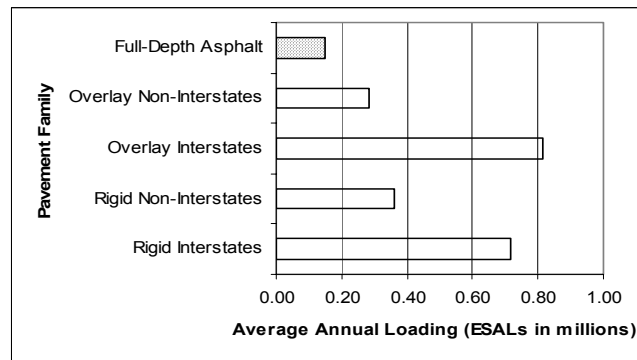


Figure 9-3: Average Annual Pavement Loading (in ESALs) for Various Pavement Families

Two alternative functional forms were considered in modeling the relationship between interval of rehabilitation and maintenance expenditure: the modified exponential curve and the S-curve. These mathematical forms were selected over several others that were also considered because they appeared to fit the data best and also because they permitted the interpretation of the resulting curve in ways that are relevant and are of interest from the perspective of pavement engineering. For instance, using such forms, it is possible to determine the rehabilitation interval corresponding to zero maintenance, the maximum rehabilitation interval to be obtained regardless of how much maintenance is carried out (i.e., the long-term maintenance effectiveness cap), and the maintenance expenditure cap (i.e., the average annual maintenance expenditure beyond which relatively very little incremental benefit in rehabilitation interval is afforded).

After several trials, the modified exponential functional form was selected for modeling the rehabilitation-maintenance trade-off, and is shown as follows:

$$Rehab\ Interval = A - B * C^{MLW} \dots\dots\dots (123)$$

Where

Rehab Interval = Interval of rehabilitation (observed service life) of given pavement section,

MLW = Maintenance-Load-Weather effects on pavement over its service life = average annual maintenance expenditure in 1995 dollars per lane-mile normalized by the product of average annual traffic loading (in millions of ESALs) and average annual level of weather severity during the study period.

A, *B*, and *C* are coefficient estimates.

9.2.2 Rigid Interstate Pavements Rehabilitation-Maintenance Trade-off Model

Figure 9-4 presents a scatter plot of rehabilitation interval and average annual maintenance expenditure, in constant dollars per lane-mile, of 16 rigid Interstate pavement sections.

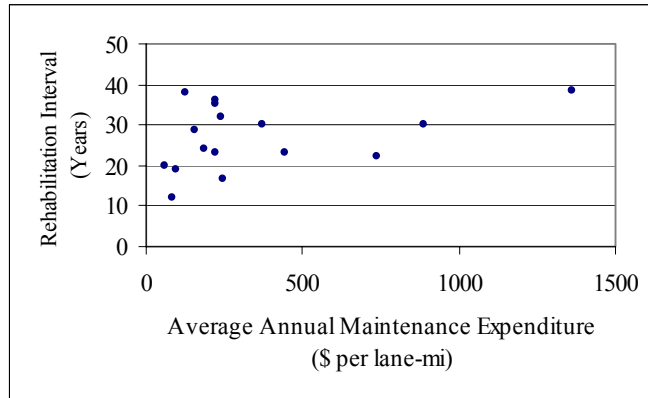


Figure 9-4: Rehabilitation Interval vs. Maintenance Expenditure, Rigid Interstates

Figure 9-4 does not seem to present indications of a strong functional relationship that rehabilitation interval increases with increased maintenance. It is well known that for a given level of

maintenance expenditure, the longevity of a pavement depends on the level of loading and the features of the climate at the region where such a pavement is located. For a given maintenance level, higher loading and greater weather severity levels (higher freeze-thaw, precipitation and freeze index), are likely to result in reduced rehabilitation interval. It is therefore obvious that maintenance expenditure is necessary but not sufficient to explain how long a rigid Interstate pavement (and indeed other pavements) will last. This point is especially crucial considering that for rigid Interstates, there is marked variation in loading levels across the various pavement sections in this family (coefficient of variation of 74.6%, see [Table 9-1](#)). Furthermore, within the study period, there were rigid Interstate pavements from as far as the northern tip (I-94 in Porter county, which was overlaid only recently) to the southern end (I-164 in Vanderburgh County). With vertical differences in freeze indices between such locations reaching over 600 degree-days, it can be inferred that there could be significant variation of weather effects from the north to the south of Indiana to warrant inclusion of weather in rehabilitation-maintenance trade-off modeling. It is therefore necessary to consider maintenance together with load and weather effects for modeling rigid Interstate pavement rehabilitation interval and maintenance expenditure.

[Figure 9-5](#) presents a scatter plot for rehabilitation interval on one axis, and a combined variable to represent the average annual effects of maintenance, load and weather on the other axis. Load is measured in millions of ESALs (MESALs), while weather is measured in weather severity units that range from 0.534 in the southern tip of Indiana to 0.859 at the northern tip. Maintenance is measured in 1000's of constant (1995) dollars expended per lane-mile of pavement.

The normalization of the maintenance variable by load and weather effects yielded more intuitive patterns of the relationship between rehabilitation and maintenance. [Figure 9-5](#) generally shows that while higher level of maintenance generally yields higher rehabilitation intervals, lower traffic loads and more favorable weather could also contribute to increased longevity. The subsequent paragraphs present the relationship between these parameters. Descriptive statistics of the variables used in modeling the trade off between rehabilitation and maintenance are provided in [Table 9-1](#), while the resulting model is presented in [Table 9-2](#).

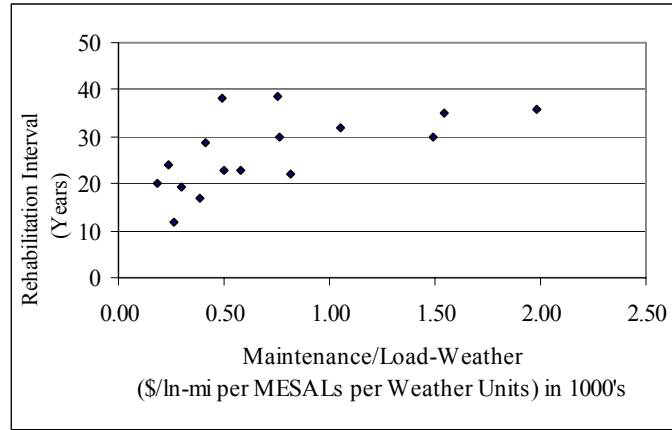


Figure 9-5: Rehabilitation Interval vs. Maintenance/Load-Weather, Rigid Interstates

Table 9-1: Descriptive Statistics: Rehabilitation-Maintenance Trade-off Modeling, Rigid Interstates

	Annual Weather Severity Level	Annual Loading (ESALS in millions)	Average annual Maintenance Expenditure (\$ per lane-mi)	Rehabilitation Interval (Years)
Mean	0.72	0.71	350	26.76
Maximum	0.84	2.43	1360	38.40
Minimum	0.55	0.20	60	12.00
Standard Deviation	0.09	0.53	350	7.94
Coefficient of Variation (%)	12.26	74.55	100.12	29.68

Table 9-2: Rehabilitation-Maintenance Trade-off Model Results for Rigid Interstates ($R^2 = 0.27$)

Variable/Constant	Coefficient Estimate	t statistic
<i>A</i>	34.0793	3.3633
<i>B</i>	19.8293	2.2132
<i>C</i>	0.2410	1.4635

The signs of the coefficients appear consistent with expectation. The term A has a value of 34.0793, and B is 19.8293. From Equation (123), when maintenance is zero, the rehabilitation interval is 14 years. In other words, if maintenance is not carried out on rigid Interstate pavement, it can be expected that rehabilitation would be necessary every 14 years. The signs of the coefficients of the terms B and C imply that higher values of maintenance (which lead to higher values of maintenance-load-weather effects, MLW) results in lower values of the exponential term (because C is a fraction between 0 and 1), and therefore yields lower values of the product term, and ultimately, higher values of the response variable (rehabilitation interval) as B is negative. Conversely, but with similar reasoning, higher values of traffic loading or weather severity lead to lower values of maintenance-load-weather effects, higher value of the exponential and product terms, and finally lower value of the rehabilitation interval. The t-statistic of the coefficient C is rather low, and implies that with the given data, this coefficient is significant only at 20% confidence. With a larger dataset, better statistics for all the above coefficients are expected.

A plot of the developed model is shown as Figure 9-6. This figure shows that for a rigid Interstate pavement with \$400 average annual maintenance expenditure per lane-mile, and annual traffic loading and weather severity of 1 million ESALs and 0.7 units, respectively, (which correspond to an MLW value of 0.5714), the corresponding rehabilitation interval is approximately 24 years.

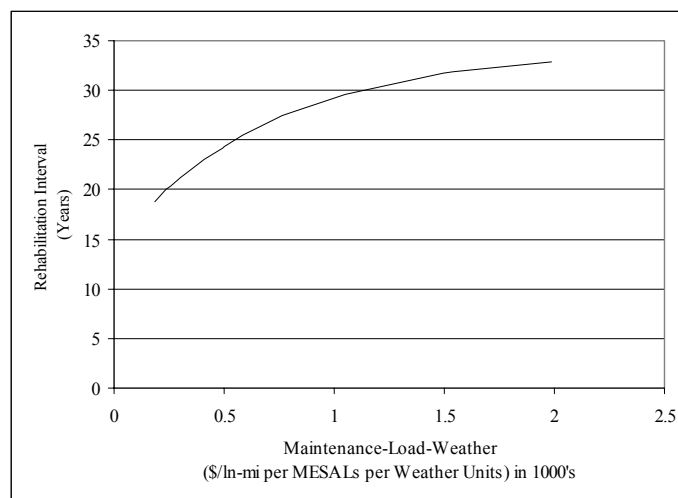


Figure 9-6: Fitted Values for Rehabilitation/Maintenance Trade-off Model, Rigid Interstates

9.2.2 Rigid Non-Interstate Pavements Rehabilitation-Maintenance Trade-off Model

Figure 9-7 presents a scatter plot of rehabilitation interval and average annual maintenance expenditure of 24 rigid non-Interstate pavement sections, normalized by their traffic loading and weather severity levels. This figure indicates that higher level of maintenance is generally associated with higher pavement longevity, while higher traffic loads and poorer (higher weather severity) translate to reduced pavement longevity. Descriptive statistics of the major variables used in modeling the trade-off between rehabilitation and maintenance, for rigid non-Interstate pavements are provided in Table 9-3, while results of the model are shown in Table 9-4.

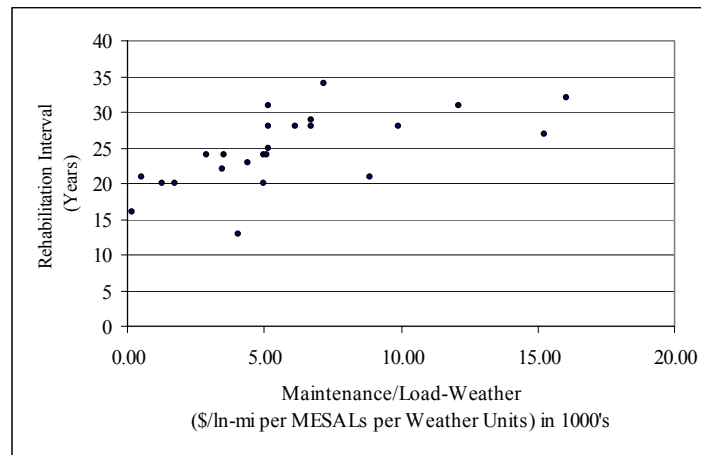


Figure 9-7: Rehabilitation Interval vs. Maintenance/Load-Weather, Rigid Non-Interstates

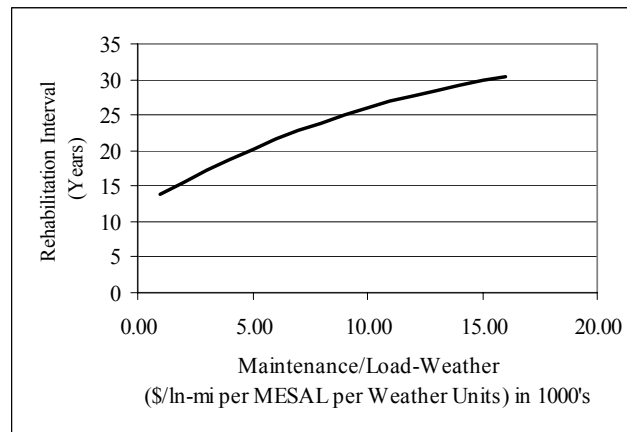
Table 9-3: Descriptive Statistics: Rehabilitation-Maintenance Trade-Off Modeling, Rigid Non-Interstates

	Average Annual Weather Severity Level	Average Annual Loading (ESALS in millions)	Average annual Maintenance Expenditure (\$ per lane-mi)	Rehabilitation Interval (Years)
Mean	0.68	0.26	880	24.71
Maximum	0.80	0.97	3940	34.00
Minimum	0.55	0.07	50	13.00
Standard Deviation	0.07	0.19	790	5.13
Coefficient of Variation	10.46%	73.31%	89.25%	20.76%

Table 9-4: Rehabilitation-Maintenance Trade-off Model Results for Rigid Non-Interstates ($R^2 = 0.35$)

Variable/Constant	Coefficient Estimate	t-statistic
<i>A</i>	39.0608	1.6906
<i>B</i>	27.0552	1.4182
<i>C</i>	0.9281	8.6095

The results obtained are similar to that for rigid Interstates. The signs of the estimated values of the constant terms are as expected. Because the constant terms *A* and *B* have values of 39.06 and 27.06, respectively, the rehabilitation interval (or pavement longevity) if no maintenance is carried out in the life of the pavement (represented by the extrapolated intercept on the ordinate), is computed as 12 years, signifying that for a zero-maintenance scenario for this family of pavement, rehabilitation would have to be carried out every 11 years. The signs of the coefficients of the terms *B*, and *C*, imply that higher values of maintenance (which lead to higher values of maintenance-load-weather effects, MLW) results in lower values of the exponential term (because *C* is a fraction between 0 and 1), and therefore yields lower values of the product term, and ultimately, higher values of the response variable (rehabilitation interval) as *B* is negative. Similarly, higher values of traffic loading or weather severity lead to lower values of maintenance-load-weather effects, higher value of the exponential and product terms, and finally lower value of the rehabilitation interval. A plot of the developed model is shown as [Figure 9-8](#).



[Figure 9-8](#): Fitted Values for Rehabilitation/Maintenance Trade-off Model, Rigid Non-Interstates

9.2.3 Overlay Interstate Pavements Rehabilitation-Maintenance Trade-off Model

A scatter plot of rehabilitation interval and average annual maintenance expenditure of 23 overlay Interstate pavement sections is provided as Figure 9-9. In order to sharpen the relationship, load and weather effects were included to normalize the maintenance expenditure values. Descriptive statistics of the variables used in modeling the trade-off between rehabilitation and maintenance, for overlay Interstate pavements, are provided in Table 9-5, while the model results are presented in Table 9-6.

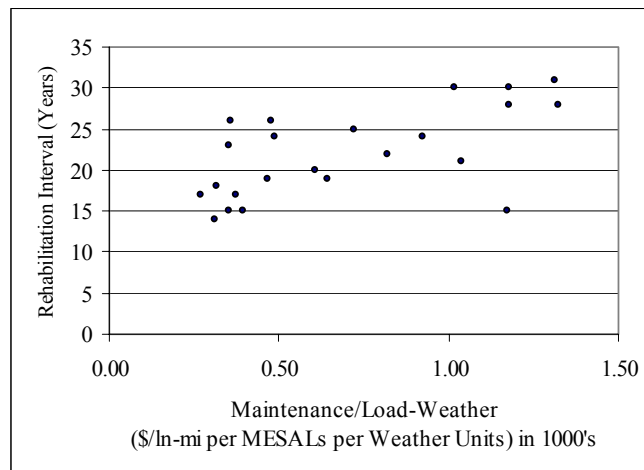


Figure 9-9: Rehabilitation Interval vs. Maintenance/Load-Weather, Overlay Interstates

Table 9-5: Descriptive Statistics: Rehabilitation-Maintenance Trade-Off Modeling, Overlay Interstates

	Average Annual Weather Severity Level	Average Annual Loading (ESALS in millions)	Average annual Maintenance Expenditure (\$ per lane-mi)	Rehabilitation Interval (Years)
Mean	0.75	0.81	420	22.04
Maximum	0.86	1.96	1220	31.00
Minimum	0.58	0.36	90	14.00
Standard Deviation	0.08	0.48	320	5.36
Coefficient of Variation	10.58%	58.98%	76.36%	24.29%

Table 9-6: Rehabilitation-Maintenance Trade-off Model Results for Overlay Interstates ($R^2 = 0.38$)

Variable/Constant	Coefficient Estimate	t statistic
<i>A</i>	37.5909	2.2218
<i>B</i>	23.3918	4.2154
<i>C</i>	0.5394	1.7715

The results obtained for overlay Interstates are similar to that for rigid Interstates. The signs of the estimated values of the constant terms are expected. The terms *A* and *B* have values of 37.59 and 23.39, respectively. This means that the zero-maintenance pavement life is expected to be 14 years. As explained for the results of the rehabilitation-maintenance modeling for the previous pavement families, the signs of the coefficients of the terms *B* and *C* imply that higher values of maintenance (which lead to higher values of maintenance-load-weather effects, MLW) results in lower values of the exponential term. Similarly, higher values of traffic loading or weather severity lead to lower values of the maintenance-load-weather variable, higher value of the exponential and product terms, and finally lower value of the rehabilitation interval. A plot of the developed model is shown as Figure 9-10.

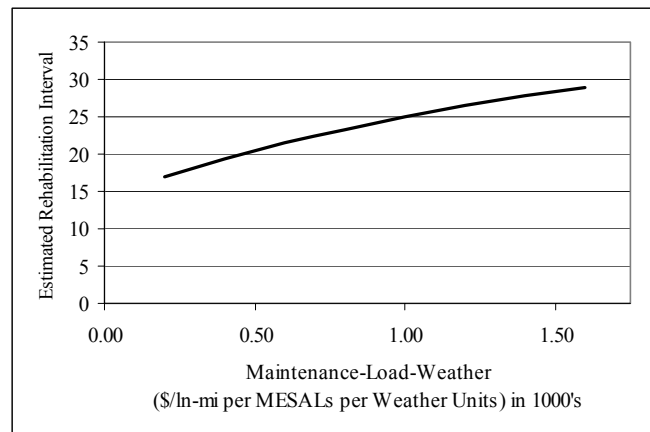


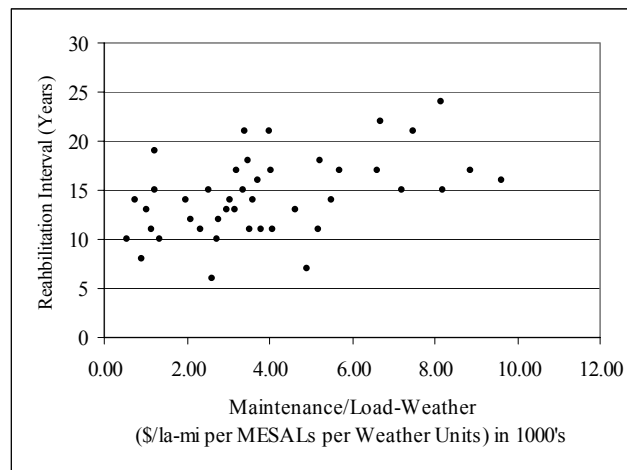
Figure 9-10: Fitted Values for Rehabilitation/Maintenance Trade-off Model, Overlay Interstates

9.2.4 Overlay Non-Interstate Pavements Rehabilitation-Maintenance Trade-off Model

The average annual maintenance expenditure (normalized by loading and weather severity) of 43 overlay non-Interstate pavement sections, as well as their corresponding rehabilitation

intervals (years to resurfacing or reconstruction), is provided as [Figure 9-11](#). Table 9-7 presents the descriptive statistics of the major variables used in modeling the trade-off between rehabilitation and maintenance, for overlay non-Interstate pavements. Details of the model are shown in [Table 9-8](#).

The results obtained for overlay non-Interstates were slightly different from those obtained for their Interstate counterparts, even though the signs of the estimates of the constant terms were still the same. The terms *A* and *B* have values of 61.87 and 50.70, respectively, implying that in the hypothetical event of zero maintenance throughout the life of the pavement, rehabilitation would be needed every 11 years.



[Figure 9-11](#): Rehabilitation Interval vs. Maintenance/Load-Weather, Overlay Non- Interstates

Table 9-7: Descriptive Statistics: Rehabilitation-Maintenance Trade-off Modeling, Overlay Non-Interstates

	Average Annual Weather Severity Level	Average Annual Loading (ESALS in millions)	Average Annual Maintenance Expenditure (\$ per ln-mi)	Rehabilitation Interval (Years)
Mean	0.76	0.28	790	14
Maximum	0.86	0.94	5220	24
Minimum	0.55	0.03	80	6
Standard Deviation	0.09	0.20	890	4.03
Coefficient of Variation	11.65%	69.35%	111.72%	28.00%

Table 9-8: Rehabilitation-Maintenance Trade-off Model Results for Overlay Non-Interstates ($R^2 = 0.23$)

Variable/Constant	Coefficient Estimate	T statistic
<i>A</i>	61.8719	1.3254
<i>B</i>	50.7040	2.0547
<i>C</i>	0.9824	4.8721

As *B* is negative and *C* is a fraction between 0 and 1, higher values of maintenance lead to higher values of maintenance-load-weather effects, MLW, resulting in lower values of the exponential term, lower values of the product term, and finally, higher rehabilitation interval. Similarly, higher values of traffic loading or weather severity lead to lower values of maintenance-load-weather effects, higher value of the exponential and product terms, and finally lower value of the rehabilitation interval. A plot of the developed model is shown as [Figure 9-12](#).

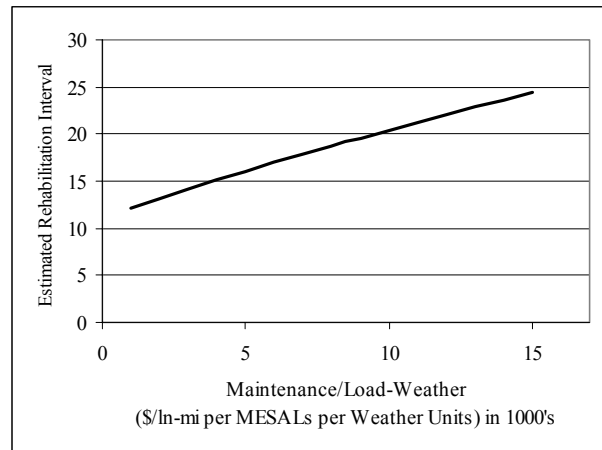


Figure 9-12: Fitted Values for Rehabilitation/Maintenance Trade-off Model, Overlay Non-Interstates

9.2.5 Full-depth Asphalt Pavements Rehabilitation/Maintenance Trade-off Model

[Figure 9-13](#) illustrates the variation of average annual maintenance expenditure of 25 full-depth asphalt non-Interstate pavement sections with their corresponding rehabilitation intervals (years to resurfacing or reconstruction). The maintenance values shown are normalized by their traffic loading and weather severity levels. Table 9-9 and 9-10 present the descriptive statistics of the variables used in modeling the trade off between rehabilitation and maintenance, and model results, respectively.

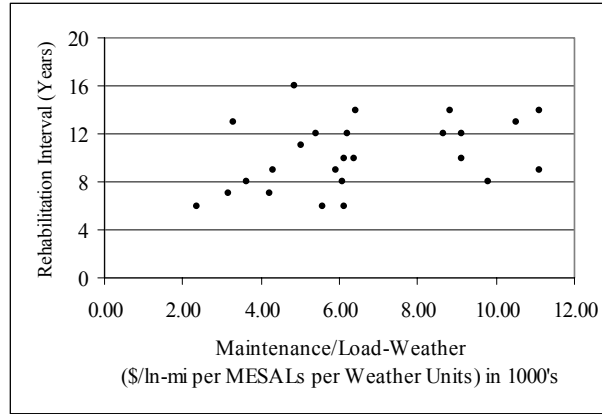


Figure 9-13: Rehabilitation Interval vs. Maintenance/Load-Weather, Full-depth Asphalt Pavements

Table 9-9: Descriptive Statistics: Rehabilitation-Maintenance Trade-off Modeling, FDA Pavements

	Average Annual Weather Severity Level	Average Annual Loading (ESALS in millions)	Average annual Maintenance Expenditure (\$ per lane-mi)	Rehabilitation Interval (Years)
Mean	0.62	0.15	550	10.18
Maximum	0.77	0.39	1270	16.00
Minimum	0.54	0.00	20	6.00
Standard Deviation	0.08	0.12	390	3.03
Coefficient of Variation	12.23%	79.88%	71.85%	29.80%

Table 9-10: Rehabilitation-Maintenance Trade-off Model Results for Full-depth Asphalt Pavements ($R^2 = 0.14$)

Variable/Constant	Coefficient Estimate	Asymptotic Standard Error
<i>A</i>	11.9249	3.7251
<i>B</i>	8.2775	1.4352
<i>C</i>	0.7678	2.4516

The results obtained for full-depth asphalt pavements indicate that a zero maintenance scenario is associated with a relatively very short pavement life, i.e., 4 years. This value seems rather low, but could be possible. The signs of the estimates of the constant terms are the same as those found for the other pavement families: B is negative and C is a fraction between 0 and 1. Therefore higher values of maintenance lead to higher values of maintenance-load-weather effects (MLW), resulting in lower values of the exponential term, lower values of the product term, and finally, higher rehabilitation interval. Also, higher values of traffic loading or weather severity lead to lower values of maintenance-load-weather effects, higher value of the exponential and product terms, and finally lower value of the rehabilitation interval. A plot of the developed model is shown as [Figure 9-14](#).

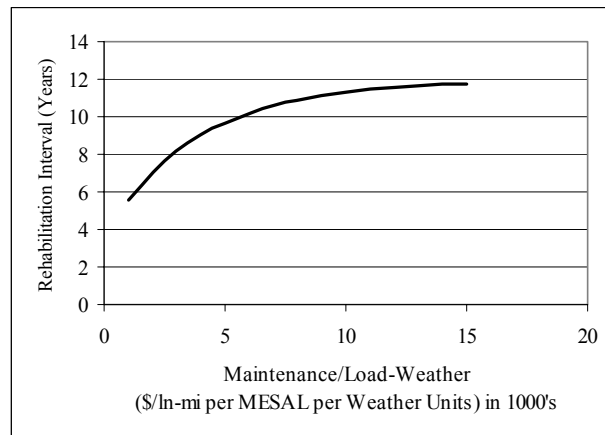


Figure 9-14: Fitted Values for Rehabilitation/Maintenance Trade-off Model, Full-depth Asphaltic Concrete Pavements

9.3 Marginal Effects of Rehabilitation/Maintenance-Load-Weather

As seen from the previous section, the models for estimating trade-offs between rehabilitation interval on one hand and the dynamic factors of pavement deterioration on the other, generally increases with increasing maintenance and decreasing load and weather severity, but does so in ways that differ significantly not only from one pavement family to another, but also within each pavement family depending on the average annual values of the factors experienced by a particular pavement. For instance,

the plot of rehabilitation interval versus pavement maintenance experience (the ratio of maintenance expenditure to the product of load and weather factors) for rigid Interstates (Figure 9-25) showed that an annual increase of \$100 per lane-mile per load per weather units is associated with approximately 2.5 years of extended service life when the value of maintenance-load-weather effects is between 0 and 0.5 units, but yields less than a year of extended service life when the value of maintenance-load-weather effects is between 1.5 and 2 units. This suggests that the trade-off between rehabilitation intervals changes significantly with each level of maintenance-load-weather effects (or the level of each individual factor), and actually diminishes beyond certain values of maintenance, load, or weather.

While rehabilitation/maintenance trade-off functions enable the determination of rehabilitation interval for a given level of maintenance experience, they do not readily provide the expected increase or decrease in rehabilitation interval (or pavement service life) for specified changes in average annual maintenance, average annual traffic loading, or average annual weather severity. For this reason, marginal effects models were developed as part of the present study to facilitate the estimation or prediction of such expected changes on rehabilitation interval. The theoretical basis for computation of marginal effects and the reasons for selection of the concept of elasticities over that of derivatives to represent marginal effects, are presented in Chapter 6. The discussion below presents the marginal effects models that were developed from the trade-off models, for each pavement family. Fixed values of two factors were selected to investigate the marginal effect of the third factor on rehabilitation interval. For example, to investigate the effect of average annual maintenance, average annual weather and traffic loading were taken as 0.7 weather units and 1 million ESALs, respectively, which are typical values for this pavement family.

9.3.1 Marginal Effects Models for Rigid Interstates

Figures 9-15 to 9-17 shows that the levels of average annual maintenance, load and weather each have influential marginal effects on rehabilitation interval of rigid Interstate pavements. As seen from the figure, at low levels of average annual maintenance (0-\$400 per lane-mile), increasing maintenance has positive and rapidly increasing elasticity. In other words, the percentage change in rehabilitation interval increases in response to a unit change in maintenance expenditure, up to a peak of average annual

maintenance expenditure of \$400 per lane-mile, after which the marginal benefits in increasing maintenance decreases gradually in a convex fashion. At a maintenance level of approximately \$3,500 per lane-mile, the marginal benefits of increasing maintenance on rigid Interstate pavements dwindle to zero, meaning that beyond this level of maintenance expenditure, little or no increase in rigid Interstate pavement service life can be expected. Generally, this curve makes it possible to estimate the percentage change in rehabilitation interval of rigid Interstate pavements in response to a unit change in maintenance expenditure (due to for instance, sudden or expected shortfalls in maintenance funding).

The trend for the marginal effects of traffic loading on rehabilitation interval is somewhat different from that for maintenance. At low levels of pavement loading (less than 0.35 million ESALs), increasing traffic loading has adverse albeit relatively little effect on pavement longevity. After an inflexion point at approximately 0.45 million ESALs, the adverse effect of increased loading becomes more obvious. Another inflexion point is obtained at a loading level of approximately 1.2 million ESALs, after which the rate of change in marginal effects seems to slow to an estimated maximum of approximately -0.3, signifying that the maximum reduction of pavement service life due to very high loading levels is about 30%. In general, this marginal effects function enables the determination of the percentage reduction in rehabilitation interval of a rigid Interstate pavement in response to unit changes in average annual traffic loading of a pavement in this family due to socio-economic, institutional or policy changes involving economic growth, regulation or deregulation.

Marginal effects of rehabilitation interval with respect to weather severity is marked by relatively less intricacy, compared to that for maintenance and load, and is characterized by a line (slightly concave) ranging from about -0.1 elasticity at relatively low levels of weather severity, i.e., 0.5 units, to -0.2 elasticity at relatively high levels of weather severity, i.e., 0.9 units. This marginal effects function suggests that for a unit increase in weather severity, rigid Interstate pavements in southern Indiana suffer a relatively lower increment in deterioration, compared to that experienced by such pavements in northern Indiana. In other words, the effect of a global or continental change in weather patterns will have a more profound incremental effect on rigid Interstate pavements located in regions of relatively severe weather than those located in areas of relatively mild weather.

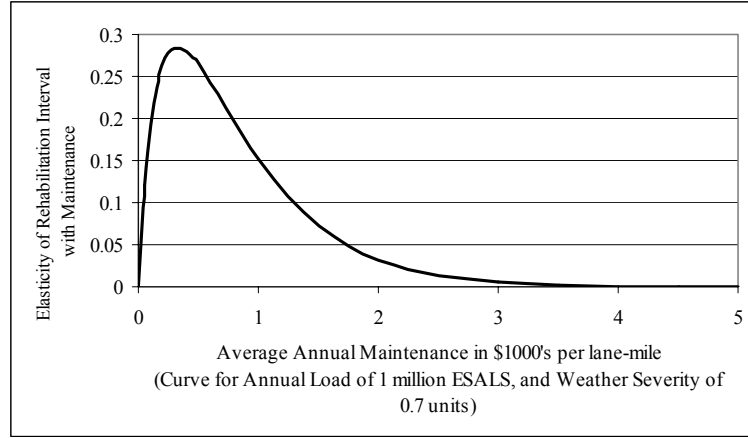


Figure 9-15: Curves for Marginal Effects of Maintenance Expenditure, Rigid Interstates

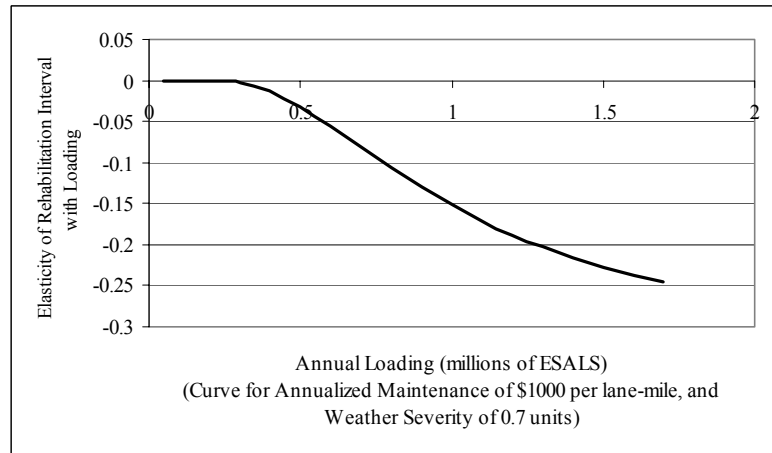


Figure 9-16: Curves for Marginal Effects of Traffic Loading, Rigid Interstates

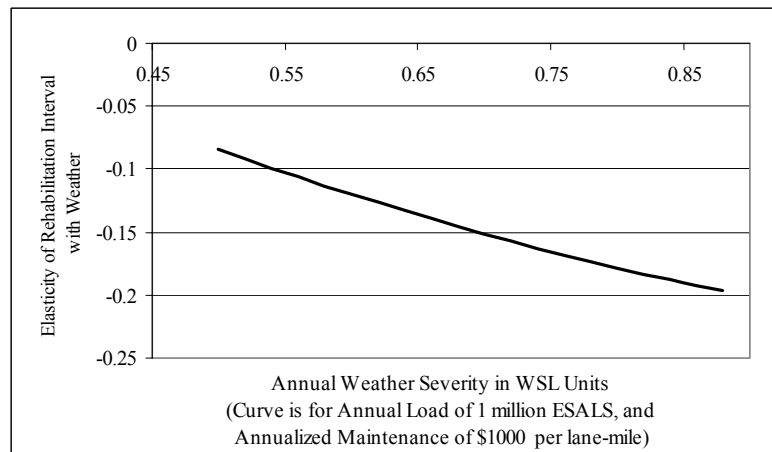


Figure 9-17: Curves for Marginal Effects of Weather Severity, Rigid Interstates

9.3.2 Marginal Effects for Rigid Non-Interstates

The marginal effects of average annual maintenance expenditure, traffic loading, and weather on rehabilitation interval are illustrated in **Figures 9-18 to 9-20**. The figure indicates a large and positive elasticity of rehabilitation interval with respect to maintenance, i.e., increasing levels of maintenance results in increasing rehabilitation interval in a concave fashion. The elasticity reaches a high of 0.37 at an average annual maintenance level of approximately \$5,000 per lane mile per year. Compared to their Interstate counterparts, rigid non-Interstates have a higher marginal effects peak (0.37 versus 0.28) and a higher level of corresponding average annual maintenance expenditure (\$5,000 versus \$400 per lane-mile). In other words, the maximum percentage change in rehabilitation interval in response to a unit increase in maintenance is higher for rigid non-Interstates than it is for rigid Interstates. This signifies that rigid non-Interstates, obviously because of their relatively inferior design and construction standards, typically have “more room” for improvement, and therefore exhibit higher marginal effects, compared to their Interstate counterparts, at a given level of maintenance spending.

Also, the picture for marginal effects of traffic loading on rehabilitation interval is very different for rigid non-Interstates than it is for rigid Interstates. The point of maximum elasticity is not only higher for rigid non-Interstates, but is reached much more quickly (i.e., it is associated with a lower loading level) in comparison to their Interstate counterparts. There are two reasons for this: first, rigid non-Interstates carry far less traffic than rigid Interstates, as evidenced in the trends of pavement loading (Chapter 2). More importantly, the smaller slab thickness and other inferior design and construction features of rigid non-Interstates, compared to their Interstate counterparts, renders such pavements more susceptible to the damaging effects of traffic loading, and are consequently associated with greater values of elasticity (-0.37 versus -0.3) at any loading level. For the range of pavement loading considered, the marginal effects of rigid non-interstates peaks off rather early (after approximately 0.2 million ESALs), unlike the curve for rigid Interstates, and decreases rather gently thereafter. From the graph it is seen that the gentle decrease in marginal effects with loading occurs within a range of high loading levels that are atypical of loading on such pavements, and therefore suggests that for failed rigid pavements, marginal effects of loading are relatively little.

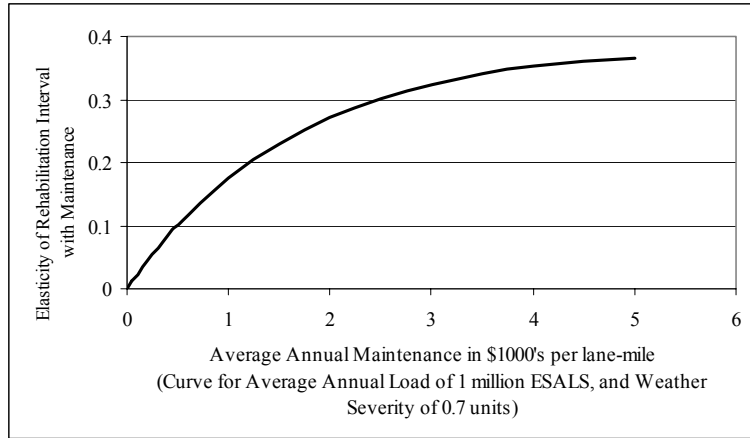


Figure 9-18: Curves for Marginal Effects of Maintenance Expenditure, Rigid Non-Interstates

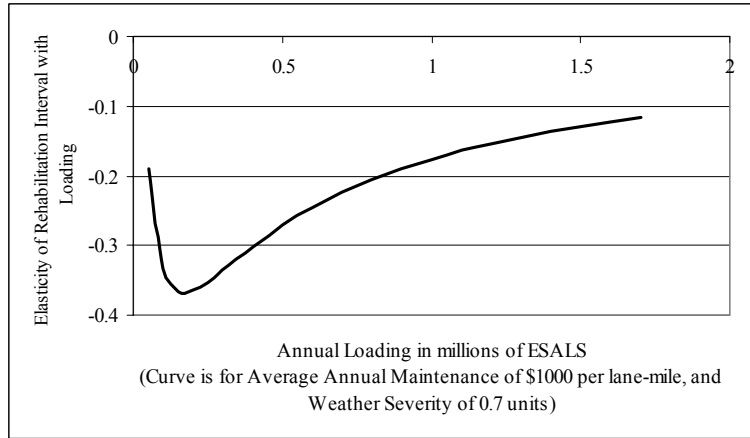


Figure 9-19: Curves for Marginal Effects of Traffic Loading, Rigid Non-Interstates

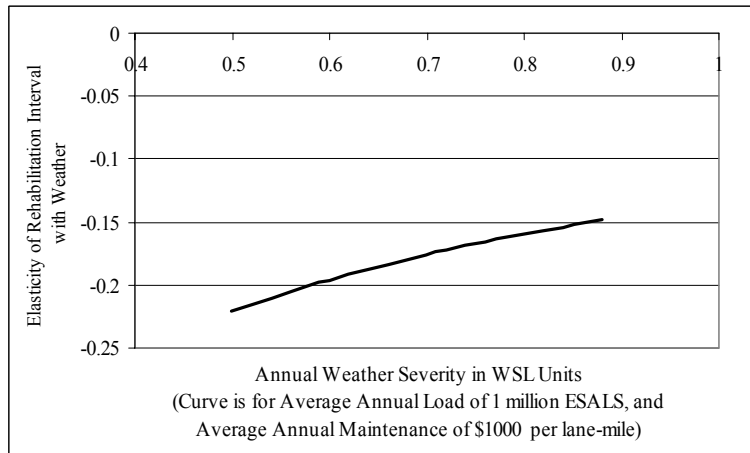


Figure 9-20: Curves for Marginal Effects of Weather Severity, Rigid Non-Interstates

9.3.3 Marginal Effects Curves for Overlay Interstate Pavements

The marginal effects curves obtained for overlay (AC-on-PCC) Interstates (Figures 9-21 to 9-23) are similar to those obtained for rigid Interstates but markedly dissimilar to those for overlay non-Interstates. This suggests that the marginal effects of the dynamic factors of pavement deterioration (maintenance, load, and weather) on pavement rehabilitation intervals (and probably on pavement performance in general) are dictated to larger extent, by their route type (or functional class) and to a lower extent by their surface type. This is obviously because there is relatively little variation in design and construction features such as subgrade quality requirements, compaction standards, etc., across highway route type compared to pavement surface type. In other words, Interstate pavements are associated with design and construction (D&C) features that are very different from (far superior to) those of non-Interstate pavements. On the other hand, rigid pavements (apart from their surface material) have little differences in D&C features compared to overlay pavements. This is consistent with expectation, as Interstate pavements are appropriately designed to withstand the high loading levels they experience (5-10 times the loading on non-Interstates) and therefore have very different standards. Also, the similarity in D&C features between rigid and AC-on-PCC overlay pavements is borne out of the fact that the latter were formerly rigid pavements that merely received an AC overlay at some point in time, with little or no change in the configuration or quality of the underlying pavement layers (base, subbase, and subgrade).

As the marginal effects curves for rigid Interstates and overlay Interstates are similar, the general trends exhibited by the marginal effects curves for the latter can be explained in a similar manner as was done earlier for rigid Interstates (Figures 9-21 to 9-23). However, there are a few subtle differences that merit discussion. For the overlay Interstates pavement family, the elasticity of rehabilitation interval with respect to maintenance peaks at values of elasticity that are higher than for their rigid counterparts (0.31 versus 0.28), and is associated with a higher level of average annual maintenance to reach that peak (approximately \$900 versus \$400 per lane-mile). This observation, as well as the difference in the positions of the two curves, implies that overlay Interstates have a higher maximum potential for maintenance effectiveness per unit change in maintenance expenditure compared to rigid Interstates. Also, the point at which the marginal effects curve converges to zero is over twice as high for overlay Interstates than it is for

their rigid counterparts, implying that for the latter, there is relatively “less room” for improvement (extension in rehabilitation interval) that can be expected from each additional dollar of maintenance. Also, a comparison of the marginal effects and loading curves for the two pavement families shows that the points of inflexion are reached much earlier for overlay Interstates compared to their rigid counterparts, and the maximum elasticity is lower for the latter (-0.3 versus -0.32). This is inferential of the greater incremental effect of a unit change in traffic load on the longevity of overlay Interstates compared to rigid Interstates. An examination of the marginal effects of rehabilitation interval with respect to weather for the two pavement families showed that higher elasticities are associated with overlay interstates than rigid Interstates. For pavements at the southern tip of Indiana for instance, (weather severity level of 0.5 units), a unit increase in weather severity would result in about 27% reduction in pavement rehabilitation interval, in contrast to a corresponding value of 8% for rigid Interstates located in that region. This suggests that overlay Interstate are more vulnerable to the effects of weather changes compared to rigid Interstates. Also, the elasticity/weather curve for overlay Interstates is markedly concave, while that for rigid Interstates is only slightly concave-up, implying that the peak elasticity (not shown on the graphs for the intervals of weather units investigated) is attained much earlier (at a lower level of weather severity) for overall than it is for rigid interstates. Again, this is reflective of the relatively higher vulnerability of asphaltic concrete to severe weather, compared to Portland cement concrete.

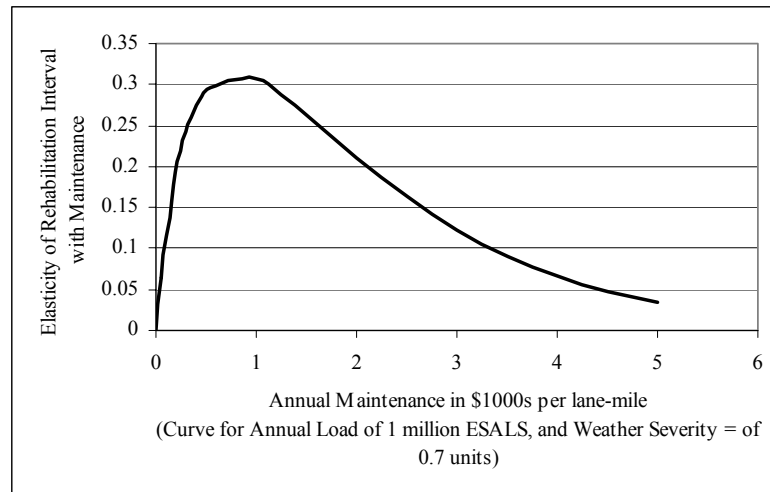


Figure 9-21: Curves for Marginal Effects of Maintenance Expenditure, Overlay Interstates

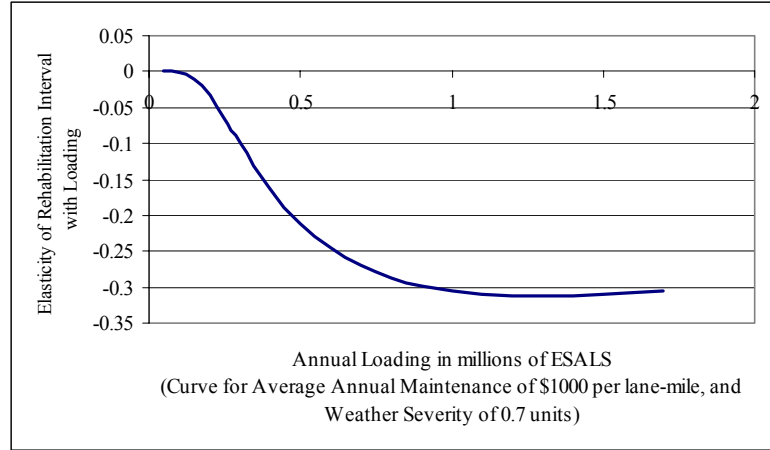


Figure 9-22: Curves for Marginal Effects of Traffic Loading, Overlay Interstates

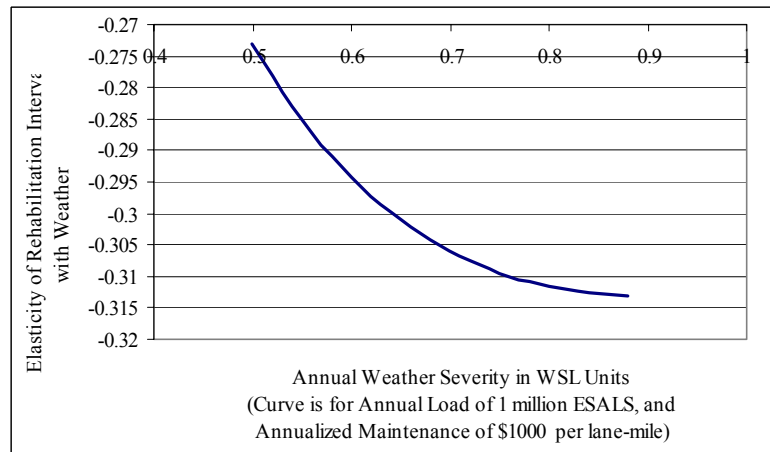


Figure 9-23: Curves for Marginal Effects of Weather Severity, Overlay Interstates

9.3.4 Marginal Effects Curves for Overlay Non-Interstates

The marginal effects curves developed for overlay non-Interstates (**Figures 9-24 to 9-26**) were generally similar to those for rigid non-Interstates, with the exception that the rehabilitation/loading curves differ at low levels of loading. Again, this suggests that marginal effects of maintenance, load and weather on pavement rehabilitation frequency exhibit more similarity by functional class than by surface type, for the same reasons as explained in the preceding section. The explanations proffered for rigid non-Interstates therefore generally apply to overlay non-Interstates, with a few exceptions that are hereby discussed.

For rigid non-Interstates, the elasticity of rehabilitation with respect to loading has a noticeable peak of -0.37 , while for overlay non-Interstates, an estimated maximum value of -0.5 is observed. This means that the maximum reduction in rehabilitation interval due to a unit increase in load is higher for overlay non-interstates compared to rigid non-Interstates. Also, the value of average annual pavement loading corresponding to the peak marginal effect is higher for rigid non-Interstates than it is for overlay non-Interstates (0.3 and 0.07 million ESALs, respectively). These observations are probably explained by the relatively lower levels of loading experienced by overlay non-Interstate pavements, and the higher vulnerability of overlay pavements to effects of increased loading, relative to their rigid counterparts.

The elasticity of rigid non-Interstates with respect to weather effects is greater for rigid non-Interstates than for overlay non-Interstates. In other words, the incremental effect of weather changes on rigid non-Interstate pavements exceeds that for their overlay counterparts, a finding that is in direct contrast to that observed for Interstate pavements. An explanation for this is borne in the fact that overlaying an existing rigid pavement is a two-edged sword: Such overlay reinforces the structural integrity of the pavement, but at the same time “burdens” it with a new surface material that is more susceptible to the vagaries of the weather compared with the old surface type. The net influence of these two effects of overlaying an existing rigid pavement with asphaltic concrete in the long-term (rehabilitation interval) depends on the existing strength of the pavement and the severity of the weather at that location. Overlaying a rigid non-Interstate pavement offers a net benefit to the pavement because the benefits of the overlay (increased strength) outweigh the dis-benefits (increased exposure to weather effects). This, most likely, explains why incremental effect of weather changes on rigid non-Interstate pavements exceeds that for such pavements that have received an overlay. However, the case for Interstate pavements is quite opposite: overlaying a rigid Interstate pavement offers a net disservice to the pavement because the dis-benefits of the overlay outweigh the benefits. Therefore, the incremental effect of weather changes on rigid Interstate pavements is less than it is for such pavements that have received an overlay. Obviously the already superior D&C features (and consequently, strength) of Interstate pavements make the benefits (structural reinforcement) of an overlay relatively little.

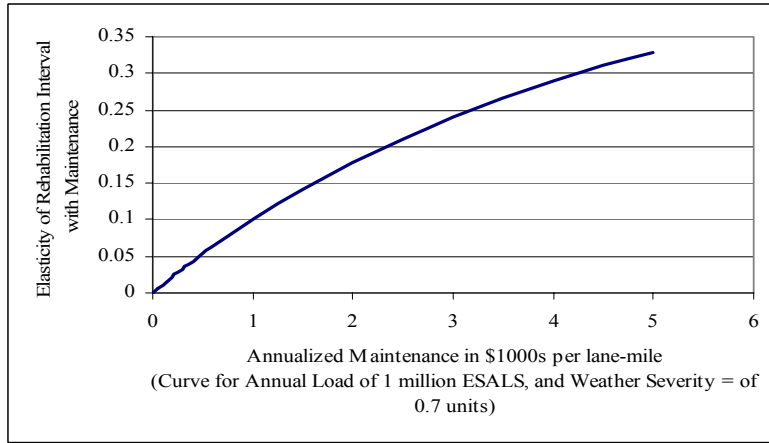


Figure 9-24: Curves for Marginal Effects of Maintenance Expenditure, Rigid Interstates

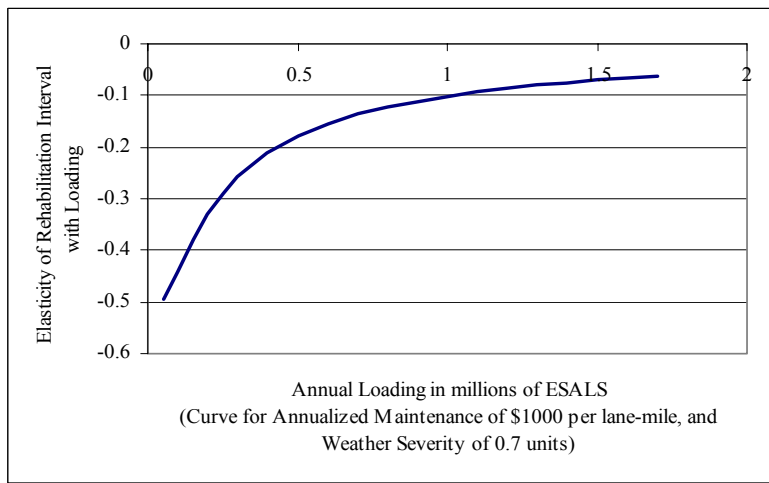


Figure 9-25: Curves for Marginal Effects of Traffic Loading, Rigid Interstates

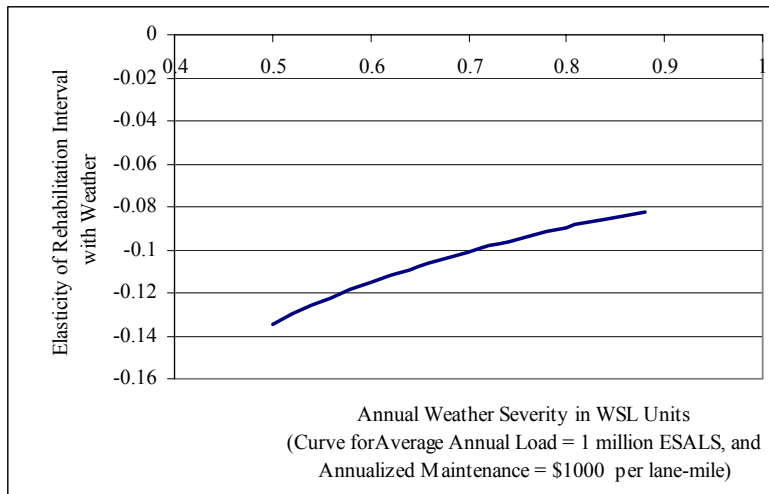


Figure 9-26: Curves for Marginal Effects of Weather Severity, Rigid Interstates

9.3.5 Marginal Effects Curves for Full-depth Asphaltic Concrete Pavements

The general patterns of the marginal effects of rehabilitation interval with respect to maintenance, weather and loading, for full-depth asphaltic concrete pavements generally follow a similar trend to those of other pavement types (**Figures 9-27 to 9-29**). The elasticity of rehabilitation interval with average annual maintenance expenditure for AC pavements is highest among all other pavement families. Furthermore, the estimated maximum elasticity, for full-depth asphaltic concrete pavements, is attained at an average annual maintenance level that is highest compared to other pavement families. Finally, for a given level of maintenance, the values of elasticity of rehabilitation interval with respect to load and weather for this pavement family are higher than those for other pavement families. The rehabilitation/maintenance elasticity curve for full-depth asphaltic concrete pavements dwindles to a value of zero at a relatively high maintenance value (over \$10,000 per lane-mile per year), much higher than the corresponding value for other pavement families. These observations signify that the percentage increase in rehabilitation interval of full-depth asphaltic concrete pavements in response to a unit change in maintenance is by far the most significant. Also, the adverse effects of weather and traffic loading are reflected in the negative sign of elasticities, which reach an early peak for this family of pavements, and reduce gently thereafter.

The curve also shows that the elasticity of rehabilitation interval with respect to weather is higher for full-depth asphalt pavements than it is for overlays or rigid pavements, regardless of weather regime (northern or southern Indiana). However, the elasticity of rehabilitation interval with respect to loading is slightly lower for full-depth asphalt pavements than their overlay or rigid counterparts. While this implies that the reduction in service life for this family of pavements in response to a unit change in traffic loading is less compared to that for rigid or overlay pavement, even though full-depth AC service life, for a given level of loading, is least compared to all other pavement families.

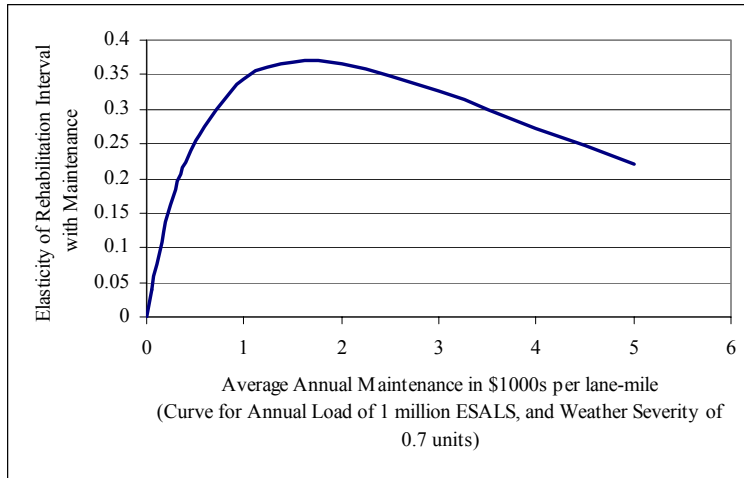


Figure 9-27: Curves for Marginal Effects of Maintenance Expenditure, Full-depth Asphalt Pavements

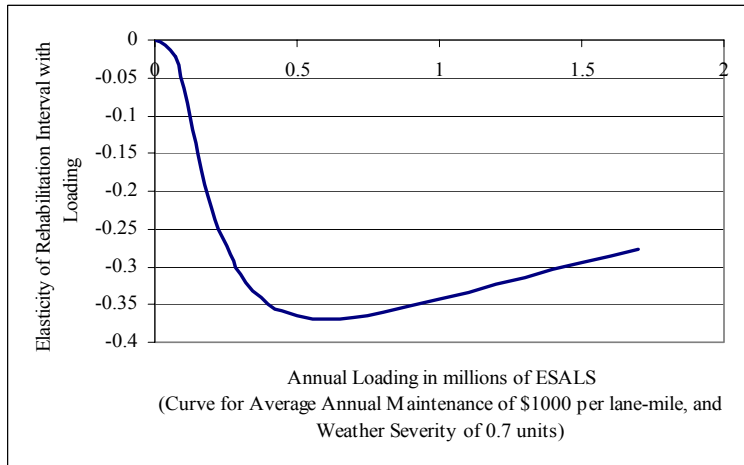


Figure 9-28: Curves for Marginal Effects of Traffic Loading, Full-depth Asphalt Pavements

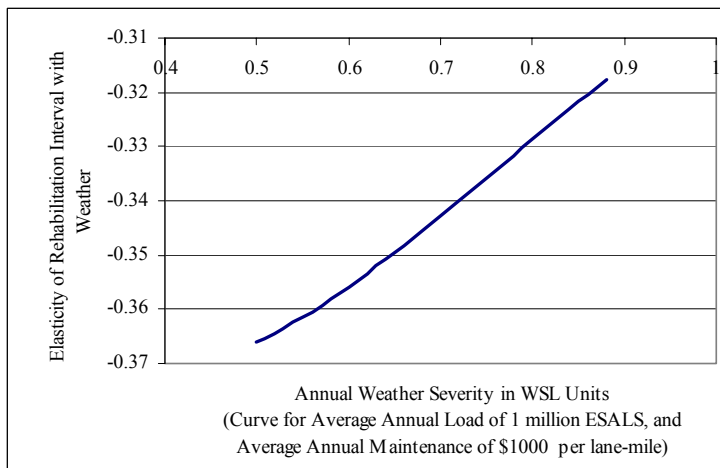


Figure 9-29: Curves for Marginal Effects of Weather Severity, Full-depth Asphalt Pavements

9.4 Modeling of Trade-off Between Preventive and Corrective Maintenance

Trade-off analyses were conducted to examine the relationship between preventive and corrective maintenance over a short-term period of three years. Levels of maintenance were expressed in terms of dollar expenditure per lane-mile, expressed in equivalent 1995 values. Among several mathematical forms considered, the Gompertz Curve functional form was found to be the most appropriate for modeling the 3-year trade-off relationship between preventive and corrective maintenance. This is because this curve provided a direct and intuitive engineering interpretation of the model shape, goodness of fit to observed data, and ease of convergence of the non-linear optimization algorithm. The developed model was as follows:

$$3YCM = C * A^{(B^{3YPM})} \dots\dots\dots(124)$$

Where

$3YCM$ = Total expenditure on corrective maintenance activities in a three year period

$3YPM$ = Total expenditure on preventive maintenance activities in the subsequent three year period.

A , B , and C are coefficient estimates.

Two separate models were developed: for “young” pavements, i.e., pavements that had not yet reached half-way their typical service lives, and for “old” pavements, i.e., those that had exceeded one-half of their typical service lives. The models obtained for “old” and “young” pavements are presented in Table 9-11 and 9-12, respectively. An important assumption made in this aspect of the present study is that the impact of any corrective maintenance activity on pavement condition during the first three years (in which preventive maintenance was carried out) is negligible. It is also assumed that the impact of preventive maintenance activities during the subsequent three years (when corrective maintenance is considered) is negligible.

Table 9-11: CM/PM Maintenance Trade-off Model Results for Young Pavements ($R^2 = 0.14$)

Variable/Constant	Coefficient Estimate	t-statistic
A	3.8866	1.2547
B	0.5582	2.2544
C	0.7632	3.6643

Table 9-12: CM/PM Maintenance Trade-off Model Results for “Old” Pavements ($R^2 = 0.38$)

Variable/Constant	Coefficient Estimate	t-statistic
A	39.0885	1.6858
B	0.7601	1.3521
C	0.1978	2.6524

The obtained models show that increasing levels of preventive maintenance translate to reduced corrective maintenance in the subsequent years. For a given level of preventive maintenance expenditure, mid-age pavements were afforded relatively greater effectiveness (reduction in levels of subsequent corrective maintenance) compared to young pavements. The fit for mid-age pavements was relatively good, while that for young pavements was relatively poor, indicating that the variability in the effectiveness of preventive maintenance is higher for young pavements than it is for older pavements. A plot of the fitted values is shown as Figure 9-30.

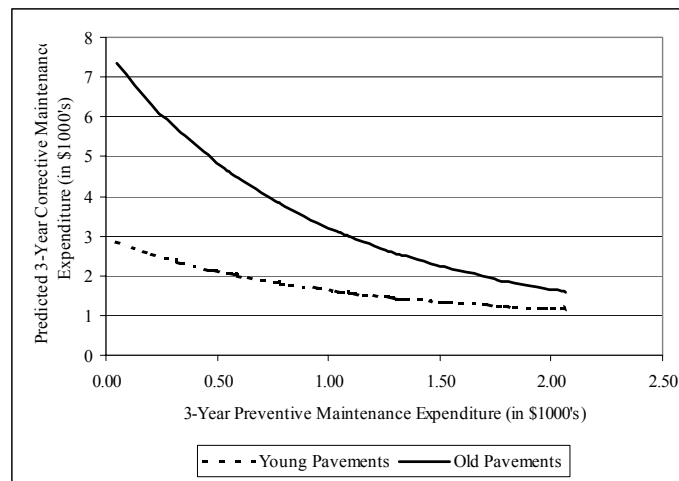


Figure 9-30: Fitted Values for Preventive Maintenance/Corrective Maintenance Trade-Off Model

9.5 Marginal Effects of Preventive and Corrective Maintenance Modeling

The previous section established the existence of a trade-off between preventive maintenance and corrective maintenance. It was shown that higher levels of preventive maintenance during a period generally correspond to reduced levels of corrective maintenance at subsequent periods. This was observed for pavements in both early and advanced phases of their typical service lives. It can also be hypothesized that the marginal effect of preventive maintenance expenditure on corrective maintenance varies in a manner that depends on the level of preventive maintenance. To investigate this variation, marginal effects models were developed by determining the elasticities of the trade-off functions, and plotting fitted values of the resulting models. The model functional form and specification are discussed in Chapter 5.

9.5.1 Model Results

The marginal effects curves in Figures 9-31 and 9-32 indicate large and positive elasticities. For pavements in their early lives, increasing the level of preventive maintenance by 1% when it is at a value of \$500 per lane-mile yields a 27% reduction in subsequent corrective maintenance (Figure 9-31). However, at \$2,500 per lane-mile, a \$1000 increment in preventive maintenance is associated with a 180% reduction in subsequent corrective maintenance. For pavements in their advanced phases of service life, the potential benefits per unit increase in preventive maintenance expenditure is even higher, as seen from Figure 9-32.

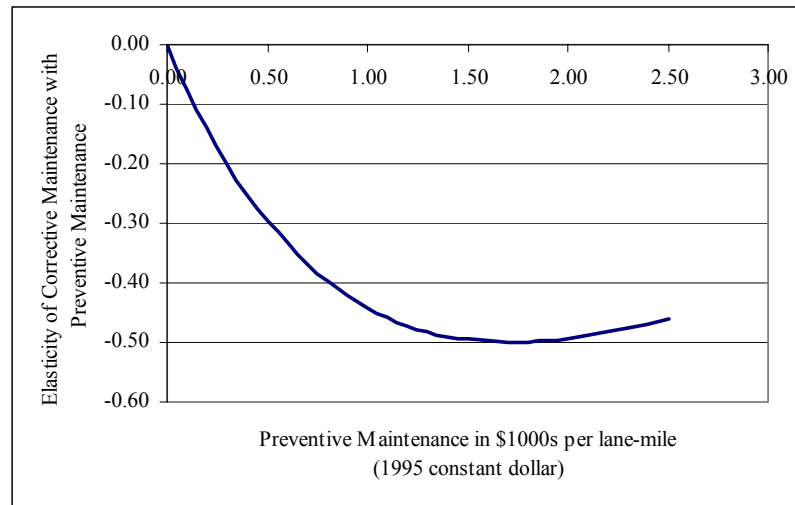


Figure 9-31: Marginal Effects Curve for Corrective and Preventive Maintenance, “Young” Pavements

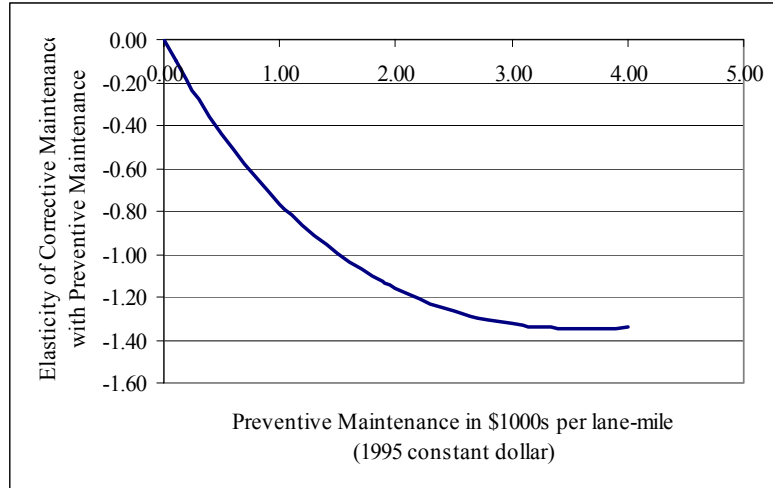


Figure 9-32: Marginal Effects Curve for Corrective and Preventive Maintenance, for “Old” Pavements

The trade-off and marginal effects functions between preventive and corrective maintenance enable the determination of changes in corrective maintenance in response to preventive maintenance in a past 3-year period. Increased preventive maintenance leads to decreased corrective maintenance, and obviously, decreased overall maintenance. The preventive/corrective maintenance trade-off relationship was useful in the evaluation of maintenance in the long-term (Chapter 8) where each maintenance scenario consisted of a preventive maintenance strategy and a default set of corrective or “remedial” preventive maintenance treatments.

9.6 Chapter Summary

In this chapter, it was found that trade-off relationships exist not only between rehabilitation intervals and average annual maintenance expenditure, but also between rehabilitation interval and traffic loading as well as weather severity. For all pavement families, increased maintenance expenditure translates to increased rehabilitation interval. On the other hand, increasing levels of traffic loading and weather severity result in reduction of rehabilitation interval. Marginal effects of maintenance expenditure, traffic loading, and weather severity were determined using the concept of elasticity. The marginal effects models demonstrated that the extent to which rehabilitation interval changes with changes in pavement attributes

varies considerably from one pavement family to another, implying that the pavement type and especially, road functional class, are important players in the overall equation. Furthermore, the marginal effects models make it possible to determine the effect of unit changes in maintenance levels, traffic loading, and weather on changes in rehabilitation interval on a project or network level. This information is useful not only for pavement management, but also for policy formulation and analyses such as truck weight policy, and for assessment of changing needs for pavement M&R in response to changing traffic and weather conditions in the long-term. Trade-off and marginal effects models were also developed for preventive maintenance and subsequent corrective maintenance, and showed that substantial reduction in corrective maintenance are achieved for unit increases in preventive maintenance levels.

CHAPTER 10: CASE STUDIES IN PAVEMENT MAINTENANCE EFFECTIVENESS AND DISCUSSION OF MODEL ERROR SOURCES

This chapter discusses maintenance effectiveness on various pavement sections in Indiana that are especially noted for their long service lives. One section for each pavement family was selected for discussion. This discussion is carried out to illustrate the impact of pavement maintenance on pavement longevity, especially as the maintenance levels for a selected pavement in a given family is higher than the typical maintenance values for pavements in that pavement family. The chapter also discusses the sources of error associated with the various models.

10.1 Case Studies

10.1.2 Rigid Interstate: Interstate 65, Reference Post 260+0.41 to Reference Post 261+0.27, Lake County

This highway section, a vital link in the Gary-Chicago–Milwaukee (GCM) transportation corridor, connects Interstate 90 Toll Road to Interstate 80/94 (Borman Expressway). Located in an area of severe weather (average annual freeze index of 783 degree days), and having high traffic levels (over 500,000 ESALs per year), this 4-lane pavement section has nevertheless managed to survive for 32 years without any resurfacing. Since 1969, when the pavement was newly constructed, it has only received a variety of preventive and corrective maintenance treatments mostly in the form of joint and crack sealing, shallow and deep patching, and underdrain maintenance. Over the past 15 years, average annual maintenance expenditure on this pavement section has been over \$3500 in 1995 constant dollars, a rather high value compared to similar expenditures on jointed concrete pavements under similar conditions in the state. The service life of this pavement (32 years) is approximately 15% higher than the typical service life of rigid Interstate pavements under similar conditions of loading and weather elsewhere in the state..

10.1.2 Rigid Non-Interstate: State Road 37, Reference Post 157+0.46 to Reference Post 158+0.09,

Marion County

This pavement was constructed in 1958 with jointed reinforced concrete slabs laid on a 12-inch aggregate base. An important six-lane urban principal arterial that helps connect south central Indiana (including the cities of Bloomington and Bedford) to Indianapolis and beyond, this pavement carries about 4,000 vehicles on a typical day (and about 150,000 ESALs per year). The climate in the region is characterized by considerable freeze, freeze-thaw and precipitation. The freeze index and number of freeze-thaw cycles are 519 degree-days and 59 respectively, while annual precipitation is about 40 inches per year. Over its entire life, this pavement section has carried a total of 8.1 million cumulative ESALs, a figure that is considered very large from the viewpoint of pavement design. Typical maintenance treatments that have been administered to this pavement in its life-span include joint and crack sealing, underdrain maintenance and shallow patching. The estimated average annual maintenance expenditure for this pavement was approximately about \$3,254 per lane-mile, in terms of 1995 dollars, in the past 10 years.

10.1.3 Overlay Interstate: Interstate 65, Reference Post 109+0.96 to Reference Post 110+0.72,

Marion County

Located in Marion County, this 6-lane overlay Interstate pavement section is an important link in the dense transportation network in central Indiana. The original pavement was constructed as a rigid PCC pavement in 1950's, and was rehabilitated in 1976 with a 6-inch thick asphaltic concrete resurfacing layer. The pavement currently carries over 1 million ESALs per year, and has experienced over 15 million ESALs since the time of its rehabilitation in 1976. The region in which this pavement is located is known for a significant amount of freeze conditions (519 degree-days per year), freeze-thaw cycles (59 per year), and precipitation (over 40 inches per year). Since 1976, dominant maintenance treatment that have been carried out on this pavement either in-house or by-contract, include crack sealing, underdrain maintenance, thin overlay, and shallow patching. The average annual expenditure of such treatment has been over \$2,123 (in 1995 constant dollar) per lane-mile, about \$355 higher than average expenditure for similar sections in this pavement family.

10.1.4 Overlay Non-Interstates: State Road 49, Reference Post 43+0.64 to Reference Post 44+0.15, Porter County

This highway section is part of the 4-lane principal arterial link that connects US Road 6 (GAR Highway) to Interstate 90 Toll Road. This pavement was originally constructed as a PCC pavement in 1967, and was resurfaced in 1974 using a 4-inch thick asphaltic concrete layer under a rehabilitation project. The pavement section, located in a predominantly urban area in Laporte highway district in northern Indiana, experiences rather high traffic levels and severe weather. Since 1994, this pavement section has experienced over 5 million cumulative ESALs. Annual freeze indices are as high as 764 degree-days, and the region is also subject to annual non-winter precipitation and freeze-thaw cycles of 38 inches and 52, respectively. Also, the region experiences one of the highest levels of winter precipitation (snowfall) in the state. The 1990's saw a sharp upsurge in maintenance expenditure for this pavement, with average annual expenditures as high as \$21,000 per lane-mile per year, in terms of 1995 dollars. These funds have been expended on treatments such as joint-bump repair, crack sealing, shallow patching, and underdrain maintenance. It can be argued that the long life of this pavement is largely attributable to the high levels of maintenance expended on this pavement section. However, one wonders whether it might not have been a better option to carry out rehabilitation in early 1990's, as that would have obviated the subsequent high annual maintenance expenditures of the mid 1990's.

10.1.5 Full-depth Asphalt Highways: State Road 13, Reference Post 137+0.16 to Reference Post 137+0.94, Elkhart County

Located in northeastern Indiana, near the cities of Elkhart and Goshen, this section of the 2-lane State Road 13 highway links US Road 6 and US Road 20 to Interstate 90 Toll Road, and ultimately to St. Joseph County in Michigan. Given the poor nature of the subgrade in the region and near saturated ground conditions, coupled with rather high traffic levels for a rural minor collector (5,443 vehicles per day and 230,000 ESALs per year), and the severity of weather at that location (the highest freeze index in the state), a long service life is not expected of this road pavement. Therefore the fact that this pavement has been able to hold its own for 21 years is indeed remarkable. This is probably due to the vigilance of pavement managers in that sub-district who obviously administer prompt preventive pavement maintenance as well as corrective maintenance.

Average annual maintenance levels have hovered around \$2500 per lane-mile, in 1995 dollar value, a rather high figure compared to that for other full-depth asphalt pavements in similar conditions.

10.2 Discussion of Model Error Sources for Various Models Developed in the Present Study

Human errors in measurement of the amount of work done, costing, monitoring measurements of pavement condition, or categorization of type of work done, are typically encountered in studies of this kind. An example of measurement error is associated with the conduction of maintenance at the edges of the road carriageway, which does not really affect roughness measurements because of the limited transverse coverage of the roughness bar.

Another source of error is the life of individual treatments. Methods used in the short-term effectiveness evaluation assume that treatments effectiveness last for at least one year. With this assumption, the use of yearly pavement condition measurements may suffice. However, in reality, the life of certain treatments such as pothole patching is often less than a year. Therefore, any benefits accrued to pavement condition may not be captured using yearly condition measurements. In consequence, the benefits of certain short-lived maintenance treatments may be grossly underestimated, if not occluded.

Systematic errors in the measurement of pavement condition are inherent with the procedure of pavement condition monitoring used in Indiana: INDOT monitors only first 500 feet of every mile, and assigns that condition to the entire mile. If the condition on the first 500 feet is not representative of the remaining distance along that mile, errors are likely to be encountered in any model that utilizes such data. Furthermore, changes in condition measurement equipment types and vendors in the last decade have resulted in possible errors and inconsistencies in such values. The issue of errors inherent with the computation of PSI from roughness measurements has been discussed in Chapter 5.

Pavement maintenance work carried out by field crew at sub-district level may be erroneously entered in maintenance records under categories other than those for pavement maintenance, and may therefore be missed during data collation. If the amount of work involved in such misclassifications is relatively significant, this oversight could be costly in terms of model accuracy.

To reflect changing time value of money, all costs and expenditure were brought to a common value using maintenance indices that pertain to the entire country [FHWA (2), 1999]. However, it is obvious that differences in average national price trends vis-à-vis Indiana price trends may be another source of error in the effort to bring all costs to a constant dollar value. The national price trends are values that have been average for all states. Some states may have rates of price increases that increase relatively sharply, while others may have rates that increase relatively gently.

The computation of weather severity levels assumes that weather conditions (precipitation, freeze-thaw cycles, and freeze indices) do not change within each county as such data was obtained for each county and assigned to all pavements in that county. However, in reality, there may be some differences in county-wide weather features to cause differences in pavement condition, all else being equal. Also, the computation of traffic loading using ESALs is associated with several assumptions. These include the assignment of a certain pavement thickness or structural number and terminal serviceability index to the pavement section whose ESAL is being determined. In the real situation, the assigned values of these parameters may be different from the real situation, and may result in over- or underestimation of traffic loading.

Another source of error is the lack of a consistent causal relationship between maintenance and pavement condition. In other words, a pavement needing maintenance does not always receive it due to funding limitations. On the other extreme, a pavement that is in relatively good condition may receive maintenance due to non-technical reasons. Modeling maintenance impact using such pavement sections is likely to yield unintuitive parameter estimates and poor values of statistical correlation. This is a likely reason for the generally relatively low values of R-square obtained in some of the models developed in this study.

CHAPTER 11: SUMMARY AND CONCLUSIONS

11.1 Overall Summary and Discussion

The study found that there are significant benefits associated with maintenance treatments, and that the short-term benefits generally involve an increase in pavement condition or a decrease in the rate of deterioration. Table 11-1 provides a synthesis of the average values of treatment effectiveness as well as the developed models that estimate effectiveness as functions of pavement and treatment attributes). For most treatments, a greater benefit is obtained for a larger effort expended in the maintenance treatment, at a given level of pavement condition. Also, a greater benefit is generally accrued for pavements in poor condition compared to those in fair condition, at a given level of maintenance. For the accomplishment costs of various maintenance treatments, the study determined mean cost values and also developed accomplishment costs models as functions of treatment type, pavement location, and pavement characteristics (Table 11.2).

The results of the short term impacts analyses were used as inputs in the evaluation on long-term maintenance impacts. Coupled with due consideration of work zone user costs, development of various treatment cost and effectiveness models provided a basis to evaluate the cost effectiveness of pre-defined preventive maintenance long-term strategies for each pavement family. Effectiveness was measured as the area under the performance curve, a surrogate for non-work zone user costs. It was found that there are significant differences in cost-effectiveness for various strategies, and the most cost-effective strategies, which were determined for each pavement family (Table 11-3), were found to be generally different from strategies currently being practiced at sub-districts and districts in Indiana. Plots and models of life-cycle cost-effectiveness versus preventive maintenance expenditure and marginal effects thereof, provided insights into the

long-term implications of alternative pavement maintenance practices. Thus, for each pavement family, “optimal” amounts of annualized preventive maintenance expenditure (that correspond to highest life-cycle cost effectiveness) were determined (Table 11-4).

The level of maintenance expenditure typically associated with various pavement types and functional classes is often of interest to planner and programmers. The present study utilized data from Indiana to develop models that enable estimation of expected annual maintenance pavement expenditure given its age, functional class, surface type and other characteristics (Table 11-5).

The study also found that trade-off relationships exist between rehabilitation intervals and maintenance, traffic loading, and weather (Table 11-6), a result that has profound implications in asset management functions involving analysis of relationships between various DOT programs (in this case, capital improvements vs. maintenance). For all pavement families, increasing maintenance leads to increased rehabilitation interval, while increasing traffic loads and weather severity leads to reduction in rehabilitation interval. However, the trade-off marginal effects analyses showed that the extent to which rehabilitation interval changes with these pavement attributes varies considerably from one pavement family to another, implying that the pavement type and especially, road functional class are important factors in such trade-of analyses. The marginal effects models make it possible to determine the effect of unit changes in maintenance levels, traffic loading, and weather on changes in rehabilitation interval on a project or network level. This information is useful not only for asset and pavement management, but also for policy analyses of such issues as truck weight and assessment of pavement repair needs in response to changing traffic and weather conditions in the long-term. Trade-off and marginal effects models were also developed for preventive maintenance and subsequent corrective maintenance, and showed that substantial reductions in corrective maintenance are achieved when levels of preventive maintenance are increased.

Table 11-1: Short-term Effectiveness Models for Selected Maintenance Treatments

Treatment	Measure of Effectiveness	Model		Average value
Thin HMA overlay	PJ	$\frac{71.63}{42.01 + (10^{-5.11} * 97.17^{IPC})}$		0.93
Micro-surfacing	PJ	-		0.76
Seal Coating	PJ	Linear	$1.158 - 0.275 JPC$	0.22
		Intrinsically Linear	$1.36 * e^{(IPC-1.8)^{1.408}}$	
	DRR	Linear	$-0.159 RT + 0.998 IPC$	3.42
		Intrinsically Linear	$e^{0.335 * IPC}$	
		Non-linear	$e^{1/(8.79 - 7.02 * 1.037 IPC)}$	
Crack Sealing	DRR	$1/[-12.08 - 12.99FC + 6.61TL + 1.47IPC + 0.09 WD]$		0.177
Crumb Rubber Sealing	DRR	$e^{-123.21 + 12.78TL + 2.63PPN - 0.025Q + 62.28EXP}$		0.318
Bump Grinding	PJ	$0.087 + 1.32 EXP$		0.202

FC – Functional Class

TL – Traffic Loading

WD – Number of wet days a year

PPN – Precipitation (inches per year)

EXP – Treatment Expenditure

RT – Route Type

SQ – A measure of subgrade quality (product of percentage of fines and plasticity index)

IPC – Initial Pavement Condition (i.e. condition before treatment)

Table 11-2: Unit Accomplishment Costs of Maintenance Treatments (\$1995)

	Treatment Type	Units	Mean
“Minor” Preventive Maintenance	Crack Sealing	Lane-miles	444.19
	Crumb Rubber Sealing	Lane-miles	714.21
	Joint/Bump Repair	Number	73.00
	Under-drain Maintenance	Number	5.81
“Moderate” Preventive Maintenance	Seal Coating	Lane-miles	4799.69
“Major” Preventive Maintenance	Micro-surfacing	Lane-miles	27,434
	Thin HMA Overlay	Lane-miles	61,664
Corrective Maintenance	Shallow Patching	Tons	302.54
	Deep Patching	Tons	227.46
	Premix Leveling	Tons	70.54

Table 11-3: Cost Effective Long-Term Preventive Maintenance Strategies for Various Pavement Families

Pavement Families	Suggested Strategy
Full Depth Asphalt Non-Interstates	<ul style="list-style-type: none"> • Chip sealing every 6 years after resurfacing or reconstruction • Crack sealing every 3 years after reconstruction, resurfacing or chip sealing • Default (typically corrective) activities to be carried out as needed: <ul style="list-style-type: none"> - Shallow Patching, Deep Patching, Bump Grinding.
Rigid (PCC) Interstates	<ul style="list-style-type: none"> • Cracks and joint sealing 8 years after reconstruction, and every 8 years thereafter, • Under-drain maintenance every year, • Default (typically corrective) activities to be carried out as needed: <ul style="list-style-type: none"> - Shallow Patching, Deep Patching, Premix Leveling, Fault Grinding, Under-sealing, and Stitching.
Rigid (PCC) Non-Interstates	<ul style="list-style-type: none"> • Cracks and joint sealing 7 years after reconstruction, and every 7 years thereafter, • Under-drain maintenance every year, • Default (typically corrective) activities to be carried out as needed: <ul style="list-style-type: none"> - Shallow Patching, Deep Patching, Premix Leveling, Fault Grinding, Under-sealing, and Stitching.
Composite (AC-on-PC) Interstates	<ul style="list-style-type: none"> • Thin Overlay every 8 years after reconstruction/ rehabilitation and every six years thereafter, • Under-drain maintenance every year after reconstruction/ resurfacing/ thin overlay, • Default (typically corrective) activities to be carried out on a three-year cycle: <ul style="list-style-type: none"> - Shallow Patching, Deep Patching, Joint/Bump Repair.
High-volume Composite (AC-on-PC) Non-Interstates	<ul style="list-style-type: none"> • Thin Overlay every 5 years after resurfacing, • Crack sealing every 3 years after reconstruction/resurfacing/thin overlay, • Under-drain maintenance every year, • Default (typically corrective) activities to be carried out on a three-year cycle: <ul style="list-style-type: none"> - Shallow Patching, Deep Patching, Joint/Bump Repair.
Low-volume Composite (AC-on-PC) Non-Interstates	<ul style="list-style-type: none"> • Micro-surfacing every 8 years after resurfacing or reconstruction, • Chip sealing 4 years after micro-surfacing, • Crack sealing 2 years after micro-surfacing, • Default (typically corrective) activities to be carried out as needed: <ul style="list-style-type: none"> - Shallow Patching, Deep Patching, Joint/Bump Repair.

Table 11-4: Relationship Between Life Cycle Maintenance and Cost Effectiveness

Pavement Family	Relationship Equation	Suggested Level of Maintenance (Annualized, \$1995)
Full Depth Asphalt Non-Interstates	$2021904.8 X - 2021860.5 X^{1.000097}$	\$480 per lane-mi
Rigid (PCC) Interstates	$80001.1 X - 79990.9 X^{1.000115}$	\$1130 per lane-mi
Rigid (PCC) Non-Interstates	$221687.6 X - 221672.2 X^{1.000136}$	\$600 per lane-mi
Composite (AC-on-PC) Interstates	$501257.4 X - 501217.3 X^{1.000124}$	\$700 per lane-mi
High-volume Composite (AC-on-PC) Non-Interstate	$123011.5 X - 122985.5 X^{1.000150}$	\$1500 per lane-mi
Low-volume Composite (AC-on-PC) Non-Interstate	$229742.4 X - 229809.1 X^{1.000097}$	\$280 per lane-mi

Table 11-5: Average Annual Pavement Maintenance Expenditure Models

Pavement Type	Model
Full-depth AC Pavements	$4.7089 + 0.7926 INT_CLASS + 0.0265 AGE + 1.5780 WSL$
Overlay Pavements	$35.1424 - 4.52 INT_CLASS - 0.2252 AREA_CLSS - 0.3278 SOUTH_F + 0.2403 TRAD_F + 0.0364 AGE$
Rigid Pavements	$5.1100 + 1.102 INT_CLASS - 1.2012 SR_CLSS - 0.6428 AREA_CLSS - 1.0251 CONT_F + 0.0459 AGE$

INT_CLASS - 1 if pavement is an interstate pavement, 0 otherwise

AGE - Years since last rehabilitation or reconstruction/replacement, whichever is more recent

WSL - Weather Severity Index

AREA_CLSS - 1 if pavement is in rural area, 0 if urban

SOUTH_F - 1 if pavement is located in southern Indiana, 0 if otherwise

TRAD_F - 1 if pavement is a traditional overlay, 0 if otherwise

SR_CLSS - 1 if pavement is on a state road, 0 if otherwise

CONT_F - 1 if pavement is a CRC, 0 if JRC or JPC

Table 11-6: Relationships between Maintenance and Frequency of Capital Investment (Resurfacing)

Pavement Family	Relationship Equation
Rigid (PCC) Interstates	$Resurfacing\ Interval = 34.0793 - 19.8293 * 0.2410^{M/CLW}$
Rigid (PCC) Non-Interstates	$Resurfacing\ Interval = 39.0608 - 27.0552 * 0.9281^{M/CLW}$
Composite (AC-on-PC) Interstates	$Resurfacing\ Interval = 37.5909 - 23.3918 * 0.5394^{M/CLW}$
Composite (AC-on-PC) Non-Interstates	$Resurfacing\ Interval = 61.8719 - 50.7040 * 0.9824^{M/CLW}$
Full-depth Asphalt Pavements	$Resurfacing\ Interval = 11.9249 - 8.2775 * 0.7678^{M/CLW}$

11.2 Implementation Issues

The products of this research are as follows:

- Models for short-term impacts of maintenance, that are necessary as input data in pavement management system software for adjustment of deterioration curves to reflect conduction of specific maintenance treatments,
- Models and average values of annual maintenance expenditure for various pavement surface types, for use in state-wide maintenance sketch planning and program development,
- Models for maintenance treatment accomplishment unit costs, useful for planning and budgeting at a sub-district level,

- A set of recommendations for selecting maintenance strategies that are associated with the highest cost-effectiveness, for each pavement family
- Models to predict or estimate the impacts of changing maintenance levels, weather or traffic loading levels on pavement longevity.

When properly implemented by operators of INDOT's pavement and maintenance management systems, and the divisions responsible for overall planning and budgeting for pavement maintenance and repair, it is expected that the results of this study will engender a significant shifts in pavement maintenance management and practices.

Results of the analysis of short-term effectiveness of maintenance treatments is not only useful for adjusting the performance curves in existing pavement management software, but also provides a useful guide to districts and sub-districts regarding the choice of appropriate treatments to address a given problem that would yield the maximum effectiveness in the short-term. This is particularly applicable to pavement maintenance activities carried out in-house on a force account basis. The most significant contribution of this research to the state of practice in pavement maintenance lies in the results of the long-term evaluation of maintenance strategies. The best maintenance strategies that have been determined for each pavement family enables the application by the districts and sub-districts, of consistent treatment and timing schedules for each pavement family to yield maximum cost-effectiveness. This will result in obtaining maximum return from each dollar of maintenance investment, and overall savings to the state in the long run, without sacrificing pavement performance. It is therefore expected that implementation of the results of this research will result in changes in maintenance and capital planning and programming. Finally, the trade-off and marginal effects models between frequency of capital investment (resurfacing, rehabilitation) and maintenance load and weather are useful for determining the impacts of shortfalls or increases in maintenance funding, changing levels of truck loading on state highway pavements, and continental changes in weather. Determination of such impacts is useful for trucking policy formulation and evaluation, long-term needs assessment of maintenance, and highway management in general.

It is envisioned that the positive impacts of improved practices will resonate at project and network levels, from top management to maintenance crew, and from state highway agencies to local street administrators. Implementation of any improvements to an existing pavement and maintenance management systems requires an effort at informing not only district and sub-district highway pavement managers and engineers but also state legislators and local policy makers, top-level highway executives, and indeed, the general public. While the findings of this research are especially applicable to states with similar highway characteristics as those of Indiana, methodologies built upon or developed in the present study can be applied by researchers and highway practitioners worldwide who seek to answer questions similar to those addressed in the present study.

Tools to facilitate implementation of the results of this research include training of personnel, and organization of workshops to demonstrate the cost effectiveness of maintenance, especially preventive maintenance. Possible impediments to successful implementation includes public perceptions (such as resistance to proactive maintenance conducted to address imminent, rather than obvious, distress), desire to minimize work zones, peculiar nature of budgeting procedures, and the fact that some management systems are have not been fully implemented. However, with close cooperation between concerned parties, improved public relations, and learning from the experience of agencies that have experimented with implementing different pavement management policies, the impact of such barriers to implementation of the study findings, can be reduced.

As most “baby boomers” approach retirement age, the strength and quality of the maintenance work force of pavement engineers, managers, supervisors and crew leaders are expected to suffer. In this regard, the implementation of guidelines for optimal timing of preventive maintenance treatments would enable the various districts and sub-districts to carry out rational and consistent practices of pavement preservation and will help to obviate the effects of knowledge “gaps” resulting from staff turnover. *All other factors being equal, the effectiveness of maintenance either in the short term or in respect to its pavement life extension advantages are greatly enhances if skilled maintenance personnel at all levels are available. As such, there is a vital need for INDOT to preserve the continuity of such personnel at district and sub-district level.*

11.3 Future Work

The present study utilized past observed data on maintenance effectiveness and expenditure to carry out the various analyses. A major limitation of observational data is that they may not always provide adequate information about the cause-effect relationships [Neter et al., 1990]. For example, a positive relation between two variables may not imply that one is a direct result of the other. In this regard, it can be argued that the effectiveness of maintenance treatments in the short-term is best carried out with the aid of controlled field experiments. A crack sealing effectiveness evaluation project, an example of such research, is currently being conducted by INDOT/Purdue University's Joint Transportation Research Program (JTRP). Such experiments could be expanded to investigate the effectiveness of several other preventive and corrective maintenance treatments in the short term. Also, in the long term, field experiments could be carried out to evaluate the effectiveness of various maintenance strategies over the pavement life cycle, rather than just treatments, for each pavement family.

Future performance models could be preceded by meticulous acquisition of include subgrade data (e.g., as plasticity index, resilience modulus, and moisture content), as well as specific design and construction features (e.g., as presence of underdrains, dowels, etc.) for use in various models in lieu of using "grouping" variables such as Interstate/non-Interstate as a surrogate for such pavement features.

Other future work includes the evaluation of cost effectiveness of rehabilitation options over the entire life-cycle of a pavement. This could be carried out using observational data from INDOT's PMS and SHRP's LTPP SPS-2 database, or/and a carefully designed field experiment for the state.

Finally, as pavement research is a continuing effort, and various federal organizations, universities and research institutions, and industry are currently engaged in research towards identification and implementation of new methods and materials to enhance pavement longevity. Therefore, it is recommended that any future work on maintenance effectiveness should appropriately consider the findings of such research, and/or make use of database that have been developed as a by-product of such studies.

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APPENDIX A1

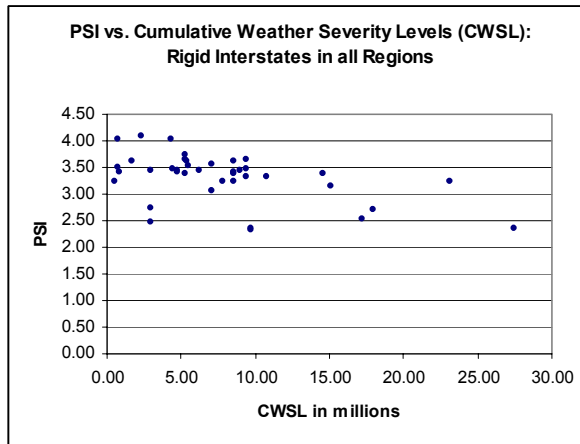
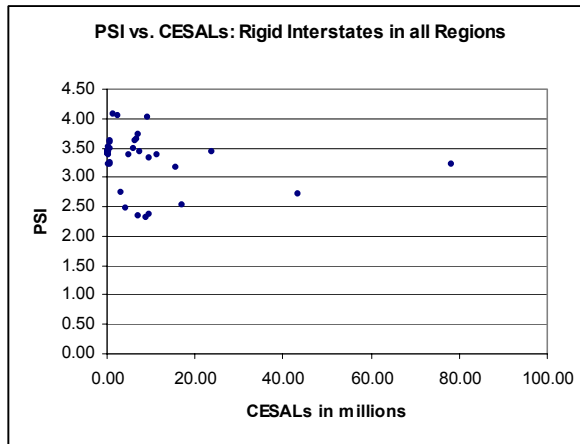
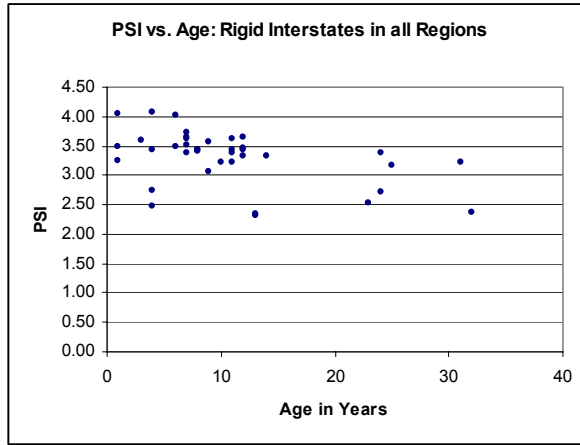


Figure A-1: Performance Trends for Rigid Interstates-All Regions (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX A2

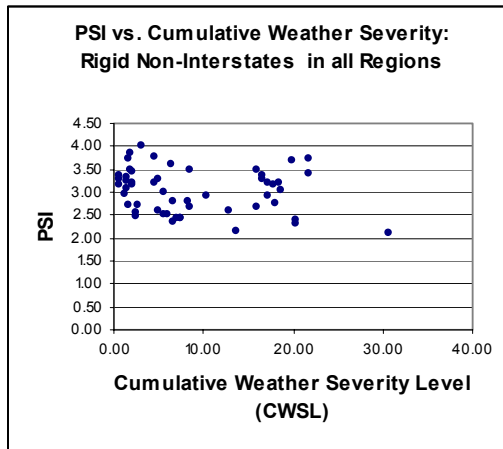
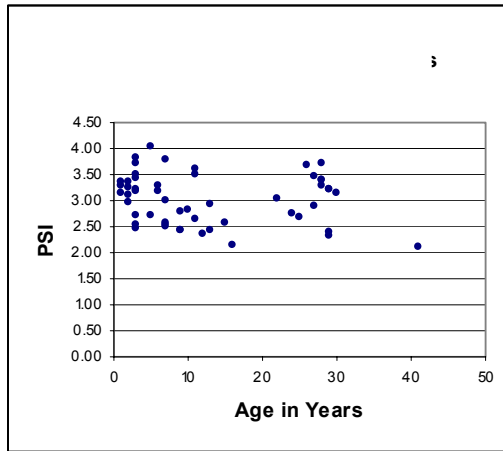
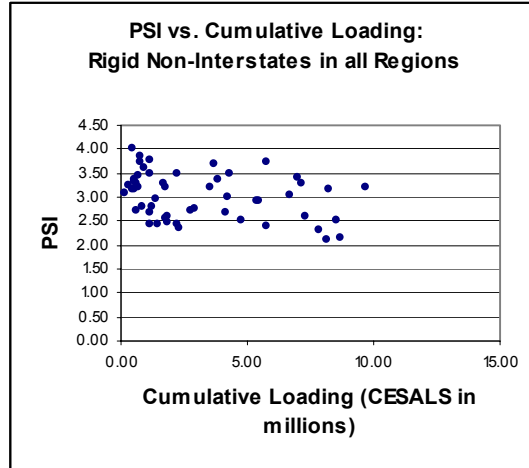


Figure A-2: Performance Trends For Rigid Non-Interstates-All Regions (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX A3

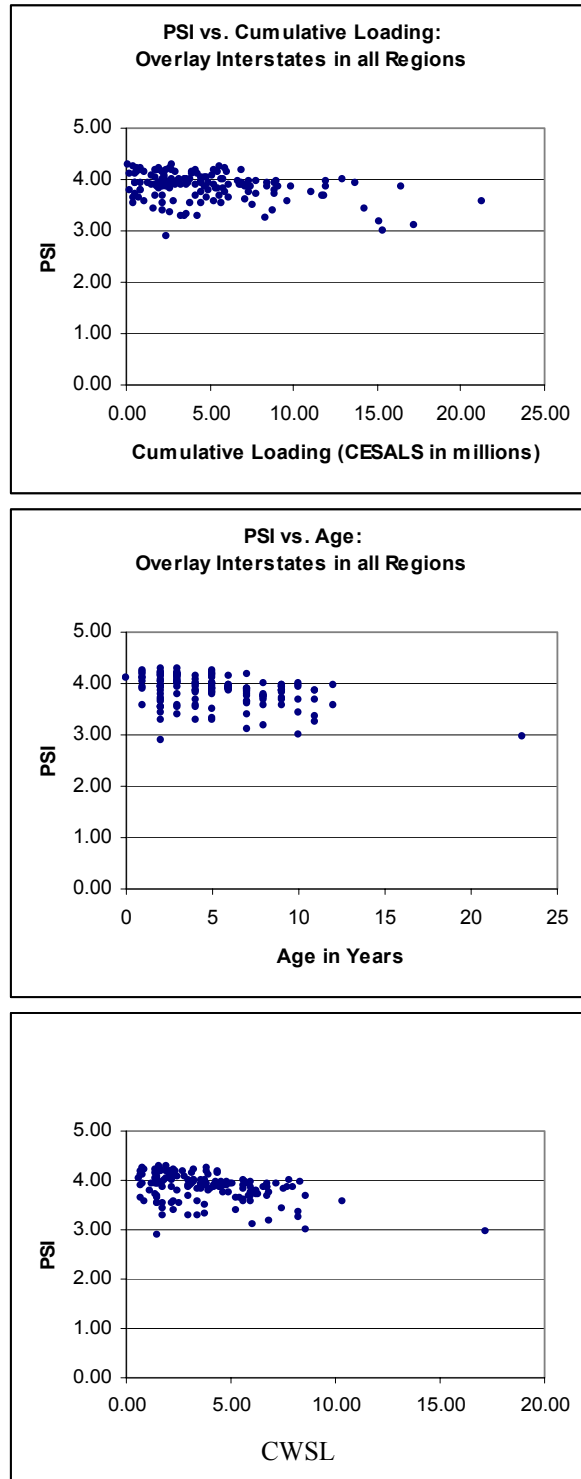


Figure A-3: Performance Trends For Overlay Interstates-All Regions (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX A4

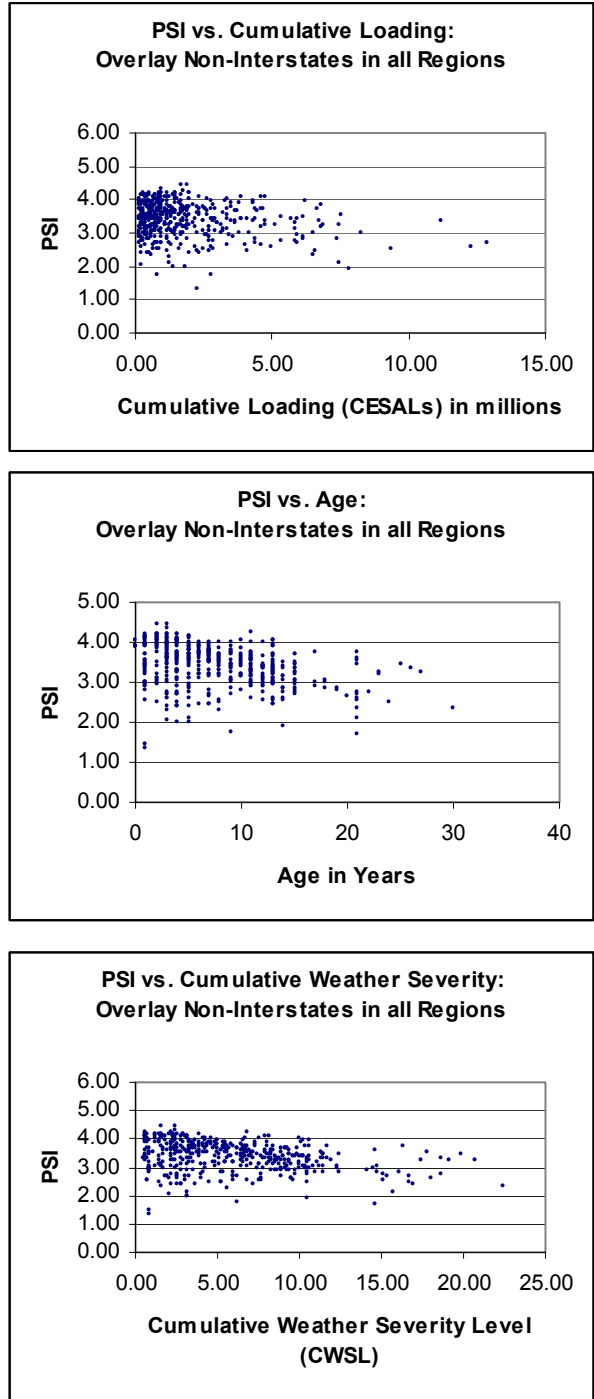


Figure A-4: Performance Trends For Overlay Non-Interstates- All Regions (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX A5

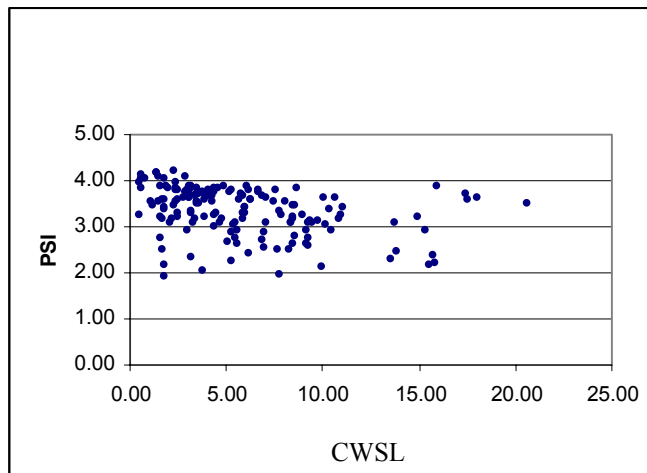
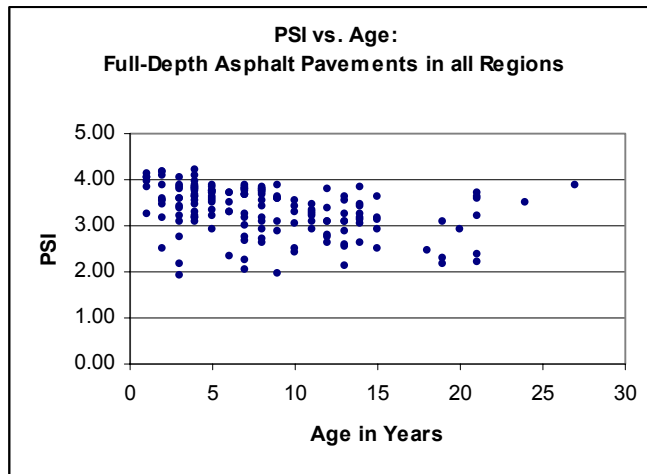
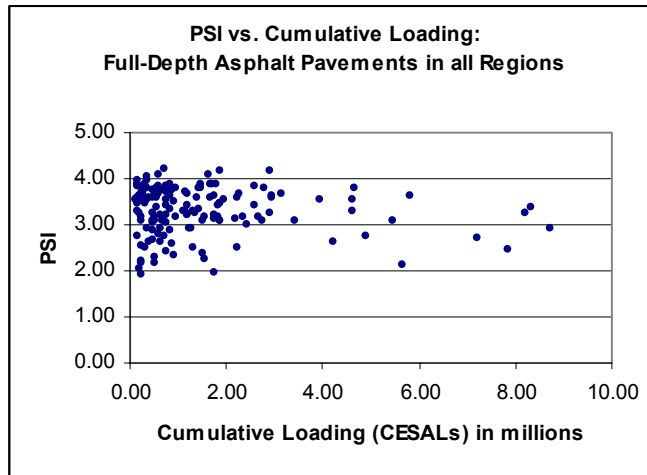


Figure A-5: Performance Trends for Full-Depth Asphalt Pavements- All Regions (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX B1

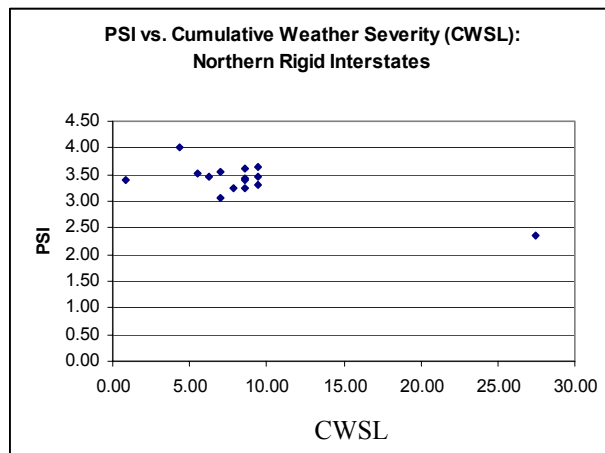
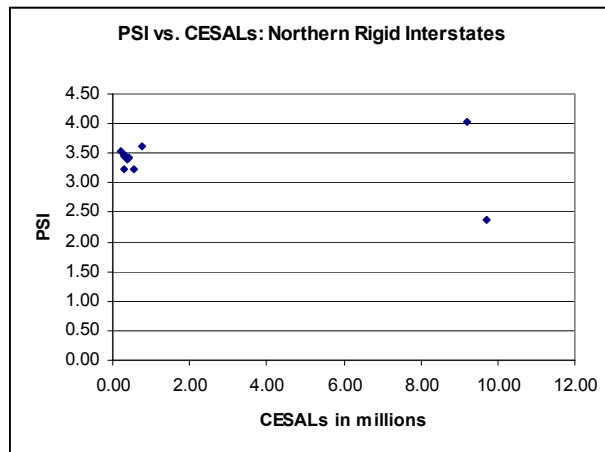
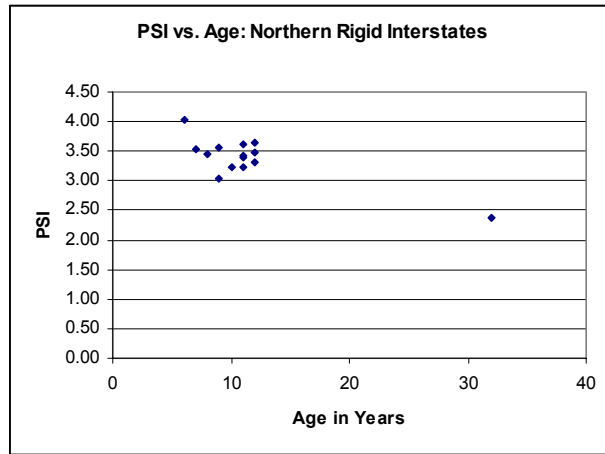


Figure A-6: Performance Trends for Rigid Interstates-Northern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX B2

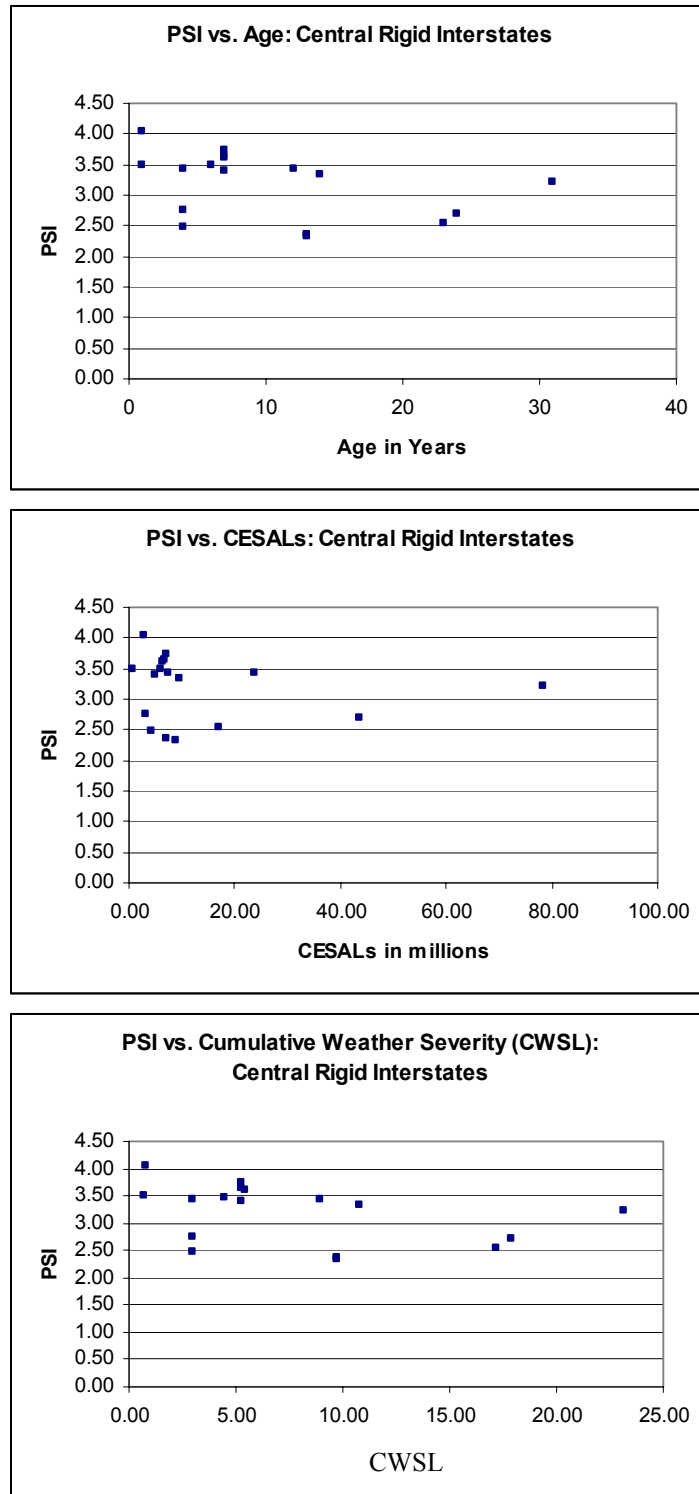


Figure A-7: Performance Trends for Rigid Interstates-Central Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX B3

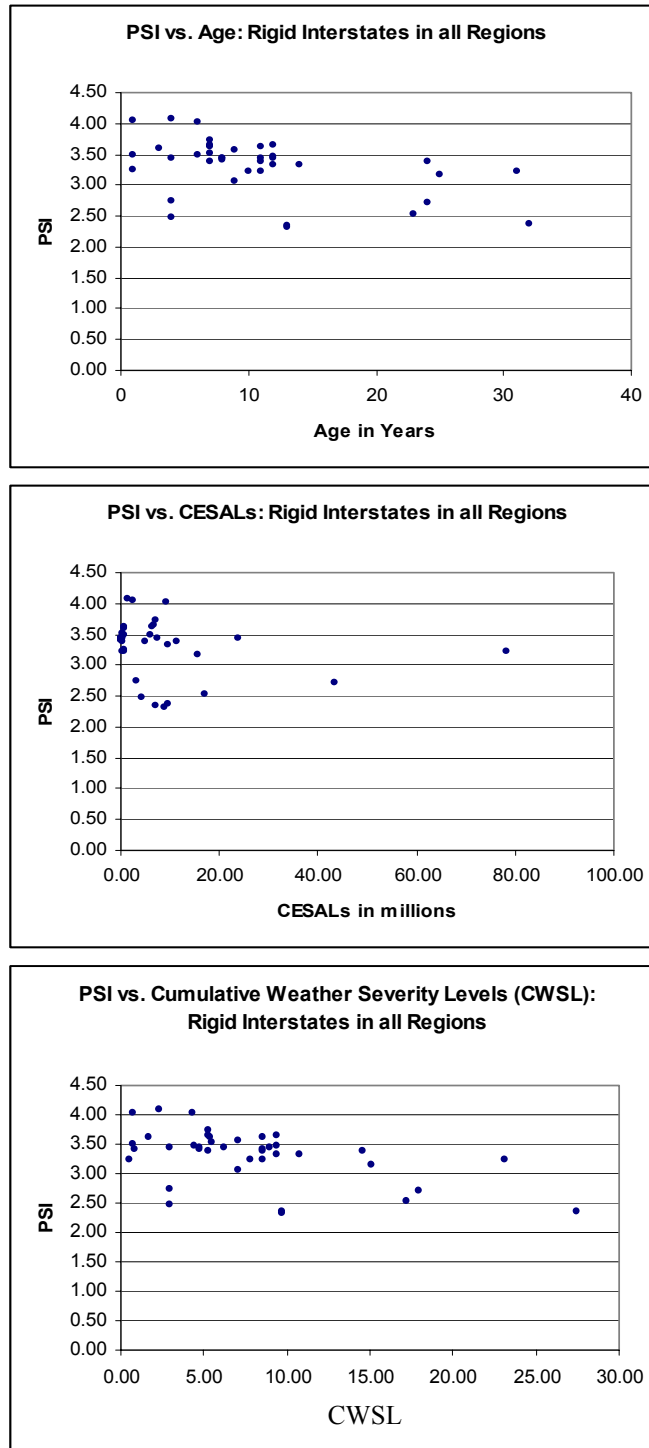


Figure A-8: Performance Trends for Rigid Interstates-Southern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX C1

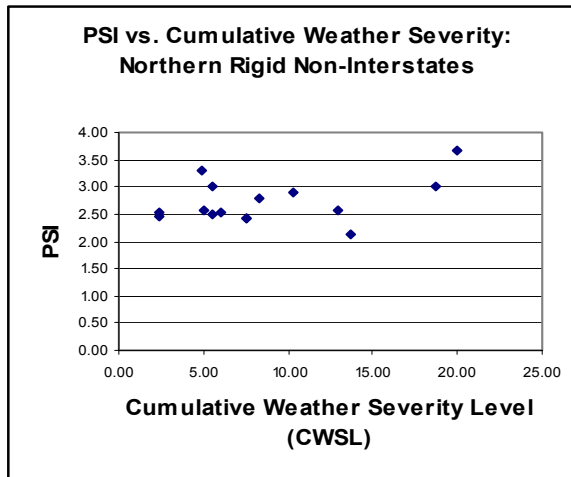
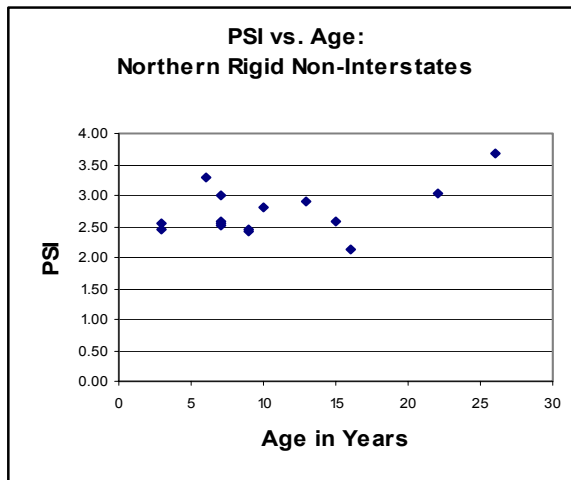
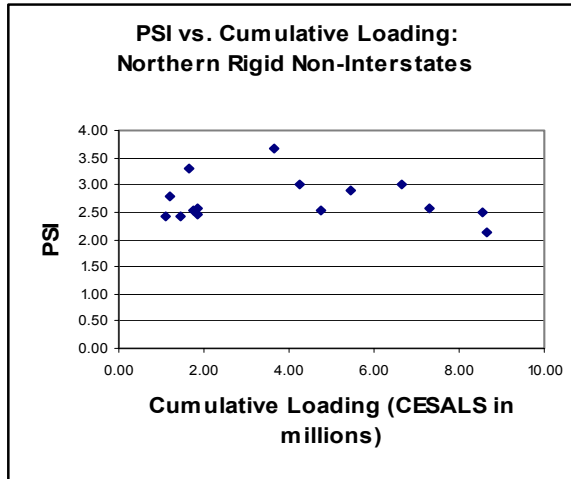


Figure A-9: Performance Trends for Rigid Non-Interstates- Northern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX C2

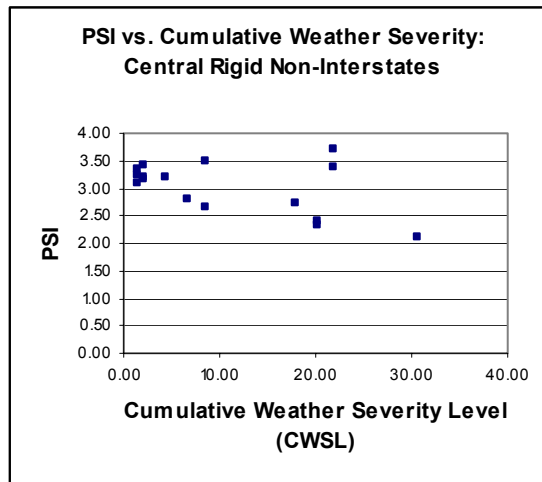
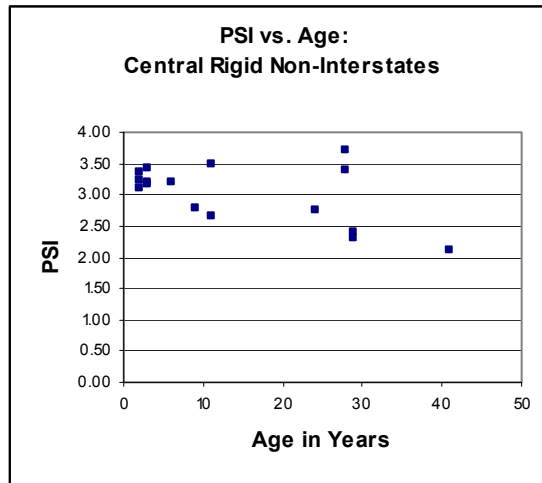
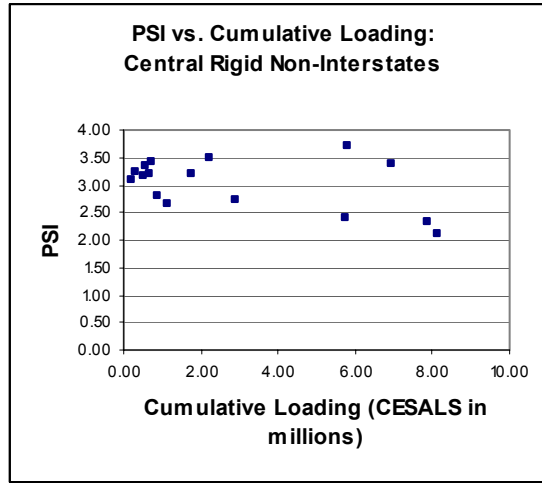


Figure A-10: Performance Trends for Rigid Non-Interstates- Central Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX C3

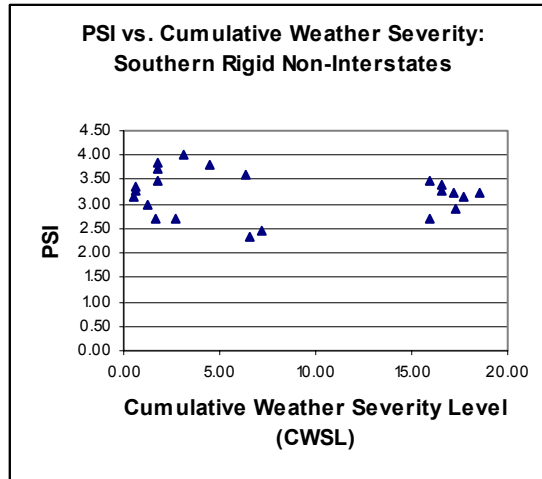
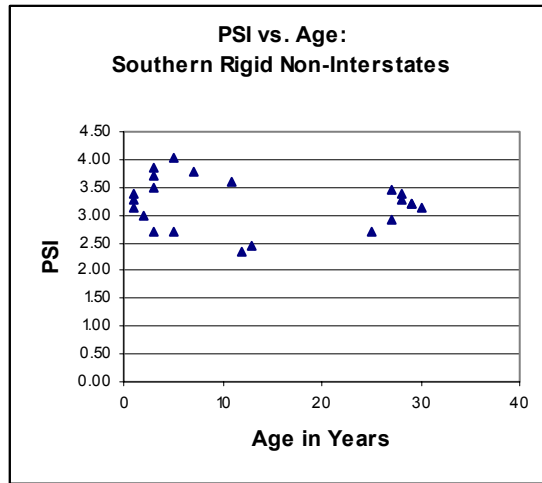
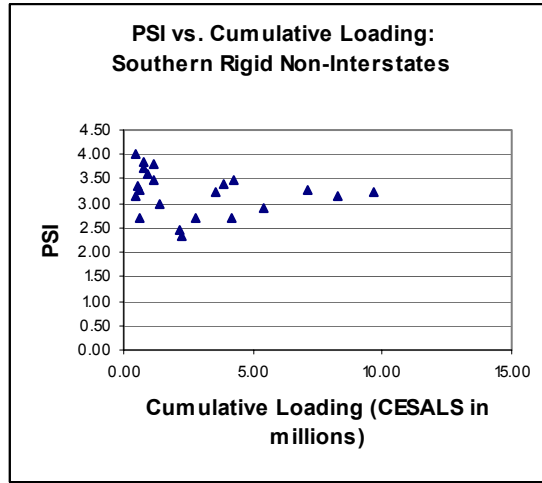


Figure A-11: Performance Trends for Rigid Non-Interstates- Southern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX D1

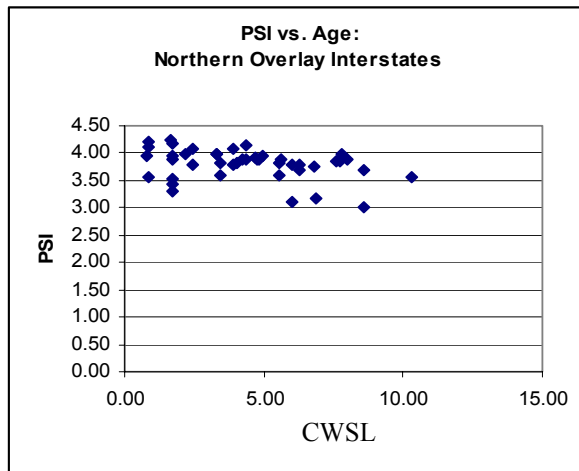
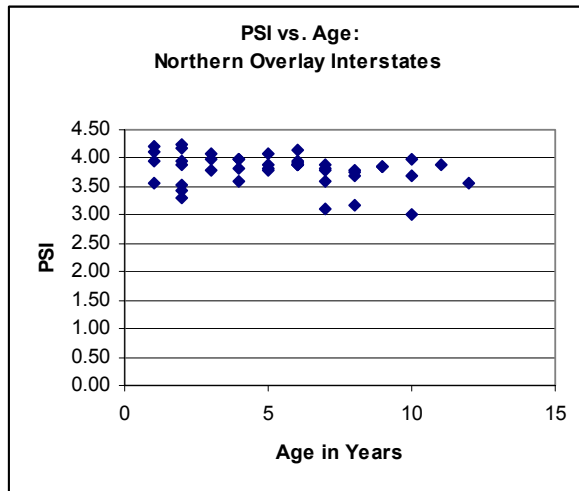
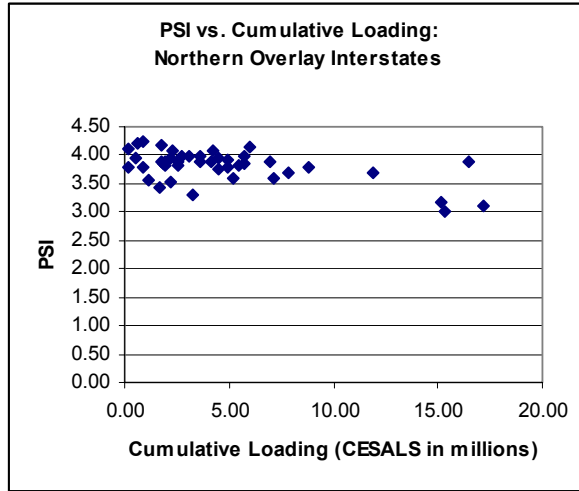


Figure A-12: Performance Trends for Overlay Interstates-Northern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX D2

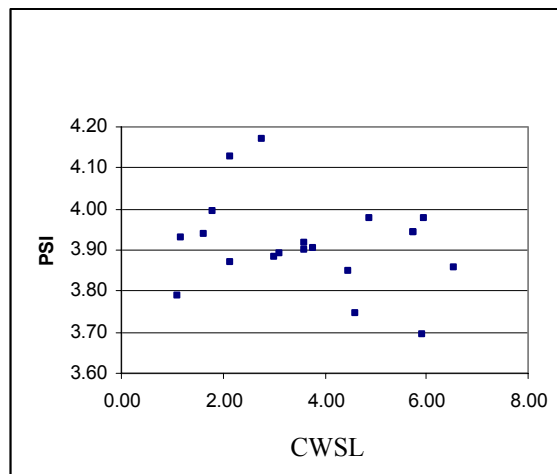
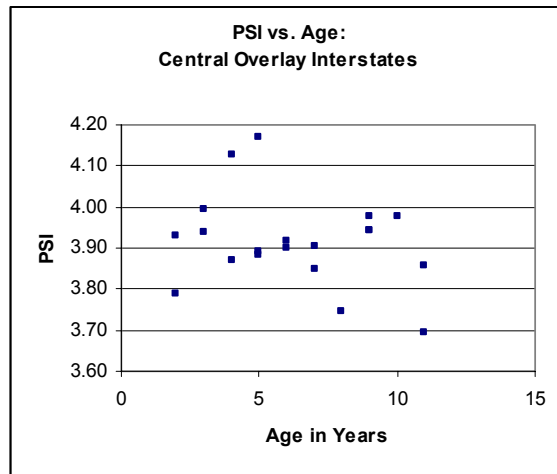
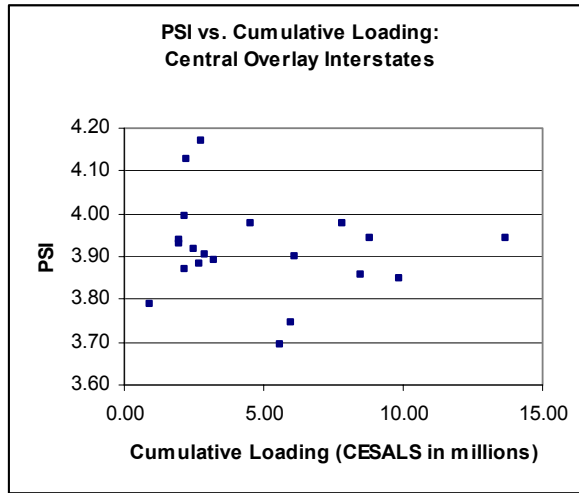


Figure A-13: Performance Trends for Overlay Interstates-Central Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX D3

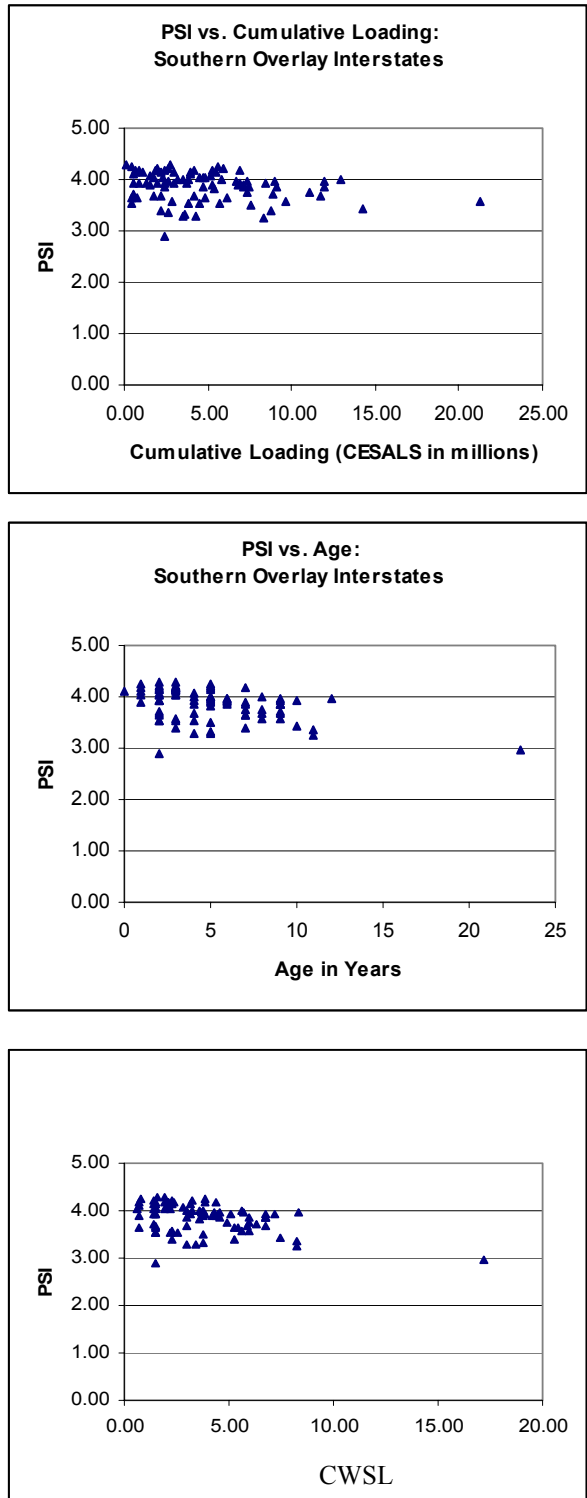


Figure A-14: Performance Trends for Overlay Interstates-Southern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX E1

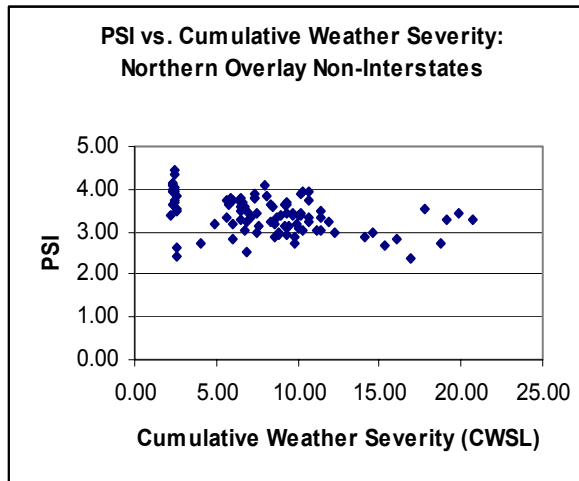
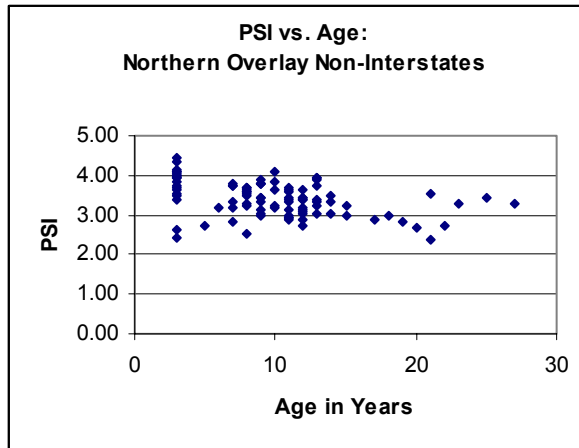
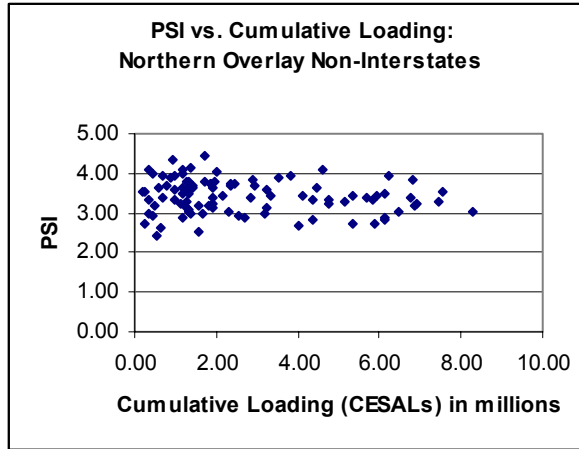


Figure A-15: Performance Trends for Overlay Non-Interstates-Northern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX E2

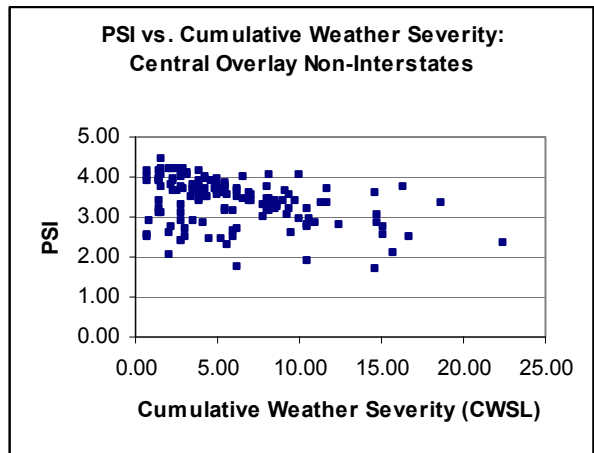
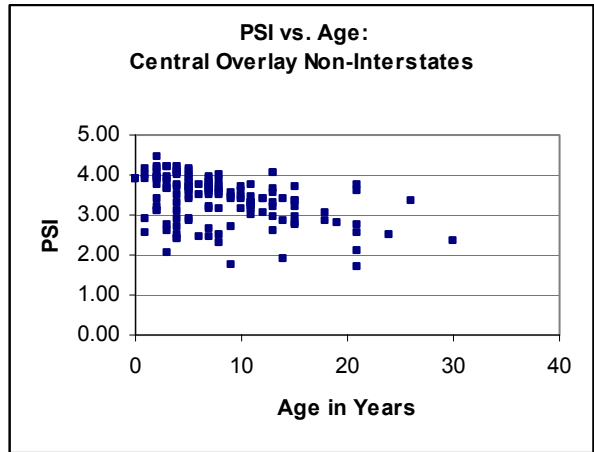
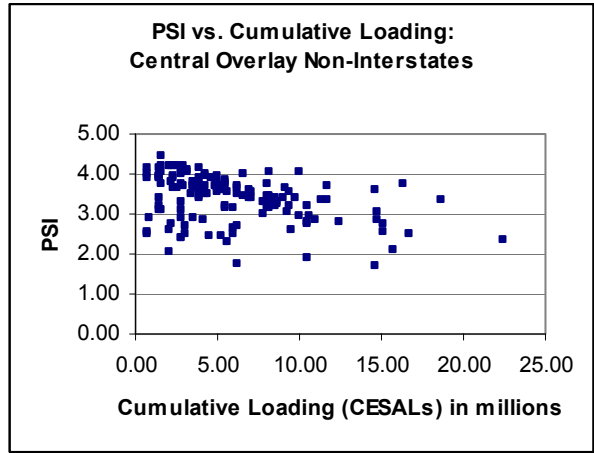


Figure A-16: Performance Trends for Overlay Non-Interstates- Central Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX E3

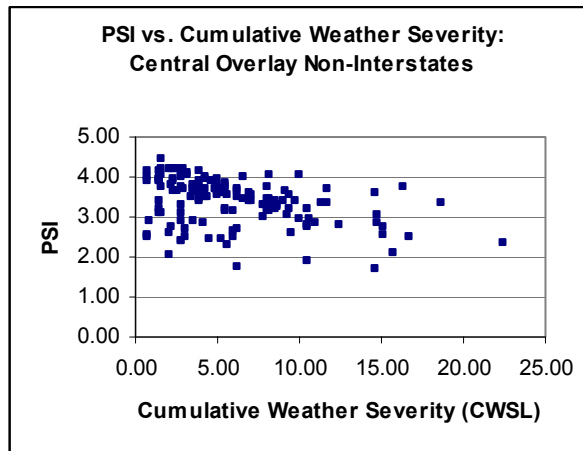
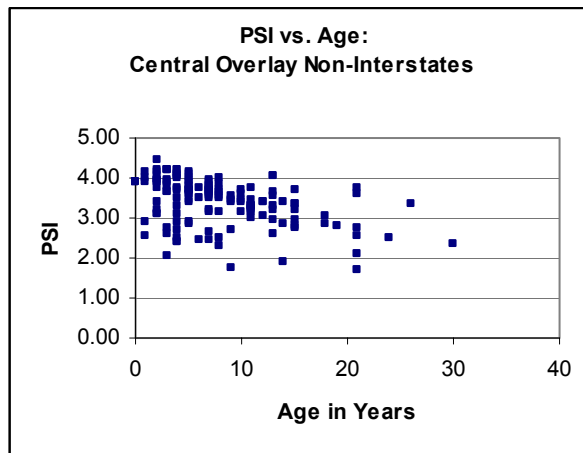
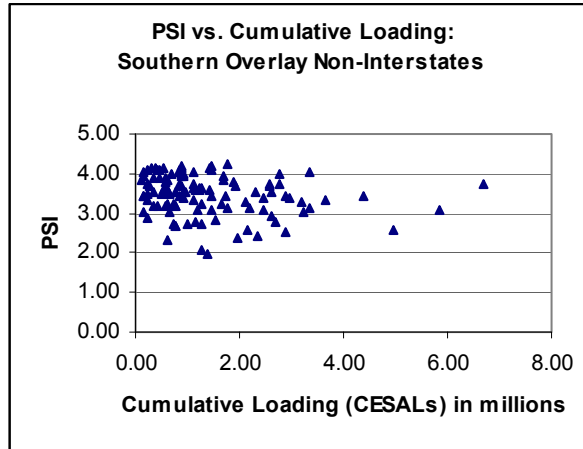


Figure A-17: Performance Trends for Overlay Non-Interstates- Southern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity)

APPENDIX F1

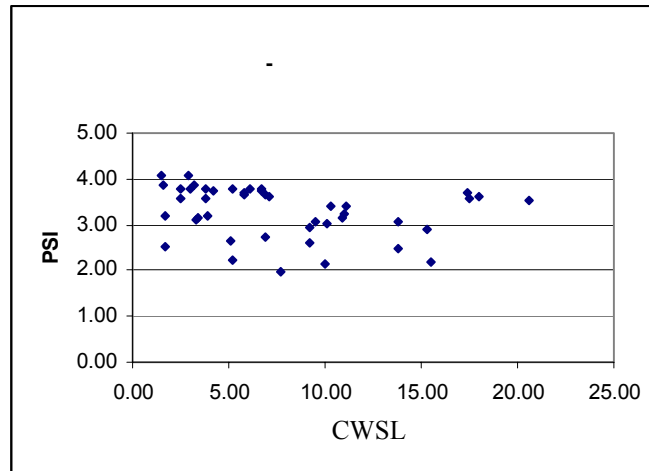
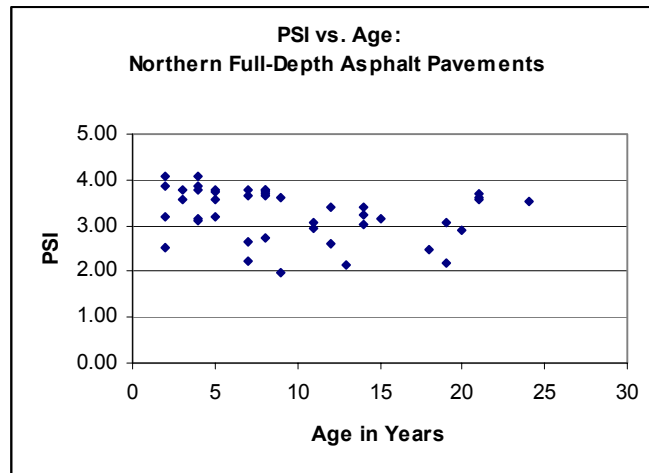
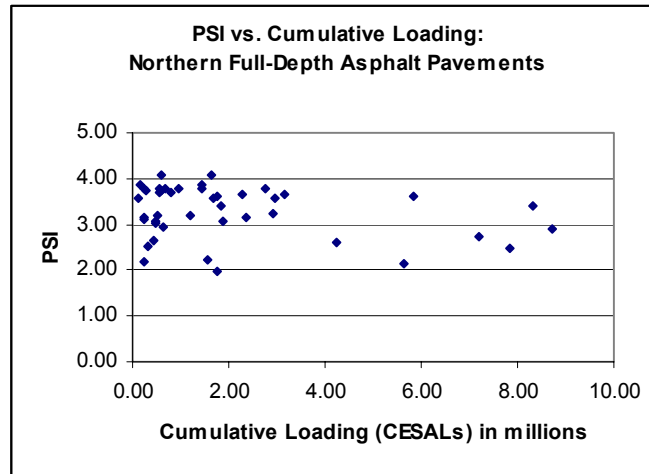


Figure A-18: Performance Trends for Full-Depth Asphalt Pavements- Northern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX F2

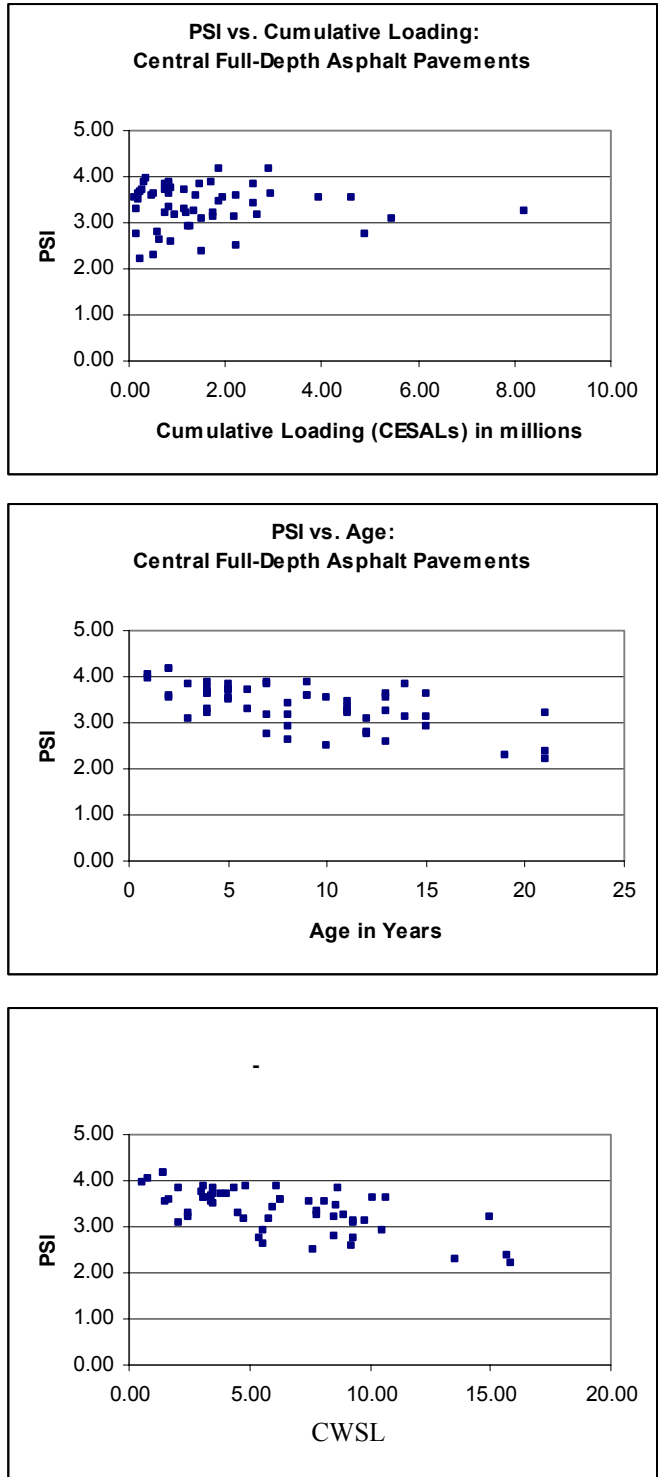


Figure A-19: Performance Trends for Full-Depth Asphalt Pavements- Central Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX F3

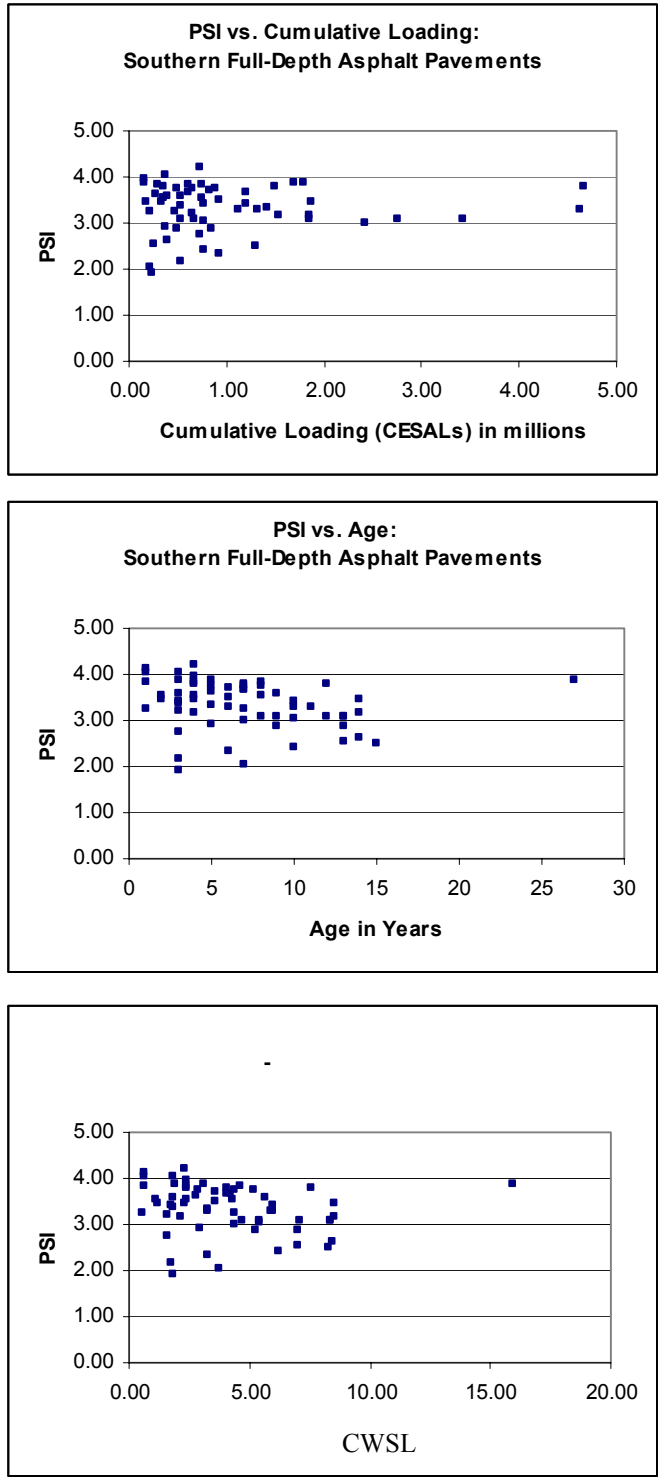


Figure A-20: Performance Trends for Full-Depth Asphalt Pavements- Southern Region (PSI vs. Age, PSI vs. Cumulative Loading, and PSI vs. Cumulative Weather Severity Level (CWSL))

APPENDIX G

Table A-1: HIGHWAY PRICE TRENDS AND CONSUMER PRICE INDEX
 (Based on 1987 Index Year)
 Source: FHWA Highway Statistics 1999, Page IV-16

INDEX	1983	1984	1985	1986	1987	1988	1989	1990	1991
Construction ¹	87.6	92.6	102	101.1	100	106.6	107.7	108.5	107.5
Maintenance ²	87.7	91.5	94.7	96.5	100	104.1	109.2	115.1	119.9

INDEX	1992	1993	1994	1995	1996	1997	1998	1999
Construction ¹	105.1	108.3	115.1	121.9	120.2	130.6	126.9	136.5
Maintenance ²	123.5	127.2	130.5	134.2	138.1	141.3	143.5	146.7

1: Refers to Federal-Aid Highway Construction. Capital Outlay constant 1987 dollars are calculated using the Federal Aid Highway Construction Index (See Table PT-1 in Highway Statistics 1999).

2. Maintenance constant 1987 dollars are calculated using the Consumer Price index.

APPENDIX H

UNIT RATES FOR LONG-TERM EVALUATION OF MAINTENANCE STRATEGIES

Table A-2: Values of Travel Time

	1999	1995
Autos	\$11.24	\$8.90
Single Unit Trucks	\$14.40	\$11.41
Multiple Unit Trucks	\$23.78	\$18.84
Buses	\$12.27	\$9.72

Source: Adjusted from MicroBENCOST User's Manual [TTI, 1993].

Table A-3: Costs of Police-reported Crashes [FTA, 1992]

	1999	1995
Urban Interstates	\$75,126	\$59,506
Urban Non-Interstates	\$55,772	\$44,176
Rural Interstates	\$129,607	\$102,661
Rural Non-Interstates	\$124,027	\$98,241

Table A-4: Crashes per Million VMT by Highway Functional Class [INDOT, 1997]

Urban Interstates	0.52
Urban Non-Interstates	4.25
Rural Interstates	0.89
Rural Non-Interstates	2.63

Information on Maintenance Work Zones

(Based on experience of INDOT's Operations Support Division)

- 1) Typical decrease in speed in work zones = 20 mph for Interstates, 15 mph for non-Interstates
- 2) Crash rates in maintenance work zones = 2 * typical crash rates (Table 69)
- 3) Typical duration of maintenance activities per lane-mile (Assuming average frequency and severity of distresses):
 - Crack Sealing: 2 hours
 - Chip Sealing: 48 hours
 - Micro-surfacing: 24 hours
 - Thin Overlays: 96 hours
 - Underdrain Maintenance: 0 hours (treatment does not interfere with passing traffic).

-

APPENDIX I

DISTRIBUTION OF WEATHER SEVERITY LEVELS I

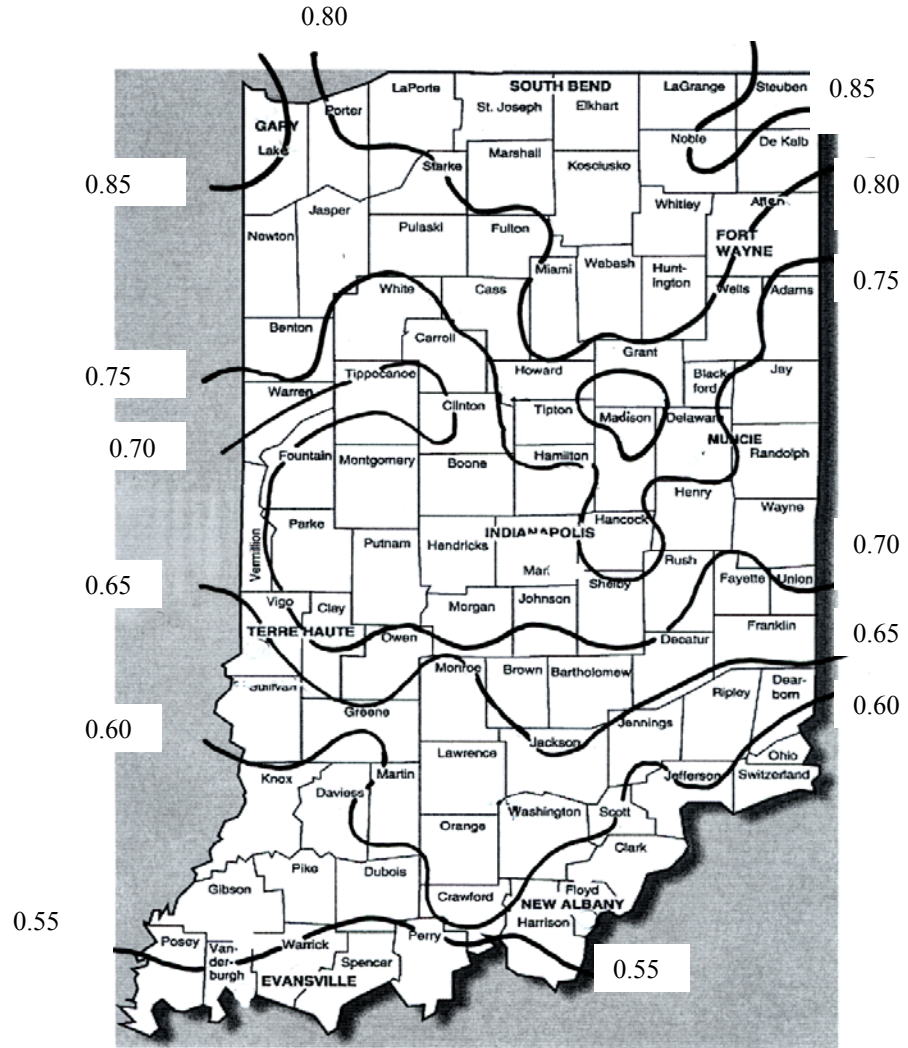


Figure A-21: Distribution of Weather Severity Levels using Contours

APPENDIX J

DISTRIBUTION OF WEATHER SEVERITY LEVELS II

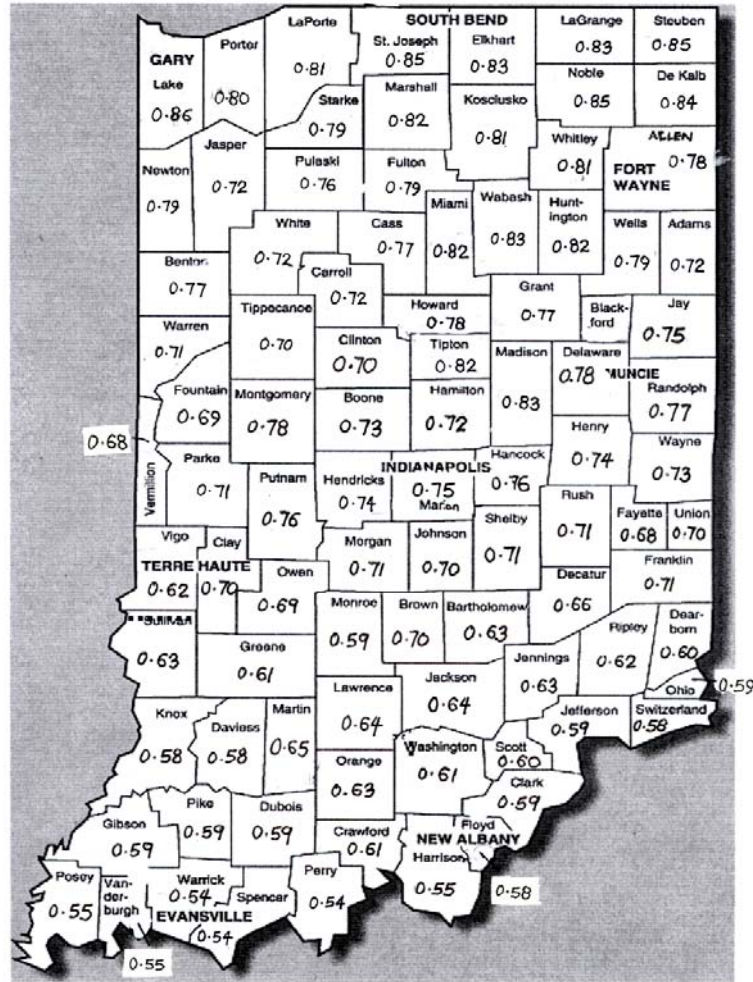


Figure A-22: Distribution of Weather Severity Levels Using Individual Values by County

APPENDIX K

RESULTS OF CLUSTER ANALYSIS I



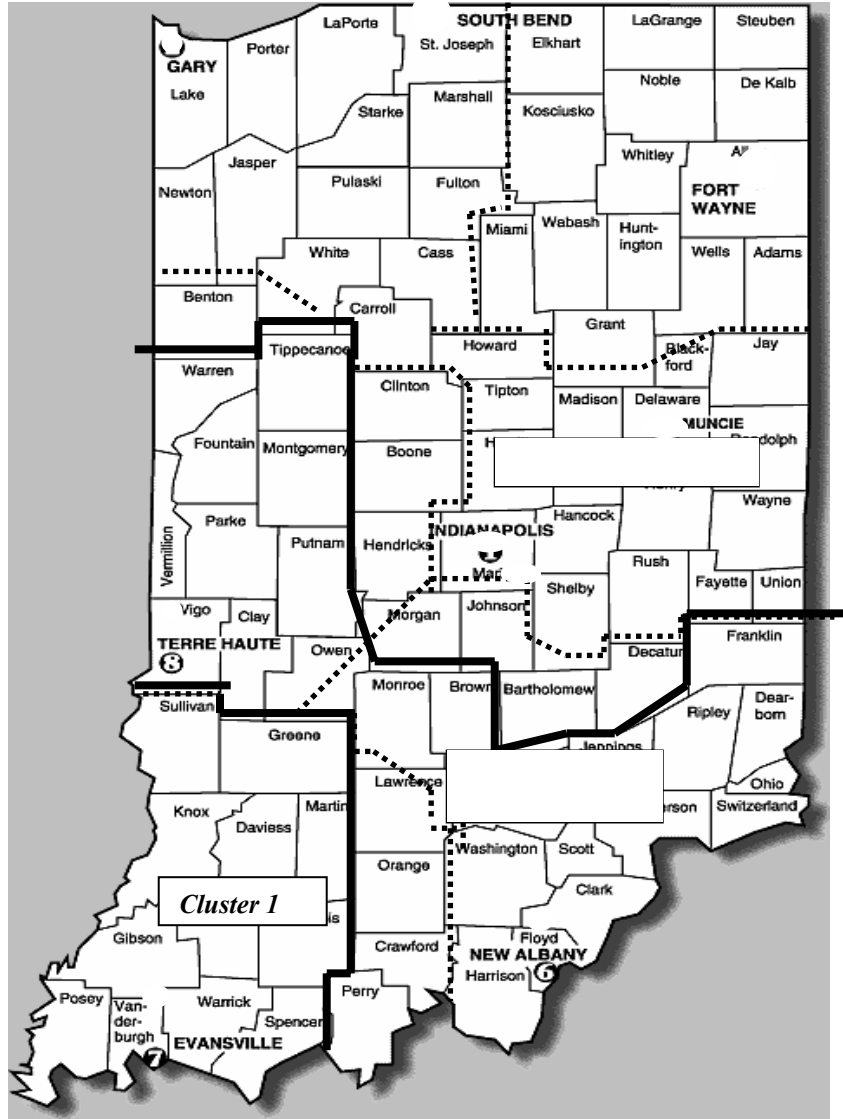
Legend

- Boundaries of Highway Administrative Districts
- Boundaries of Environmental Regions Formed Using Cluster Analysis

Figure A-23: Climatic Regions in Indiana Formed Using Cluster Analysis of Climate Data for each Indiana Climatic Center Zone.

APPENDIX L

RESULTS OF CLUSTER ANALYSIS II



Legend

- Boundaries of Highway Administrative Districts
- Boundaries of Environmental Regions Formed Using Cluster Analysis

Figure A-24: Climatic Regions in Indiana Formed Using Cluster Analysis of Climate Data for each Indiana County

**APPENDIX M: QUESTIONNAIRE SURVEY FOR MAINTENANCE PRACTICES
A21EFFECTIVENESS PERCEPTIONS**

Purdue/Indiana DOT Project # SPR-2397

Effectiveness Evaluation of Pavement Maintenance

Questionnaire for District Maintenance Engineers- Series #1

1. GENERAL INFORMATION

District Name: _____

Address: _____

Name and title of person compiling response:

Phone and fax numbers: _____

E-mail address: _____ Date _____

2. PURPOSE OF THIS QUESTIONNAIRE

The purpose of this questionnaire is to obtain information on your agency's experiences, observations and evaluations, if any, of preventive pavement maintenance practices you may have used in the past. With this information, it is expected that the cost-effectiveness of each treatment type can be assessed and reported back to you in the near future.

3. BACKGROUND INFORMATION

The following definitions are provided to establish a common understanding of the terms used in this questionnaire.

Preventive Pavement Maintenance: Planned maintenance activities done to prevent or delay future pavement deterioration. These activities are normally cyclical in nature and may correct minor surface defects as a secondary benefit.

Preventive Pavement Maintenance Treatment: The performance of a preventive maintenance activity at a specific point in time, e.g., crack sealing of AC pavements.

Preventive Pavement Maintenance Strategy: A plan for applying a series of preventive maintenance treatments at specified time intervals over the life of the pavement, e.g., seal cracks every 4 years and apply thin overlay every 12 years.

Cost-effectiveness of Preventive Pavement Maintenance: Any measure of the benefits of the preventive maintenance activity, usually in relation to cost. For example, increased service life of the pavement, increased length of the rehabilitation cycle, decreased levels of demand maintenance (e.g. patching), increased levels of pavement condition or performance.

4. PREVENTIVE MAINTENANCE TREATMENTS

The purpose of this question is to obtain an indication of the types or categories of preventive pavement maintenance treatments used by your agency and an indication of the overall performance of those treatments. Your agency may have more than one specific treatment in each category. For instance, an agency may have four specific crack filling treatments depending on the circumstances. It may or may not rout the cracks before filling and it may use two different filler materials. The purpose of this question is **not** to obtain detailed

information on each treatment, but rather to obtain information about the **types** of treatment used.

A. Please identify by a check mark the following types or categories of preventive pavement maintenance treatment used by your agency.

(1) For PCC pavements

- Joint Spall Repair
- Joint Sealer Replacement
- Other _____

(2) For AC pavements

- Crack filling (specify with or without routing)
- Single application chip seal
- Multiple application chip seal
- Slurry seal
- Micro-surfacing
- Thin HMA overlays
- Other (please describe) _____

(3) For Overlaid pavements

- Fill sawed and sealed joints in AC over old joints in PCC
- Crack filling (specify with or without routing)
- Single application chip seal
- Multiple application chip seal
- Slurry seal
- Micro-surfacing
- Thin HMA overlays
- Other (please describe) _____

B. For each of the treatment categories checked in section A above, please complete the appropriate area in the attached form (Appendix 1). In completing the form, please report on the treatment used in each category that provides the best overall performance for your agency.

5. PREVENTIVE MAINTENANCE STRATEGIES

A. Does your agency have any preventive maintenance strategy as previously defined?
 (Please circle) Yes No

B. If yes, what uses are made of the strategies in your agency. (Please check)

- (1) To select pavement sections or treatments during the design process that provide the Least life cycle costs. _____
- (2) To prepare the maintenance organization's budget _____
- (3) To order materials
- (4) To schedule work to be done by either agency forces or maintenance contractor
- (5) Other (please describe)

C. Do your preventive maintenance strategies vary by functional class? (Circle one)

Yes No

D. Do your preventive maintenance strategies vary by traffic volume? (Circle one)

Yes No

E. If your answer to A above is yes, please describe the preventive maintenance strategies which your agency has. Normally, the analysis period for pavement life cycle costing includes at least one rehabilitation period. However, for the purposes of this questionnaire, it is sufficient to describe the strategy up to the time of rehabilitation.

6. PLANNING AND FUNDING PREVENTIVE MAINTENANCE

A. Does your agency assume a preventive maintenance strategy during the design process to minimize pavement life cycle costs? Yes No

B. If yes,

(1) Does your agency identify or earmark monies in future years to fund the preventive maintenance treatments identified in the strategy?

Yes No

(2) Are the maintenance funds appropriated to your agency adequate to fund preventive maintenance?

Yes No

(3) Does your agency decrease its capital program by transferring funds to maintenance to adequately fund the preventive maintenance program?

Yes No

C. If your agency is **not** using pavement preventive maintenance strategies, please check the reasons why:

(1) The cost-effectiveness of preventive pavement maintenance has not been adequately demonstrated. _____

(2) Agencies that provide the funding, have not accepted the demonstrated cost-effectiveness of pavement preventive maintenance. _____

(3) All the agencies agree with the benefits, but there isn't enough money available to fund preventive maintenance. _____

(4) Other (please describe)

7. ANY FURTHER WORK NEEDED (in your opinion)

- A. Additional research is needed to demonstrate the cost effectiveness of preventive pavement maintenance activities _____
 - B. Presentations and literature needs to be prepared to convince the funding agencies of the benefits of preventive pavement maintenance _____
 - C. Other (please describe)
-
-

.....

Please return (using enclosed self-addressed and stamped envelope) to:

Professor Kumares. C. Sinha
1284 Civil Building
Purdue University
West Lafayette, IN 47907

or

Samuel Labi
1284 Civil Building
Purdue University
West Lafayette, IN 47907

Phone: (765)494-2211
Email: sinha@ecn.purdue.edu

Phone: (765)494-2206
Email: labi@ecn.purdue.edu

A response by the 30th of November would be appreciated.

THANK YOU FOR YOUR ASSISTANCE!

.....

APPENDIX 1: FORM FOR PREVENTIVE MAINTENANCE TREATMENTS

Pavement type	Preventive Maintenance Treatment Types	Typical observations for preventive maintenance treatment			
		Age of pavement at time of first treatment	Frequency of treatment application	Unit cost per treatment (\$ per sq.ft.)	Observed increase in pavement life ¹
PCC					
AC					
Overlaid					

1. Refers to the extension in time until the next rehabilitation or demand maintenance (such as patching).

APPENDIX 2: FORMS FOR PREVENTIVE PAVEMENT MAINTENANCE STRATEGY

Pavement type: PCC				
Year	Preventive maintenance treatments			
	Treatment 1	Treatment 2	Treatment 3	Other treatments
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				

1. Rehabilitation cycle length if strategy is not used

<10 years 10-12 years 13-15 years 16-20 years 21-25 years other ____

2. Cost effectiveness of strategy

A. Increased rehabilitation cycle length by ...

<2 years 3-4 years 5-6 years 7-8 years 9-10 years other

B. Reduction in the amount of **time** spent on pavement demand maintenance activities (e.g., unplanned or unscheduled pavement work which must be done for motorists safety, such as blow-up repairs, pothole patching, etc).

<5% 5-10% 11-15% 16-20% 21-25% other ____

C. Reduction in the amount of **cost** of pavement demand maintenance activities

<5% 5-10% 11-15% 16-20% 21-25% other ____

D. Improvement in pavement condition or performance

<5% 5-10% 11-15% 16-20% 21-25% other ____

Pavement type: AC				
Year	Preventive maintenance treatments			
	Treatment 1	Treatment 2	Treatment 3	Other treatments
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				

1. Rehabilitation cycle length if strategy is not used

<10 years 10-12 years 13-15 years 16-20 years 21-25 years other _____

2. Cost effectiveness of strategy

C. Increased rehabilitation cycle length by ...

<2 years 3-4 years 5-6 years 7-8 years 9-10 years other _____

D. Reduction in the amount of **time** spent on pavement demand maintenance activities (e.g., unplanned or unscheduled pavement work which must be done for motorists safety, such as blow-up repairs, pothole patching, etc).

<5% 5-10% 11-15% 16-20% 21-25% other _____

C. Reduction in the amount of **cost** of pavement demand maintenance activities

<5% 5-10% 11-15% 16-20% 21-25% other _____

D. Improvement in pavement condition or performance

<5% 5-10% 11-15% 16-20% 21-25% other _____

Pavement type: Overlaid				
Year	Preventive maintenance treatments			
	Treatment 1	Treatment 2	Treatment 3	Other treatments
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				

1. Rehabilitation cycle length if strategy is not used

<10 years 10-12 years 13-15 years 16-20 years 21-25 years other _____

2. Cost effectiveness of strategy

E. Increased rehabilitation cycle length by ...

<2 years 3-4 years 5-6 years 7-8 years 9-10 years other _____

F. Reduction in the amount of **time** spent on pavement demand maintenance activities (e.g., unplanned or unscheduled pavement work which must be done for motorists safety, such as blow-up repairs, pothole patching, etc).

<5% 5-10% 11-15% 16-20% 21-25% other _____

C. Reduction in the amount of **cost** of pavement demand maintenance activities

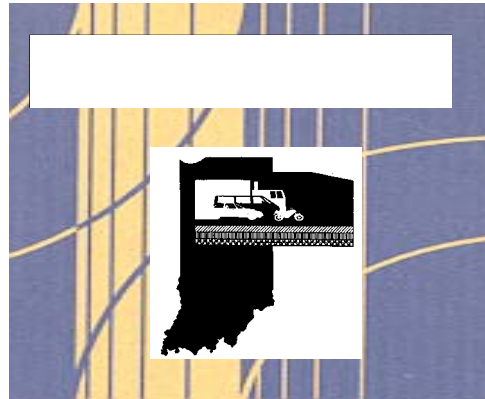
<5% 5-10% 11-15% 16-20% 21-25% other _____

D. Improvement in pavement condition or performance

<5% 5-10% 11-15% 16-20% 21-25% other _____

ADDENDUM TO FINAL REPORT
ON
THE EFFECTIVENESS OF MAINTENANCE
AND ITS IMPACT ON CAPITAL EXPENDITURES

**INDIANA GENERAL PAVEMENT RESEARCH DATABASE (IndiPave 2000)
USERS MANUAL**



By

Samuel Labi
Visiting Assistant Professor
and
Kumares C. Sinha
Olson Distinguished Professor

School of Civil Engineering
Purdue University

Joint Transportation Research Program
File No.: 3-5-12
SPR-2397

Prepared in Cooperation with the
Indiana Department of Transportation and
The U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Federal Highway Administration and the Indiana Department of Transportation. The report does not constitute a standard, a specification, or a regulation.

Purdue University
West Lafayette, Indiana, 47907
May 2003

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6. Samples of Forms that can be generated by the database	28
7. List of Contact Persons for Various data types	28

1. Acknowledgements

The design and development of this database was carried out with the assistance of several individuals. John Weaver and Bill Flora of INDOT'S Roadway Management Division made available INDOT's PMS database, which provided the nucleus of the referencing scheme used in INDIPAVE 2000. Also, the PMS database provided valuable data on pavement condition, and design/construction features.

Dennis Belter and Mark Burton of INDOT's Operation Support Division provided data on pavement and shoulder work carried out on a sub-district level and on a force account basis in Indiana. This information was made available by OSD's Maintenance Management System, which facilitated access to information about the locations and performance levels of maintenance carried out on state road segments.

We are also grateful to Leah Snow, Roy Adams and Mahlon Bartlett of the HPMS and Pavement Archive Sections of INDOT for providing valuable data on contracts that have been carried out on the state roads over the last couple of decades.

John Nagle, Marcia Guftasson, Cordelia Jones-Hill, and Geraldine Lampley were instrumental in making available traffic information, either in the form of coverage count data or ATR records.

Nayar Zia of INDOT Materials and Tests Division provided access for us various reports and documents through which we mined for pavement geotechnical information that was collected as part of pre -engineering studies prior to road construction. Transportation graduate students Zongzhi Li and Mohammad Islam were also instrumental in the development of this databse.

Finally, the contribution of student workers (Paul Turbyfill, Yifei Zang, John Groth, David Devine, Ruth Pilimon, and Cynthia Boley), volunteers (Ashad and Shadia Hamideh and Ngiseli Joor Ise Diagne) are very much appreciated.

2. INTRODUCTION

IndiPave2000 is a relational database that contains pavement-related information on approximately 5000 miles of road segment on Indiana's highway system. Such information includes pavement condition, design and construction features, traffic characteristics, climatic features, and M&R activities.

Utilizing the referencing system (and consequently, the road segmentation scheme) used in INDOT's PMS database as its nucleus, IndiPave2000 essentially brings together information from various sources within INDOT and beyond and presents them using a common reference, thereby obviating any problems associated with the use (by such data sources) of different referencing systems.

The data items generally span the years 1992 to 1998, but there are some data items that go as far back as 1985 (such as AADTs) and 1957 (some contract data) or are as recent as 1999 (such as construction contracts data). Entity-relation diagrams that present the various data items as well as their relationships are shown as Figures 1 and 2.

It is expected that the development of this database will be a continuous effort that would include the following:

- Expansion of data units to include the remaining 6000 miles of state roads that are not currently covered by IndiPave2000,
- Extension of the range of data types, i.e., inclusion of new tables (such as a Pavement History Archive, HPMS datafile, etc), and addition of new columns in existing tables such as the dominant surficial geology in the Geotechnical Table
- Possible expansion of the time frame for all data types to the most current year.

Hopefully, this database signifies the advent of a new era of pavement research in Indiana, and it is hoped that pavement researchers and managers will take full advantage of the data contained herein.

*Kumares C. Sinha, Olson Distinguished Professor of Civil Engineering
Samuel Labi, Visiting Assistant Professor of Civil Engineering*

May 2003

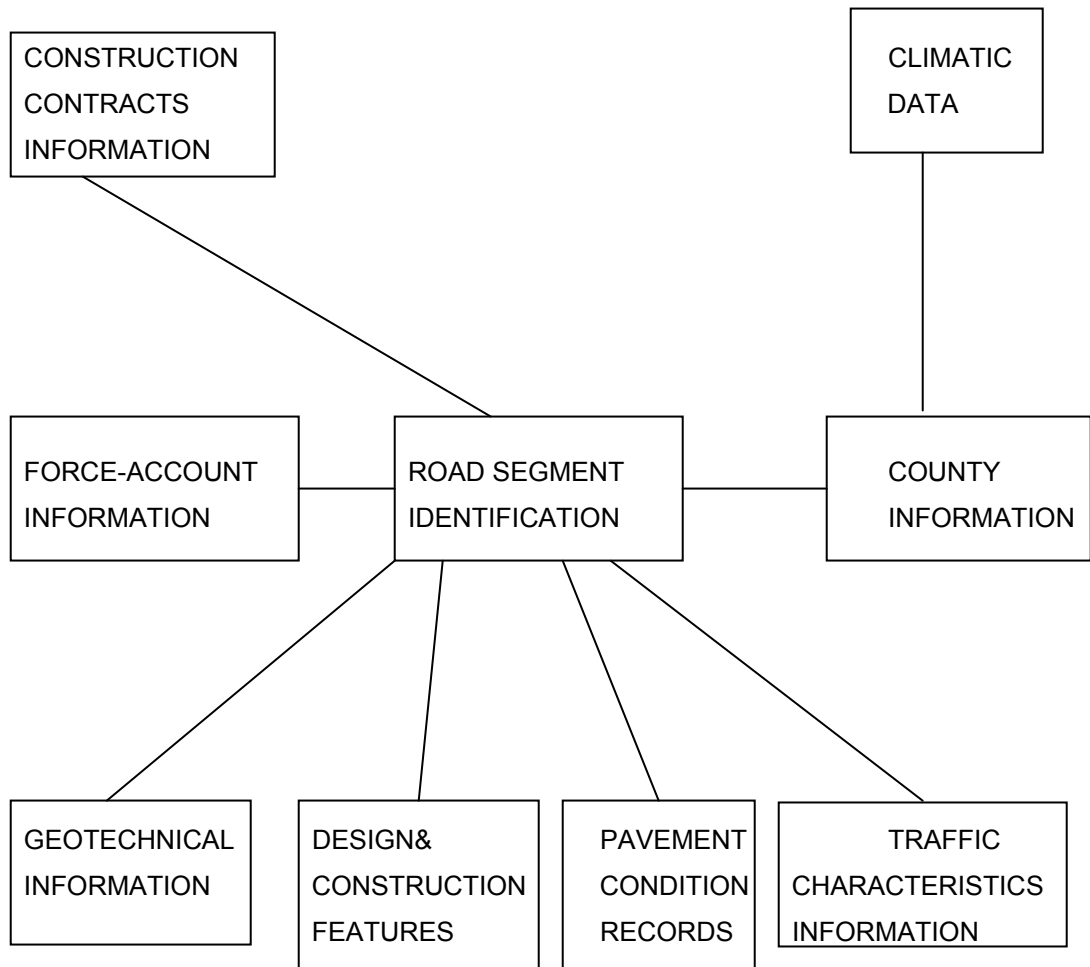


Figure 1: Entity Relation Diagram (General)

3. LOGICAL DESIGN OF THE DATABASE

IndiPave2000 is a relational database consisting of several tables (entities) as shown as Figure 1. Each table consists of several fields (for instance the Traffic Characteristics table has fields that include AADT and %trucks). The relational nature of this database allows the user to query from any table for any instance of any data unit, for example: "What is the number of freeze-thaw cycles for Pavement Segment 2231 in 1995?" The database also lets the user build his/her own custom-tailored tables, selecting various fields from any tables that are of his or her interest.

Each table has a primary key that is usually the first column of that table. The primary key alone is sufficient for accessing the rest of the data items for a given segment. For instance, in the road Identification table, the Segment_ID is the primary key, given which one can determine the starting and end points of that segment, the NHS classification, etc. For this reason, the primary key of each table is unique: No two segments have the same primary key.

Most tables also have a foreign key. The foreign key of a table provides a link to the other tables. The foreign key of the "original" table should be the same as the primary key of the "destination" table, if such a link is to be realized. For instance, County_ID is a foreign key in the Road Segment Identification table, but is a primary key in the county Information table. The database has been designed to facilitate such links.

The following table shows the primary and foreign keys of each table in the database:

Table (Entity)	Primary Keys	Foreign Key(s)	Foreign key Destination Table
Road Segment Info	Segment ID #	County ID #	County Information Table
County Information	County ID #	Indiana Climate Center (ICC) ID # Environmental Zone ID	Zonal Climate Information Regional Climate Information
Zonal Climate	ICC #	-	-
Regional Climate	Environmental Region ID	-	-
Traffic characteristics	Segment ID #	-	-
Pavement Condition Data	Segment ID #	-	-
Geotechnical Data	Segment ID #	-	-
Design and Construction Features	Segment ID #	-	-
Segment's Force-Account Histories	Segment ID #	Force Account ID #	Force Account Record
Force Account Record	Work ID #	-	-
Segment's Contracts Histories	Segment ID #	Contract ID #	Contract Record
Contract Record	Contract ID #	-	-

4. PHYSICAL DESIGN OF DATABASE

IndiPave was originally designed and implemented on a Microsoft Excel platform, and then each table was exported to a pre-designed but unpopulated MS Access database. IndiPave2000 runs on any IBM compatible PC with the following specifications as a minimum:

- 166 MHz Pentium ii Processor (MMX)
- 32MB RAM
- 24X CD ROM
- 2.5 GB hard drives

5. DETAILS OF DATABASE CONTENTS

5.1 Description of Fields for Road Segment Information Table

The Road Segment Identification Table has the following fields:

SEGMENT_id: This is the serial number assigned to each pavement segment under study, ranging from 1 to 9901.

ROAD_NAME: This is the common name of the road section, e.g., I-65.

AREA_CLASS: This is the HMPS classification of the area through which the road section passes. Area class may be urban or rural.

FHWA_CLASS: This is any one of the several classes of road as defined in the HPMS database, as defined as follows:

01- Rural Interstate	02-Rural Other Principal Arterial
03- Rural Minor Arterial	04- Rural Major Collector
05- Rural Minor Collector	11- Urban Interstate
12- Urban Other Freeways and Expressways	
13- Urban Minor Arterial	14- Urban Collector

TRAFFIC_DXN: This is the direction of traffic on the road section, i.e., Northward, Southward, Eastward, or Westward.

START_POINT: This is the starting location of the road segment, based on the PMS linear Referencing System adopted by INDOT's Pavement Management System.

END_POINT: This is the terminal location of the road segment, based on the PMS linear Referencing System adopted by INDOT's Pavement Management System.

NHS_STATUS: This indicates whether the road section is on the National Highway System.

COUNTY_ID: This is the INDOT-assigned serial number of the county through which the road section passes.

5.2 Description of Fields for the County Information Table

The County Information Table has the following fields:

CNTY_ID: This is the INDOT-assigned serial number of the county through which the road section passes.

CNTY_NAME: This is the name of the county through which the road section passes.

HWY_DIST_ID: This is the serial number of the highway district through which the road section passes:

1-Laporte District	2-Fort Wayne District	3- Crawfordsville District
4- Greenfield District	5- Vincennes District	6- Seymour District

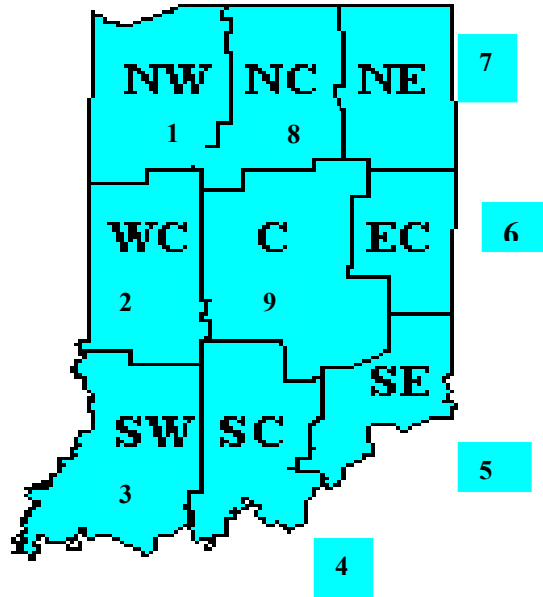
ENVR_ZONE_ID: This is the serial number of the environmental zone through which the road section passes. There are 9 environmental zones defined by the Indiana Climatic Center, as shown below:

ENVR_REGION_ID: This is the serial number of the environmental region through which the road section passes. Three distinct environmental regions have been defined by previous studies:
1- Northern Indiana, 2- Central Indiana, 3-Southern Indiana

5.3 Description of Fields for the Environmental Zone Information Table

The Environmental Zone Information Table has the following fields:

ENVR_ZONE_ID: This is the serial number of the environmental zone through which the road section passes. There are 9 environmental zones defined by the Indiana Climatic Center, as shown below:



ENVR_ZONE: This is the name of the environmental zone through which the road section passes, as defined by the Indiana Climatic Center (ICC) (see figure above).

AVG_TEMP: This is the mean average temperature of the ICC environmental zone to which the road segment belongs, in degrees Fahrenheit.

MAX_TEMP: This is the maximum average temperature of the ICC environmental zone to which the road segment belongs, in degrees Fahrenheit.

MIN_TEMP: This is the minimum average temperature of the ICC environmental zone to which the road segment belongs, in degrees Fahrenheit.

- #DAYS<0: This is the number of days for which the average temperature of the ICC environmental zone to which the road segment belongs, experienced temperatures below freezing point.
- #DAYS>32: This is the number of days for which the average temperature of the ICC environmental zone to which the road segment belongs, experienced temperatures above 32 degrees Fahrenheit.
- FRZINDX: This is the freeze index of the ICC environmental zone to which the road segment belongs.
- #F/T_CYC: This is the number of times the average temperature of the ICC environmental zone to which the road segment belongs, crosses the freezing point. (i.e., the number of freeze/thaw cycles).
- #WETDAYS: This is the number of days that the ICC environmental zone to which the road segment belongs, experienced precipitation **exceeding 2 inches**.
- (Month)AVG-TEMP: This is the mean average mean temperature of the ICC environmental zone to which the road segment belongs, experiences in the month. A 30-year monthly average was used.
- (Month)MIN-TEMP: This is the average minimum temperature of the ICC environmental zone to which the road segment belongs, experiences in the month. A 30-year period average was used was used to compute this figure.
- (Month)MAX-TEMP: This is the average maximum temperature of the ICC environmental zone to which the road segment belongs, experiences in the month. A 30-year period average was used was used to compute this figure.

5.4 Description of Fields for the Environmental Region Information Table

The Environmental Region Information Table has the following fields:

- ENVR_REGION_ID: This is the serial number of the environmental region through which the road section passes. There are 3 environmental regions as defined by previous research in Indiana.
- ENVR_REGION_NAME: This is the name of the environmental region through which the road section passes, Northern, Southern, and Central.
- AVG_TEMP: This is the mean average temperature of the environmental region to which the road segment belongs, in degrees Fahrenheit.
- MAX_TEMP: This is the maximum average temperature of the environmental region to which the road segment belongs, in degrees Fahrenheit.
- MIN_TEMP: This is the minimum average temperature of the environmental region to which the road segment belongs, in degrees Fahrenheit.
- #DAYS<0: This is the number of days for which the average temperature of the environmental region to which the road segment belongs, experienced temperatures below freezing point.
- #DAYS>32: This is the number of days for which the average temperature of the environmental region to which the road segment belongs, experienced temperatures above 32 degrees Fahrenheit.
- FRZINDX: This is the freeze index of the environmental region to which the road segment belongs.
- #F/T_CYC: This is the number of times the average temperature of the environmental region to which the road segment belongs, crosses the freezing point. (i.e., the number of freeze thaw cycles).

#WETDAYS: This is the number of days that the environmental region to which the road segment belongs, experienced significant precipitation.

(month)AVG-TEMP: This is the mean average mean temperature of the environmental region to which the road segment belongs, experiences in the month. A 30-year monthly average was used.

(month)MIN-TEMP: This is the average minimum temperature of the environmental region to which the road segment belongs, experiences in the month. A 30-year period average was used was used to compute this figure.

(Month)MAX-TEMP: This is the average maximum temperature of the environmental region to which the road segment belongs, experiences in the month. A 30-year period average was used was used to compute this figure.

5.5 Description of Fields for the Traffic Data Table

The Traffic Data Table has the following fields:

SEGMENT_ID: This is the serial number assigned to each pavement segment under study, ranging from 1 to 9901.

ROAD_NAME: This is the common name of the road section, e.g., I-65.

TRAFFIC_DXN: This is the direction of traffic on the road section, i.e., Northward, Southward, Eastward, or Westward.

START_POINT: This is the starting location of the road segment, based on the PMS linear Referencing System adopted by INDOT's Pavement Management System.

END_POINT: This is the terminal location of the road segment, based on the PMS linear Referencing System adopted by INDOT's Pavement Management System.

(Year)AADT: This is the average AADT of the road segment, for each Year.

(Year)%TRUCKS: This is the percentage of trucks (FHWA vehicle classes 4 and above) that ply the road section in any given year.

NR_LANES: This is the number of lanes that the road section has.

LDF: this is the lane distribution factor of the road section, based on theoretical considerations.

LEF: This is the Load Equivalency Factor, based on the LDF.

(Year)ESALs: This is the traffic loading on the pavement, measured in ESALs.

5.7 Description of Fields for the Design and Construction Features Table

The Design and Construction Features Table has the following fields:

SEGMENT_ID: This is the serial number assigned to each pavement segment under study, ranging from 1 to 9901.

PAVE_TYPE: This is the material of which the pavement is comprised. There are 3 principal types of pavement types in the database: Asphalt, PCC, and composite.

SURF_THICK: This is the thickness of the pavement surface, in inches.

SB/B_COMP: This is the degree of compaction of the base and subbase, expressed as a percentage of the maximum dry density.

%ASP: This is the percentage of asphaltic cement in the asphaltic concrete mix. Applies to flexible pavements only.

AGGR<#4: This is the percentage of aggregates in the asphaltic concrete Mix that pass through the #4 sieve. Applies to flexible pavements.

HMA_VOIDS: This is the percentage of air voids in the asphaltic concrete mix. Applies to flexible pavements only.

EL_MOD: This is the elastic modulus of concrete. Applies to flexible pavements only.

RUP_MOD: This is the rupture modulus of concrete. Applies to flexible pavements only.

JNT_SPAC: This is the spacing of joints in the concrete slab, in inches.

DOW_DIA: This is the diameter of the dowels used in slab construction.

SUB_DRAIN: This is an indication of whether the road section has any subdrains. .

5.8 Description of Fields for the Geotechnical Data Table

The Geotechnical Data Table covers information about the mechanical properties of the subgrade, either as a natural residual material, or as imported fill material. This is table has the following fields:

SEGMENT_ID: This is the serial number assigned to each pavement segment under study, ranging from 1 to 9901.

%NG-%FINES: This is the percentage of material passing through the 0.425mm sieve.

NG_MOIST: This is the moisture content of the natural ground over which a road segment passes.

NG_LL: This is the liquid limit of the natural ground over which a road segment passes.

NG_PL: This is the plastic limit of the natural ground over which a road segment passes.

NG_PI: This is the plasticity index of the natural ground over which a road segment passes.

NG_MDD: This is the maximum dry density of the natural ground over which a road segment passes.

NG_CBR: This is the CBR, at 93% compaction, of the natural ground over which a road segment passes.

NG_RM: This is the resilient modulus of the natural ground over which a road segment passes.

FILL_STATUS: This is an indication of whether there is any imported fill material over the natural ground, for a given road segment, for purposes of providing additional stability of the roadbed. FILL_STATUS = 1 if there is such fill material, but = 0 otherwise.

%FM-%FINES: This is the percentage of material passing through the 0.425mm sieve.

FM_MOIST: This is the moisture content of the imported fill material over which a road segment passes.

FM_LL: This is the liquid limit of the imported fill material over which a road segment passes.

FM_PL: This is the plastic limit of the imported fill material over which a road segment passes.

FM_PI: This is the plasticity index of the imported fill material over which a road segment passes.

FM_MDD: This is the maximum dry density of the imported fill material over which a road segment passes.

FM_CBR: This is the CBR, at 93% compaction of the imported fill material over which a road segment passes.

FM_RM: This is the resilient modulus of the imported fill material over which a road segment passes.

5.9 Description of Fields for the Contract Record Table

The Contracts Record is a table that has the identification numbers of various contracts executed on state road over a past couple of decades. It also includes the starting and end points of the contract and other information specific to the contract. It does not include an identification of segments covered under each contract, as that information is provided in a separate file (the segment's contract file) on a year-by-year basis. Descriptions of the field titles of each column are provided below:

(Year)CNTRCT_ID: This is the INDOT-assigned serial number of the contract.

ROAD_NAME: This is the name of the road on which the contract is carried out.

WORK_TYPE: This is the dominant type of work carried out under the contract. Examples are "New Pavement", "Resurfacing", and "Patching".

M&R_CAT: This is the general Maintenance and Rehabilitation category under which the Work Type falls. M&R_CAT of any contract is one of three: Maintenance, Rehabilitation, and reconstruction.

WORK_LOC: This is a description of the starting and ending points of the contract.

DISTANCE: This is the length of the contract, in miles.

YEAR: This is the year in which the contract was completed, and the road was opened to traffic.

START_DATE: This is the date when the contract was awarded.

END_DATE: This is the date which the contract was completed, and the road was opened to traffic.

SURF_TYPE: This is the surface type of the road after completion of the contract.

OVERALL_THK: This is the total thickness of the added pavement layers after execution of the contract, in inches.

REHAB_WIDTH: This is the width of the rehabilitated road after completion of the contract, in ft.

EXPEND: This is the amount spent on pavement-related works in the contract. It is the total contract sum less the expenditure on non-pavement-related items.

UNIT_EXP1: This is the amount spent on pavement-related works in the contract per unit length of road covered under the contract.

NR_LANES: This is the number of lanes of the rehabilitated road, in one direction.

UNIT_EXP2: This is the amount spent on pavement-related works in the contract per lane-mile of road covered under the contract.

SPEC_SUB: This is the thickness of any special subbase imported to improve stability of the roadbed, often in preparation for laying a concrete slab.

AC_BASE: This is the thickness of asphaltic concrete base laid during construction.

AC_BINDER: This is the thickness of asphaltic concrete binder laid during construction.

AC_SURFACE: This is the thickness of asphaltic concrete surface course laid during construction.

PL_CONC: This is the thickness of plain concrete slabs laid during construction.

- RF_CONC:** This is the thickness of reinforced concrete slabs laid during construction.
- UNDSEAL:** This is the amount spent on the undersealing of voids under concrete slabs.
- PATCHING:** This is the amount of money spent on patching the pavement surface, usually in preparation for an overlay.
- SUBDRAIN:** This is the amount of money spent on construction of subdrains as part of the contract.
- COUNTY:** This is the INDOT-assigned serial number of the county in which the contract was executed.
- HWY_DIST_ID:** This is the serial number of the highway district in which the contract was executed.
- | | | |
|------------------------|-----------------------|----------------------------|
| 1-Laporte District | 2-Fort Wayne District | 3- Crawfordsville District |
| 4- Greenfield District | 5- Vincennes District | 6- Seymour District |
- FHWA_CLASS:** This is a code representing the federal class of the road on which the contract was carried out.
- COMMENT:** Any comments about the contract are placed in this column.

5.10 Description of Fields for the Segments' Contract Table

The Segments Contracts Record is a table that lists all the road segments and identifies what contact was carried out on which segment and in which year. Contracts are identified only by their contract numbers, and the relational nature of this database enables the user to access details in each contact from the Contract Record Table.

Descriptions of the field titles of each column are provided below:

SEGMENT_ID: This is the serial number assigned to each pavement segment under study, ranging from 1 to 9901.

(Year)CNTRCT_ID: This is the INDOT-assigned serial number of the contract. There are 9 fields, one from each year from 1985 to 1999.

5.11 Description of Fields for the Force-Account Record Table

The Force Accounts Record is a table that has the identification numbers of various work executed by INDOT Operations Support Division, on a sub-district level, over the past decade. It also includes the starting and end points of the work and other information specific to the work. It does not include an identification of segments covered under each work, as that information is provided in a separate file (the segment's force-account file) on a year-by-year basis. OSD has segmented the roads in a certain manner, and this is not to be confused with the 1-mile segments system used in this database. Each OSD segment consists of several 1-mile segments used in this database. Descriptions of the field titles of each column are provided below:

(Year)WORK_ID: This is the serial number of the force account work activity.

This number, which was coined by the designers of the database, consists of the year of work, an abbreviation of the name of the sub-district, the road name, and the serial number of the road section (i.e., the OSD road segment).

SUBDIST: This is the name of the sub district in which the work was carried out.

ROAD_NAME: This is the name of the road on which the contract is carried out.

FROM: This indicates the starting point of the force-account road segment on which the work as carried out.

TO: This indicates the ending point of the force-account road segment on which the work as carried out.

YEAR: This is the year in which the work was carried out.

SH_PATCH: This is the level of shallow patching carried out on an OSD road segment, in tons.

SHPATCH_EXP: This is the amount spent on shallow patching on an OSD road segment.

- DP_PATCH: This is the level of deep patching carried out on an OSD road segment, in tons.
- DPPATCH_EXP: This is the amount spent on deep patching on an OSD road segment.
- CRK_SEAL: This is the level of crack sealing carried out on an OSD road Segment.
- CRKSEAL_EXP: This is the amount spent on crack sealing on an OSD road segment.
- JNT_SEAL: This is the level of joint sealing carried out on an OSD road Segment.
- JNTSEAL_EXP: This is the amount spent on joint sealing on an OSD road segment.
- SHLD_REP: This is the level of shoulder repairs, typically sealing, carried out on an OSD road segment.
- SHLDREP_EXP: This is the amount spent on shoulder repairs on an OSD road segment.
- SPTSHDR_REP: This is the level of spot repair of unpaved shoulder carried out on an OSD road segment.
- SSHDR_EXP: This is the amount spent on spot repair of unpaved shoulder carried out on an OSD road segment
- SUBDRAIN: This is the level of work carried out on the inspection and cleaning of existing underdrains on an OSD road segment.
- SUBDRN_EXP: This is the amount spent on the inspection and cleaning of existing underdrains on an OSD road segment.
- SHLD_BLD: This is the level of work carried out on the blading of shoulders.

- SBLD_EXP: This is the amount spent on the blading of shoulders,.
- SHLD_CLP: This is the level of work carried out on the clipping of shoulders, on an OSD road segment.
- SCLP_EXP: This is the amount spent on the clipping of shoulders, on an OSD road segment.
- SEAL_COAT: This is the level of work carried out on the seal coating on an OSD road segment.
- SCOAT_EXP: This is the amount spent on t the seal coating on an OSD road segment.
- PREMIX: This is the level of work carried out on the leveling of the pavement surface using premixed asphaltic material OSD road segment.
- PRMX_EXP: This is the amount spent on the leveling of the pavement surface using premixed asphaltic material OSD road segment.
- OTHER: This is the level of work carried out on other pavement and shoulder-related activities on an OSD road segment.
- OTHR_EXP: This is the amount spent on other pavement and shoulder-related activities on an OSD road segment.
- TOTAL_EXP: This is the total expense on a given OSD segment
- CM_EXP: This is the total expense on corrective maintenance activities on a given OSD segment
- PM_EXP: This is the total expense on preventive maintenance activities on a given OSD segment

SHDR_EXP: This is the total expense on shoulder maintenance activities on a given OSD segment.

NR_LANES: This is the number of lanes of the road within the segment in question.

DISTANCE: This is the length of the OSD road segment, in miles.

UNIT_CM_EXP1: This is the expense on corrective maintenance activities on a given OSD segment, per mile, for the entire year.

UNIT_CM_EXP2: This is the expense on corrective maintenance activities on a given OSD segment, per lane-mile for the entire year.

UNIT_PM_EXP1: This is the expense on preventive maintenance activities on a given OSD segment, per mile for the entire year..

UNIT_PM_EXP2: This is the expense on preventive maintenance activities on a given OSD segment, per lane-mile for the entire year..

UNIT_SHDR_EXP1: This is the expense on shoulder maintenance activities on a given OSD segment, per mile for the entire year..

UNIT_SHDR_EXP2: This is the expense on shoulder maintenance activities on a given OSD segment, per lane-mile for the entire year.

FALLCM_EXP: This is the expense on corrective maintenance activities on a given OSD segment, per lane-mile, in the second half (fall) of a given year.

FALLPM_EXP: This is the expense on preventive maintenance activities on a given OSD segment, per lane-mile, in the second half (fall) of a given year.

FALLSHD_EXP: This is the expense on shoulder maintenance activities on a given OSD segment, per lane-mile, in the second half (fall) of a given year.

SPGCM_EXP: This is the expense on corrective maintenance activities on a given OSD segment, per lane-mile, the first half (spring) of a given year.

SPGPM_EXP: This is the expense on preventive maintenance activities on a given OSD segment, per lane-mile, in the first half (spring) of a given year.

SPGSHD_EXP: This is the expense on shoulder maintenance activities on a given OSD segment, per lane-mile in the first half (spring), of a given year.

5.1 Description of Fields for the Segments' Force Account Table

The Segments Force Account Record is a table that lists all the road segments and identifies what OSD work activity was carried out on which segment and in which year. OSD work activities are identified only by their code numbers, and the relational nature of this database enables the user to access details in each contact from the Force Account Records Table.

Descriptions of the field titles of each column are provided below:

SEGMENT_ID: This is the serial number assigned to each pavement segment under study, ranging from 1 to 9901.

(Year)WORK_ID: This is the assigned serial number of the OSD work activity or road segment (assuming one activity (consisting of several sub-activities) per road segment. There are 9 fields, one from each year from 1991 to 1997.

9. LIST OF CONTACT PERSONS

Table 1: Details of Data Elements and Sources of Information

Data Category	Data Type	INDOT Contact person(s)
ROAD SEGMENT IDENTIFICATION	<u>For each segment:</u> Segment ID # Segment ID # Road Name Starting milepost reference Ending milepost reference Starting coordinates Ending Coordinates	Bill Flora and Mike Yamin (Roadway Management Div.) (233-1060) Mark Burton (Operations Support Div.) (232-5547)
ROUTINE MAINTENANCE DATA	<u>For each segment & year:</u> Category of maintenance (general, preventive, corrective) Name of treatment Location of maintenance Unit costs of maintenance	Dennis Belter (Operations Support Div.) Mark Burton (Operations Support Div.)
REHABILITATION DATA	<u>For each segment & year:</u> Type of rehabilitation Location of rehabilitation Cost of rehabilitation Thickness of overlay	John Weaver (Roadway Management Div.) Bill Flora (Roadway Management Div.) Leah Snow (HMPS Unit) Mahlon Bartlett (Roadway Management Div)
PAVEMENT STRUCTURE & MATERIALS	<u>For each segment & year:</u> Type and Thickness of ... Surface, Base and Subbase Material properties of... Surface, Base, Subbase and Subgrade	Mahlon Bartlett Nayar Zia (INDOT Materials and Tests)
PAVEMENT CONDITION	<u>For each segment & year:</u> International Roughness Index Rutting Index Pavement Quality Index Present Serviceability Index Cracking Index	John Weaver Bill Flora Mike Yamin
TRAFFIC DATA	<u>For each segment & year:</u> Traffic Volume (AADT) Percent Commercial Vehicles ESALs Cumulative ESALs Speed Limit Operating Speed	John Nagle Scott McArthur Marcia Guftasson Geraldine Lampley Cordelia Jones Hill

Table 1 (continued): Details of Data Elements and Sources of Information

Data Category	Data Type	Contact person(s)
CLIMATIC DATA	<u>For each Weather Station and Year</u> Average Precipitation Average Temperature	Ken Scheeringa, Indiana Climate Data Center
CONSTRUCTION AND DESIGN FEATURES	<u>For each segment</u> Base compaction level Joint spacing Joint width Dowel diameter	
OTHER DATA	<u>For each segment & year:</u> Age Functional class NHS classification Number of lanes	Bill Flora Mahlon Bartlett Nayar Zia