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of Transportation
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Analysis of PMS Data for Engineering Applications

REFERENCE MANUAL

NHI Course No. 131105

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PREFACE

INTRODUCTION

In 1996 TRDI was awarded an FHWA contract to develop and to conduct a training course entitled "Pavement Management Analyses – Engineering Applications". The preparation of a Notebook for Participants was part of this contract. After an extensive stage of gathering information from several state DOT's in close cooperation with FHWA staff and preparing demonstration material, the first training course was given in July 1998 in Chicago. After this pilot the course has been given also in Austin (Texas), Los Angeles, Kansas City, and Newark (New Jersey). Each course was attended by 25 to 35 representatives from DOT organizations in that particular region. Several developments since the start of the projects led to changes and additions to the course material, and consequently the need arose for updates in 1999 and 2000.

In July 2000, it was decided by the FHWA to add two more demonstration sessions, and consequently courses were organized in Salt Lake City, Utah with 33 participants, and in Nashville, Tennessee with 35 participants, both were held in November 2000.

The current Reference Manual evolved from the Participants Notebook as a result of a new contract in 2002 with the FHWA to modify the course into a typical NHI (National Highway Institute) format, and at the same time to update the course material with some recent examples of Engineering Applications of PMS data. The NHI format also required a new Instructor Guide and a new Participant Workbook.

The course material is divided over 10 Modules in two days and structured as following:

- Module 1: Overview of the course and of Engineering Applications of PMS, followed by an example from Arizona DOT by way of introduction.
- Modules 2: Overview of Database Needs and Elements for Engineering Applications,
- Module 3: Superpave Performance Monitoring with experience from Maryland SHA, Indiana DOT, Florida DOT, Arizona DOT and Washington State DOT.
- Module 4: Performance and Pavement Modeling with examples from PennDOT and Washington State DOT. Each time a particular State DOT is featured, its example will be preceded of an overview of its PMS.
- Module 5: Workshops on Engineering Applications of PMS
- Modules 6: Pavement (Overlay) Design Evaluation Analysis, with examples from the DOT's of Washington State, Texas, and Arizona,
- Module 7: Pavement Materials and Construction Performance, with examples from Arizona, Texas and Kansas,
- Module 8: PMS for Tracking Preventive Maintenance Actions, with an Introduction and a Summary of a study by Dr. Baladi in 6 State DOTs, followed by Engineering Applications in Texas and Wisconsin,
- Module 9: Pavement Preservation Strategies, with examples from Pennsylvania, and a Life Cycle Cost Analysis from Montana,
- Module 10: Effect of Maintenance on Pavement Performance with an overview of maintenance activities and strategies, the concepts of maintenance management systems, and an example of integration of maintenance and PMS in Montana DOT,
- Modules 5a and 5b: Workshops for course participants, one on each day.

Since the demonstration material for the original Notebook was collected several years ago, some of it might look dated. For the purpose of this course such examples are still very relevant because they demonstrate the main principles of using PMS and PMS related data for engineering applications.

OBJECTIVE

The objective of the course is to encourage agencies to enhance their PMS and PMS related databases and use these to their advantage for a range of engineering applications.

The objective of this Reference Manual is to provide participants of the course with a memento of the subjects discussed, relevant guidance for developing their own engineering applications based on PMS related data, and references for further study.

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1 OVERVIEW OF ENGINEERING APPLICATIONS OF PMS

1.1 INTRODUCTION

Pavement Management can be defined as:

> A coordinated systematic process for carrying out all activities related to providing pavements.

A Pavement Management System can be defined as:

> Rational procedures that provide optimum pavement strategies based on predicted pavement attributes incorporating feedback regarding the various attributes, criteria, and constraints involved.

Pavement Management Software can be defined as:

> A set of tools to assist decision-makers in preserving a pavement management system

Pavement Management Components are:

- Pavement condition analysis;
- Maintenance and rehabilitation needs analysis;
- Optimize budget allocations;
- Prioritize maintenance and rehabilitation projects;
- Select best life cycle strategies;
- Design pavement structure;
- Program and track routine maintenance.

Some essential characteristics of pavement management are:

- Formalization of pavement decision making
- Entire process to provide quality pavements
- Uses advanced tools and analysis techniques
- Strong emphasis on economics
- Involves all associated groups
    - Planning of pavement activities,
    - Design of pavements,
    - Construction of pavements,
    - Maintenance of pavements,
    - Materials for pavements,
    - Field Groups.

A 1967 NCHRP project was one of the first efforts in applying systems engineering to pavement design [Hudson 68]. In a similar but independent effort [Hutchinson 68], a systems approach was applied to structuring the overall problem and several of the subsystem design problems. Simultaneously, the Texas Transportation Institute developed a working design model in connection with a cooperative research project with the Texas Highway Department. As a result of these studies, the Texas Highway Department, recognizing the need for a system for organizing and coordinating their pavement research program and updating their design system, initiated a project in cooperation with The University of Texas Center for Highway Research and the Texas Transportation Institute of Texas A & M University [Hudson 70].

In essence, the goal was to "formulate the overall pavement problem in broad conceptual and theoretical terms"; which would enable the solution of a variety of pavement problems that had long plagued engineers. Over the years it has gradually become obvious that a PMS can offer many benefits, such as:

- It is possible to make good, well-informed, and consistent decisions on maintenance, rehabilitation or reconstruction;

- Effects of different levels of funding can be properly evaluated and quantified;
- A properly used PMS will result in longer pavement life;
- The available funds for maintenance and rehabilitation can be extended to cover more needs;
- The overall efficiency of pavement operations, use of materials and resources, planning activities, etc, is improved;
- A good PMS permits objectivity regarding the performance and the level of service of a pavement network.

1.2 THE GENERAL STRUCTURE OF SYSTEMATIC PAVEMENT MANAGEMENT

The general structure of a systematic pavement management system contains a number of coordinated modules at several organizational levels accessing a common database. A graphic representation of this concept is given in Figure 1.1

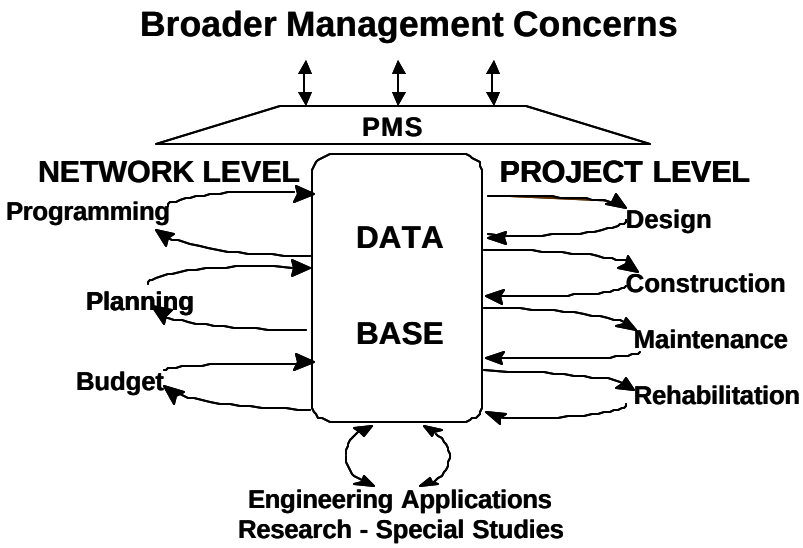

Figure 1.1 Components of a Pavement System

Network level management has as its primary purpose the use of engineering applications to development of a priority program and schedule of rehabilitation, maintenance or new pavement construction work, within overall budget constraints. Project level work comes "on stream" at the appropriate time in the schedule and incorporates additional engineering applications detail to address individual pavement sections and their specific problems.

Early efforts to introduce a systems engineering concept to the road pavement field were often directed at the project level. As an example, the design process at the project level was structured and its components were identified more specifically in 1965-67 as shown in Figure 1.2 [Hudson 68]. Tis detailed version was simplified into Figure 1.3 as a summary representation. The components of Figure 1.2 are further discussed as follows:

1. *Inputs*. Inputs, including a number of different variables, plus objectives, must be established.
2. *Models*. The need for analysis of alternatives was identified. A simple model of the pavement response expressed, for example, as a "simple design chart" does not adequately treat improved pavement materials.
3. *Behavior ¾ Distress*. Most pavement models calculate or predict pavement response such as stress or strain. Given the prediction of response, if carried to its limit the result will be distress. Prediction models for pavement response, and thus for cracking and other pavement distress, are essential.
4. *Performance ¾ Output Function*. Accumulated distress reduces pavement serviceability, and serviceability history defines pavement performance.

5. *Safety*. Skid resistance, rutting and other safety issues are important.
6. *Costs*. Life-cycle economic analysis is a vital part of the pavement management process.
7. *Decision Criteria*. Closely tied to economics are decisions on allowable costs versus the resulting benefits related to a particular choice. These factors must be explicitly defined and considered in the analyses.
8. *Compare ¾ Optimize*. Selecting the optimal alternatives or strategy is an important step in decision-making.
9. *Implementation*. Construction of the selected alternatives or strategy, and periodic maintenance plus rehabilitation when required constitutes implementation. Another view of implementation is that of implementing a pavement management system itself, as opposed to implementing a decision made within the PMS.

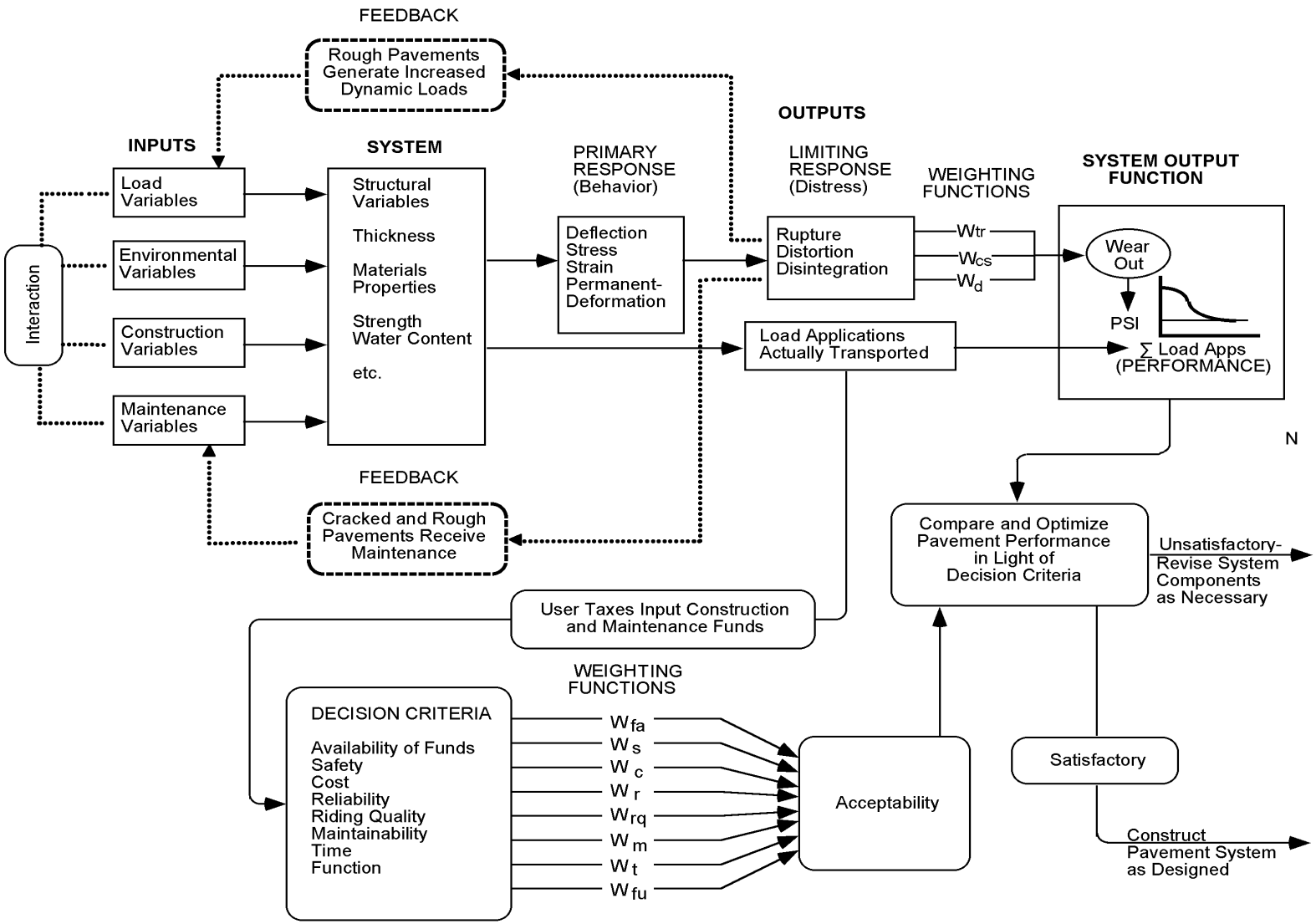

Figure1.2 Early Definition of a Project – Level Pavement Design System and its Major Components [Hudson 68}

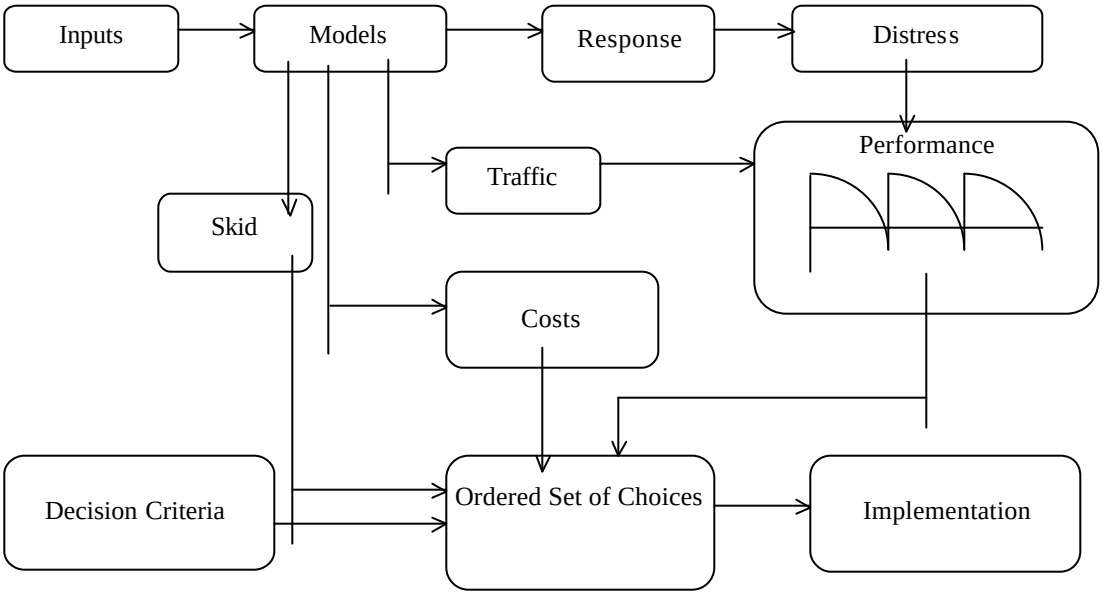

Figure 1.3 Simplified Diagram of the Major Components of a Project-Level Pavement Design System [Hudson 68]

1.2.1 Applying Systems Engineering to Pavement Behavior

A pavement is a complex structure that is subjected to many diverse combinations of loading and environmental conditions. Adding to this complexity is materials characterization, varying pavement performance and their interrelationships. These engineering factors are vital to a complete analysis. Because of this, a coordinated framework applicable to the overall problem of providing pavements is needed. Examination of available techniques for analyzing such complex relationships revealed that the concepts of systems engineering (which evolved in the electronics, communications, and aerospace industries) are quite appropriate to pavement structures. The use of systems engineering does not, per se, lead to a new and dramatic solution to pavement problems, but it does provide a means of organizing the various elements of the total problem into an understandable framework. Pavement management systems provide this framework that can incorporate engineering applications intended to improve the overall process of cost effectively providing safe, smooth pavements.

The scientific and engineering aspects of a systems problem usually span a broad spectrum of activities:

1. Use of physical observations or measurements to characterize behavior,
2. Statement of mathematical models that describe or approximate the physical observations,
3. Development of a system to predict behavior using the mathematical models, and
4. Construction of the design.

Thus, it is essential that systems engineers are able to formulate a system in terms of a mathematical or physical model, or, failing this, the system must be simulated in some realistic way to observe the necessary outputs.

1.2.2 Pavement Behavior and Performance

Pavement management systems provide extensive abilities, through engineering applications, to improve understanding of pavement performance and incorporate this improved knowledge back into the overall pavement

design and management process. Examination of the pavement design system diagrammed in Figure 1.2 illustrates the complex interrelationships that necessarily exist between (1) materials comprising the structure, (2) manifestations of pavement behavior, and (3) pavement performance. A good deal of effort has been directed to trying to develop improved models for relating behavior to the serviceability-age history of the pavement (its performance). The problem is difficult but is well worth pursuing. It should be realized that good materials evaluation and good mechanistic modeling of pavement structures is an essential part of pavement management.

1.3 THE PROCESS OF PAVEMENT MANAGEMENT: LEVELS, DECISIONS AND ISSUES

Pavement management is a process for carrying out in a coordinated, systematic way all those activities that go into providing pavements [Haas 77]. It can be viewed in terms of two basic working levels: network and project. Both of these levels incorporate engineering criteria and processes to tie the systems analysis process to the real world problem of providing long lasting pavements that provide good service.

Pavement questions and issues can be viewed from three basic organizational or user levels: legislative, administrative, and technical, whereas pavement management actually operates at two major levels: network and project. The network level is the primary responsibility of administrators, who also work with input of an engineering nature, while the project level essentially involves more detailed engineering considerations and decisions. The legislative level also requires answers from pavement management and these can range from general network impacts of budgeting to plans for specific projects.

Network level pavement management, where rehabilitation and maintenance work, due to limited budgets, is handled through a priority analysis, involves a "from the top down" flow. In other words, individual projects come on stream from the priority program established at the network level. However, project level activities are equally important to the overall process. Moreover, "from the bottom up" data are vital to the updating of network level estimates. The network level works on much more approximate data and technical analyses, and, therefore, considerable "fine tuning" may be required when individual, detailed project testing, design, and construction are done.

Thus, the structure of pavement management incorporates a project level and a network level that are both directed to providing answers to legislative, administrative and technical level users within an organization. The major issues and questions that require these answers are identified in the next section.

1.4 THE PMS DOES NOT MAKE DECISIONS – THE PEOPLE DO

It will not likely ever be possible to rely entirely on quantitative criteria and decision rules for engineering projects. Qualitative judgment is an important element in human endeavors; thus, the system or the computer used in the system does not make decisions; rather, it processes the information for use by decision makers. It is the people who use a PMS that assumes the responsibility for making decisions.

In order to make these decisions, the legislative, administrative and technical level users of a pavement management system face certain issues and questions. These may differ in focus and scope, depending on the agency (i.e., federal, state/provincial, city, and county), and the management level involved. Many of the issues at all three levels require engineering analysis of pavement management (secondary engineering applications) data to be adequately addressed. The following paragraphs provide examples under these three categories.

1.4.1 Legislative Level Users

The issues and questions at the legislative or elected level are fairly broad in scope but have to be recognized by the administrative and technical levels. They include the following:

1. *Justification of Budget Requests*: Legislators are faced with a variety of competing demands and those that "make the case" in a clear, properly supported manner are likely to receive more favorable consideration.
2. *Effects of Less Capital and/or Maintenance Funding*: Legislators may well ask what the short and long–term effects are of fewer funds, perhaps even a zero capital budget, on the deterioration of serviceability, extra maintenance costs, eventual replacement costs, effects on users, etc.
3. *Effects of Deferring Work or Lowering Standards.* The related question to lower funding is that of deferring maintenance and rehabilitation, and/or lowering the standards.
4. *Effects of Budget Requests on Future Status of the Network:* If a funding level matching the budget request is approved, a key question relates to the effect this will have on the status of the network. Will the average serviceability or condition decrease, improve, or stay the same? Alternatively, the question may be asked as to what level of funding is required to keep the network in its present state.
5. *Effects of Increased Load Limits*: This is an example of the type of issue facing many legislative bodies. Obviously a good pavement management system should be able to supply the technical and economic answers.

1.4.2 Administrative Level Users (Secondary Engineering Applications)

The administrative and planning people responsible for developing capital spending and maintenance programs (i.e., state/provincial highway administrators, division heads for planning, maintenance, city engineers) need to explicitly recognize and respond to legislative level issues and require certain answers from the technical level, in addition to facing questions at their own level. In other words, there is overlap in both directions at this level. Some example issues and questions include the following:

1. An objectively based priority program to provide justification for budget requests,
2. A summary assessment of the current status of the network, in graphical and tabular form, based on information from the database,
3. The means for quantitatively determining the effects of lower budget levels, and/or the budget level required to keep the network in some specified state,
4. The means for quantitatively demonstrating the effects of deferring maintenance or rehabilitation,
5. Estimates of the future status of the network (in terms of average serviceability, condition, safety, etc.) for the expected funding,
6. Benefits of a pavement management system, and its major features or "deliverables", etc.
7. Costs of pavement management implementation, including database development, manpower requirements; system development and installation, training, on-going costs, etc.
8. Implementation of experience of others; documentation of their experience.
9. Relationship between pavement management and other management systems, including maintenance.
10. Interfacing a pavement management system with transportation management in general.

1.4.3 Technical Level Users (Primary Engineering Applications)

From a technical perspective, pavement management involves a large number of issues and questions. In addition, the questions and issues faced at the administrative and legislative levels must be appreciated if technical activities are to be meaningful.

The following is an example of some of these key issues and questions, involving both the network and project levels:

1. Database design and operation, plus the methods, procedures and equipment for data acquisition
2. Ensuring the adequacy of the database.

3. Models for predicting traffic, performance, distress, surface friction, etc.–their reliability, consistency, reasonableness, deficiencies, etc.
4. Criteria for minimum serviceability, minimum surface friction, maximum distress, minimum structural adequacy–reasonableness, effects of changes in criteria, etc.
5. Models for priority analysis and/or network optimization.
6. Verification of models.
7. Relating project (sub) optimization to network optimization.
8. Methods for characterizing materials and using results.
9. Sensitivity of model analysis results to variations in factors.
10. Relationships between vehicle operating costs and pavement characteristics.
11. Construction quality control.
12. Effects of construction and maintenance on pavement performance.
13. Communication between design, construction and maintenance, within the organizational structure.
14. Guidelines for pavement management implementation.
15. Relating pavement management to maintenance management.
16. Improving the technology of pavement management and making use of implementation projects for this purpose.

1.5 THE IDEAL PAVEMENT MANAGEMENT SYSTEM (PMS)

A system in general consists of a set of mutually interacting components that are affected by certain external factors or inputs. In the physical pavement system (engineered structure) the mutually interacting components are usually a surface layer including traffic lanes and shoulders, base layer, subbase layer, and subgrade. The external or outside factors, which affect the pavement, are environment, traffic, and maintenance. Maintenance is carried out to reduce the rate of deterioration of the pavement from the negative impacts of the traffic and environmental inputs.

A pavement management system on the other hand consists of such mutually interacting components as planning, programming, design, construction, maintenance, and rehabilitation. All of these require application of engineering principles to be properly analyzed. External factors affecting a pavement management system include budgets, information, and non-quantifiable administrative policies.

An ideal pavement management system would yield the best possible value for the available funds while providing and operating smooth, safe, and economical pavements. The minimum requirements of such a system would include adaptability, efficient operation, practicality, quantitatively based engineering decision support, and good feedback information. There is no ideal single PMS that is best for all agencies. Every agency represents a unique situation with specific needs. Therefore, each agency must define carefully what it wants from a pavement management system.

1.6 INFLUENCE LEVELS OF PMS COMPONENTS

Four of the major components or subsystems (planning, design, construction, and maintenance, plus rehabilitation) have important but changing impacts in terms of a "level-of-influence" concept. This concept, which has been used in sectors of industry, such as manufacturing and heavy-industrial construction [Barrie 78], shows how the effect on the total life cycle cost of a project decreases as the project evolves.

Figure 1.4 illustrates the essential features of the "level-of-influence" concept. The lower portion of the figure presents a simplified picture, in bar chart form, of the length of time each major component acts over the life of a pavement. The upper portion shows plots of increasing expenditures and decreasing influence, again over the pavement life. Expenditures during the planning phase are relatively small compared with the total cost. Similarly,

the capital costs for construction are a fraction of the operating and maintenance costs associated with a pavement life cycle. However, the decisions and commitments made during the early phases of a project have far greater relative influence on later required expenditures than some of the later activities.

At the beginning of a project, the agency controls all (100 percent influence) factors in determining future expenditures. The question is to build or not to build? A decision not to build requires no future expenditure for the project. A decision to build requires more decision making, but initially at a very broad level. For example, should it be a flexible pavement or a rigid pavement, and, if rigid, with joints or continuously reinforced? How thick should it be and with what kind of materials?

Once decisions are firm and commitments are made, the further level of influence of future actions on the future project costs will decrease.

In the same manner, decisions made during construction, even within the remaining level of influence, can greatly impact the costs of maintaining or rehabilitating the pavement. For example, lack of quality control or substitution of inferior materials may save a few dollars in construction costs, but the extra maintenance costs and user delay costs due to more frequent maintenance activities may consume those "savings" several times over.

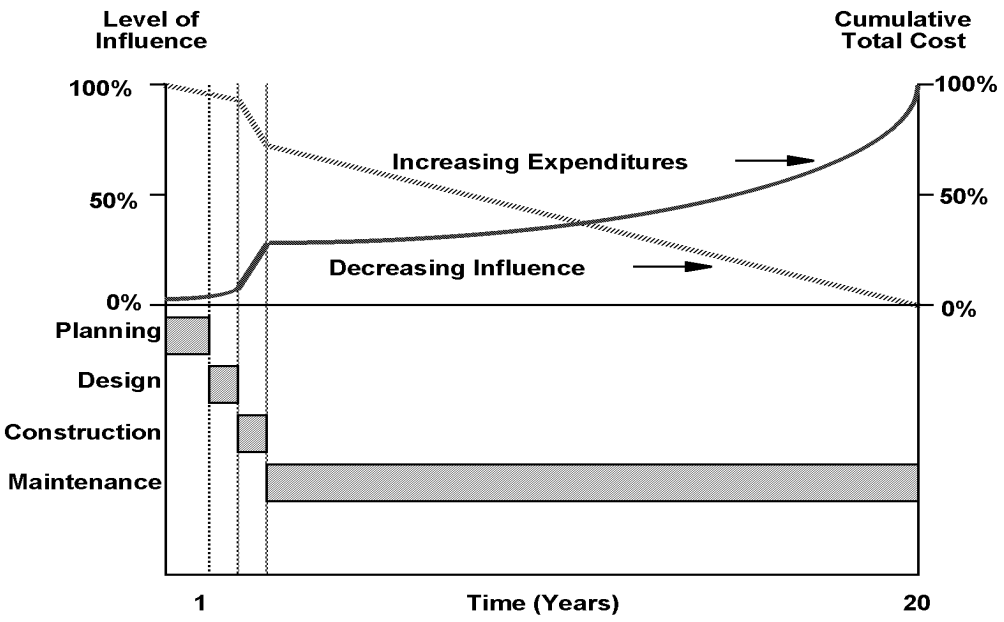

Figure 1.4 Influence Level of PMS Subsystems on the Total Costs

With construction completed, attention is now given to maintaining the existing pavement at a satisfactory level. The level-of-influence concept can also be applied to the subsystems of a Maintenance Management System (MMS). Expenditures during the planning phases of rehabilitation are relatively small compared to the total maintenance cost. However, the decisions and commitments made during early phases of a rehabilitation project have far greater relative influence on what other maintenance expenditures and user costs will be required later.

1.7 PAVEMENT AT THREE LEVELS

The various activities in a PMS, including decision making, have been generally categorized into the network level and the project level. The network level includes two sub-levels, the project selection level and the program level. The project selection level involves prioritization to identify which projects should be carried out in each year of the program period. At the program level, budgets are established and general allocations made over an entire network.

In agencies where these three levels exist, decisions at each level must include relevant engineering applications and interaction between project selection level and program level. This is evident in the case of estimating the budget required at the program level because it requires information on all the candidate projects.

This three-level concept is illustrated in Figure 1.5. The lower-left triangle represents an area of unreliability because too little information is available for models at the project level, and the upper-right triangle is an area unfeasible for modeling due to the size and complexity of the required models.

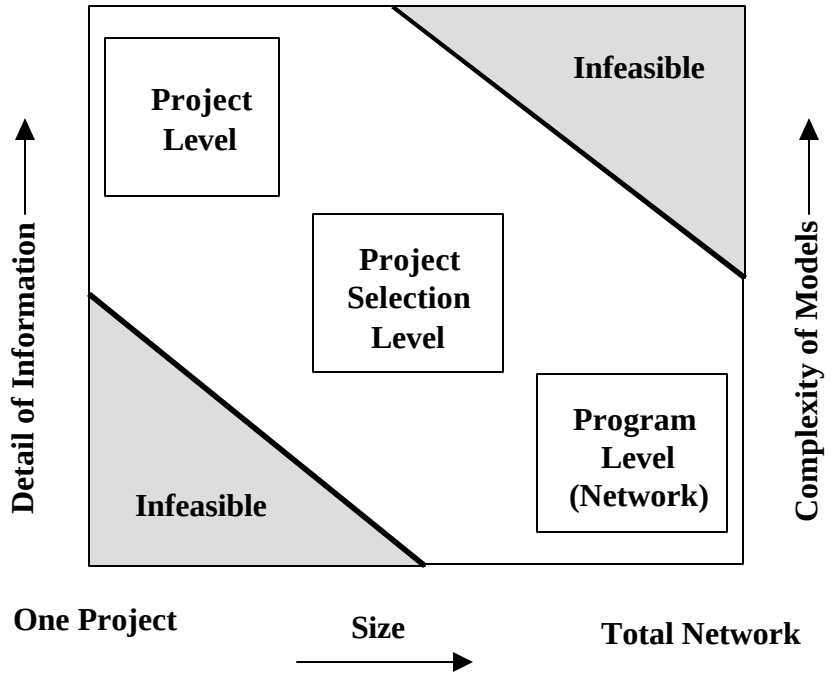

Figure 1.5 Information Detail and Complexity of Models for a Three Level PMS

1.7.1 Project Level (Primary Engineering Application)

The PMS models at the project level deal with technical concerns, such as detailed design decisions, for an individual project. As such, they require detailed information on specific sections or subsections of pavement as shown in the upper left part o f Figure 1.5.

The inputs for project level models include load, environmental factors, materials characteristics, subgrade properties, construction and maintenance variables, and costs. What specific data, and in what form, depends on the nature of the models. The typical output from such models would be a set of design strategies that minimize total life-cycle costs, including construction, maintenance, and user costs, while satisfying user, physical and administrative constraints, such as performance requirements, minimum or maximum thicknesses, and funds availability.

One of the first major working PMS models developed at the project level was the Flexible Pavement Design System (FPS) [Scrivner 68]. This computerized model had several unique features, including the ability to separately calculate deterioration due to traffic and environment and a life cycle cost analysis subroutine.

Similarly, the Rigid Pavement Design System (RPS) [Hudson 72] was the first available working PMS model relating to rigid pavements at the project level. Subsequently, the System Analysis Method for Pavements (SAMP) [Lytton 75] was developed. It has a broader application than most existing working systems at the project level. Another working project level system with similar features to FPS and SAMP was the Ontario Pavement Analysis of Costs (OPAC) System developed in Ontario [Kher 70]. Any of these system models are still extremely useful guides to any agency setting up a pavement management system at the project level.

1.7.2 Project Selection Level (Primary Engineering Application)

The project selection level involves decisions on funding for projects or groups of projects, as opposed to the program level that involves general budget allocation decisions for an entire highway network. The models employed at the project selection level are geared to less detailed data for a set of projects under consideration than ones at the project level, as shown in Figure 1.5. These models, under budget constraint conditions, involve a decision process to select feasible treatments for specific projects and prioritization models based on optimization, near optimization or other techniques.

1.7.3 Program Level (Secondary Engineering Application)

The program level involves policy decisions regarding rehabilitation or maintenance for the network as a whole. At this level, allocation of budgets is the major concern, and the models should be designed to optimize the use of funds allocated to rehabilitation and maintenance. The intent should be to consider the status or condition of the network in terms of average serviceability or amount of deficient mileage. In order to do this, data is needed to determine the existing condition of the network as a whole so that the effects of different rehabilitation and maintenance policies and standards can be evaluated.

The network-level of pavement management, as commonly used, incorporates both the project selection and program levels identified in this section.

1.8 KEY CONSIDERATIONS AND ESSENTIAL PMS REQUIREMENTS

There are several key considerations in applying an engineering based pavement management system concept, including the following:

1. The need for precise, understandable definitions, and a framework or structure that allows flexibility to use particular practices or methods.
2. The need for people in the agencies who have qualifications appropriate to the various activities of pavement management, such as economics, structural analysis, computing, statistics, field measurements, etc.
3. The need for effective incorporation of all technical, economical and other factors.

4. A well-developed interfacing mechanism between the policy level of transportation management and pavement management at the network level; also, properly coordinated interfacing between the network and project levels of pavement management.

5. A well-developed interfacing mechanism between maintenance management and other areas of pavement management.

Other essential PMS requirements are:

- Easily updated/modified as new information and models become available;
- Considers alternative strategies;
- Identifies optimum strategy;
- Rational procedures with quantified attributes, criteria, and constraints;
- Uses feedback information regarding the consequences of decisions.

1.9 THE FUNCTION OF PAVEMENT EVALUATION AND FEEDBACK

Evaluation is a key engineering component of pavement management because it provides the means for seeing how well the planning, design, and construction objectives have been satisfied. It is used in a broad sense at this point and is directed to measuring and assessing the outputs of a pavement. Figure 1.6 is a schematic representation of the major types of pavement outputs vs. time. Some or all of these outputs would be predicted in design, and would then be actually measured as part of the evaluation when the pavement is in service.

In Figure 1.6 the surface distress output has reached a limit of acceptability before any of the other outputs. At this point, the end of the initial service life of the pavement, some rehabilitation measure has been implemented, as shown by the vertical discontinuity. The rehabilitation measure has also been shown to affect the other outputs, such as increased structural adequacy, improved serviceability, improved surface friction, lower maintenance costs, and lower user costs.

The function of pavement evaluation in a PMS can be summarized as that of measuring and assessing these outputs periodically in order to:

- Provide data for checking the design predictions and updating them if necessary
- Reschedule rehabilitation measures as indicated by these updated predictions
- Improve design models
- Improve construction and maintenance practices
- Updating network programs

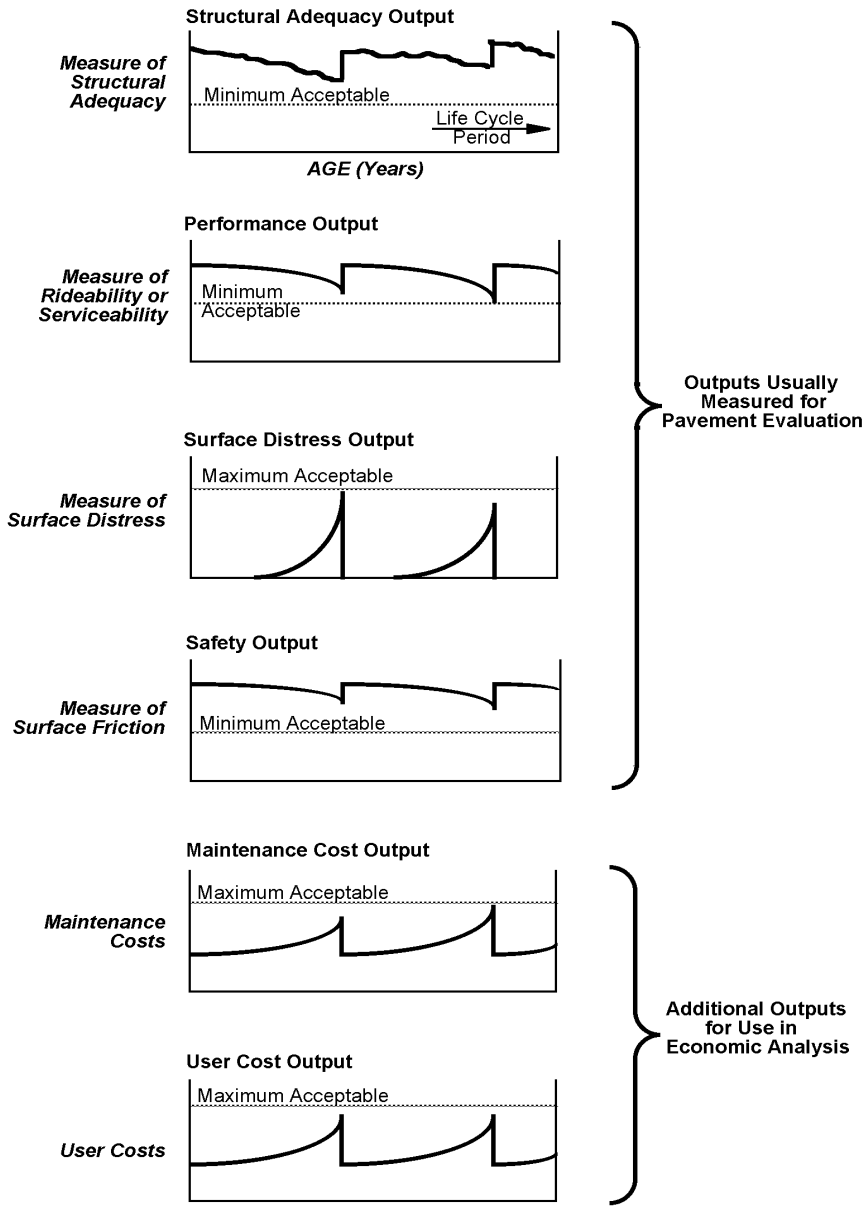

Figure 1.6 Major Types of Pavement Outputs

1.9.1 The Uses of Evaluation Information: An Illustration

An Illustration to show more explicitly the uses of evaluation information is given in Figure 1.7. It is split into an upper and lower half, the upper half including predictive models along with inputs and outputs, and the lower half showing examples of periodic measurements.

The input variables of Figure 1.7 include the physical structure and material strengths, and can be monitored by physical testing and sampling to provide direct information about layer thicknesses and materials properties. Load

testing of the total pavement structure is not included because it involves pavement response or behavior of the pavement and not the input information itself.

Safety response could in a broader way be categorized in a general response system; however, it is amenable to treatment in evaluation alone and is normally handled by measuring the surface friction or skid resistance of the pavement.

Costs can be monitored only if records of expenditures, including initial and future construction overlay costs and subsequent pavement related maintenance expenditures are accumulated.

Behavior can be defined as the immediate response of the pavement to load. Thus, deflection tests of all types fall into this category. It should be noted that these load-testing techniques evaluate only the behavioral response of the pavement and not the physical properties directly.

Distress can be defined as limiting response or damage in the pavement. Thus, the accumulated damage that the pavement has suffered is monitored and evaluated. Because maintenance may have been performed on some of the distress, the evidences of this maintenance in the form of patches and sealed areas should also be monitored. Such monitoring is done routinely by many agencies in the form of condition surveys.

Another category of pavement evaluation of major interest is pavement performance. Because performance is the serviceability or condition history of the pavement, its evaluation implies a time-related accumulation of data. This is best accomplished by periodic measurement or monitoring of the condition of the pavement. At the very minimum, evaluation of performance requires two pavement condition estimates: one at the time of evaluation and a previous one, usually at the time of construction. A single measurement at the current time can tell the pavement engineer whether or not the pavement is below the currently desirable level. The change in condition from some prior time provides information about the performance history.

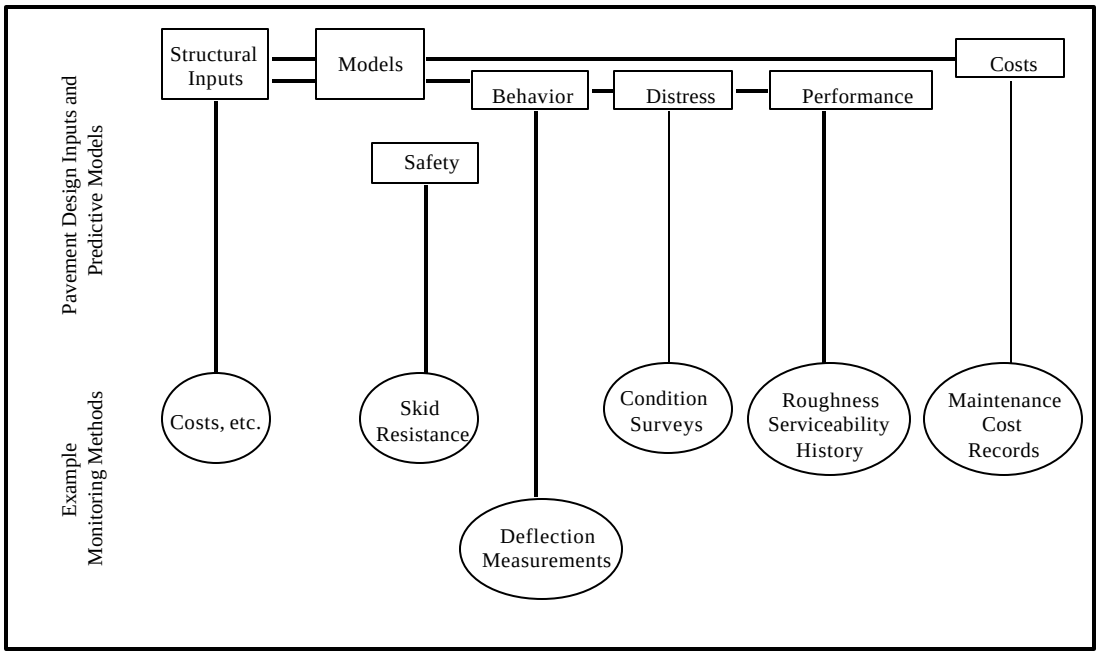

Figure 1.7 Simplified Prediction Portion of Pavement Design and Related Examples of Types of Periodic Evaluation Measurement

The foregoing pavement evaluation measurements interact and there is overlap among them. However, they should not be confused or used interchangeably. For example, the fact that some people evaluate serviceability level using a serviceability equation that includes cracking and patching terms does not mean that the equation provides an

adequate evaluation of distress and performance. A serviceability equation is primarily a performance evaluation tool. Behavioral measurements can be used to estimate inputs only in conjunction with some type of theory or model.

1.9.2 User-Related Evaluation versus Engineering Evaluation

Among the types of pavement evaluation, most agencies consider the following four as most important: serviceability, structural adequacy, surface distress and safety. Safety is primarily in terms of surface friction but can include such factors as deep ruts, which may affect steering conditions, hydroplaning potential, and icing potential. It is clearly a user-related measure. The other three types of evaluation, however, have often been confused and erroneously interchanged. They should be considered in terms of:

1. Functional behavior, using the serviceability-performance concept, and
2. Engineering or mechanistic types of evaluation of the pavement structure.

In general, serviceability-performance is concerned with the overall function of the pavement; that is, how well it performs its function as a riding surface for vehicular traffic. By and large, this is also the area of main concern to the pavement user.

On the other hand, the mechanistic evaluation of pavement structures is of vital interest to engineers. An understanding of the interrelationship between these two types of evaluation is also vital. Whereas serviceability is a measure of present condition, the mechanistic or structural evaluation is used to estimate the future response of the pavement to load. A weak, cracked pavement can illustrate the relationship today with high serviceability, but the prospects are for a rapid loss in serviceability.

1.9.3 Pavement Evaluation with Respect to User Costs

Decisions to improve pavements in practice are almost always based on structural, serviceability, distress, safety, or maintenance cost considerations. However, excessive user costs arising from surface characteristics can in some situations be an equally valid criterion for improvement needs and can in fact form the basis for determining the priorities of such needs.

Most past and indeed current practice has been to consider only capital, maintenance, and engineering or administrative costs, with the implicit assumption that user costs do not vary with level of serviceability, surface distress, and extent and time of the improvement. In other words, all strategies are assumed to result in equal user costs. In fact, such user costs may vary significantly and they should not only be evaluated, but also used in the economic analysis of alternative strategies.

1.10 ENGINEERING ANALYSIS APPLICATIONS OF PMS DATA

1.10.1 Purpose and importance of PMS Engineering Analysis

The purpose of PMS engineering analysis can be described as:

> The use of pavement management data to evaluate and improve structural designs, materials, mix designs, construction, preservation strategies, rehabilitation, and preventive maintenance of pavements.

Engineering Analysis is an essential part of PMS. Since pavements are engineered structures, engineering analysis can be used as a major tool for the following:

- Proper use improves pavement performance;
- It can be used for both network, project level or for individual problems;
- Its use is essential for feedback purposes;
- It affects future activities in many fields, such as design, construction, maintenance, standards, and specifications.

An overview of the various sources and the inter-relationship of PMS related data, and the flow of data are given in Figure 1.8. Note that in the case of pavement and performance modeling, the essential feedback loop of data for updating models is indicated with a dashed arrow.

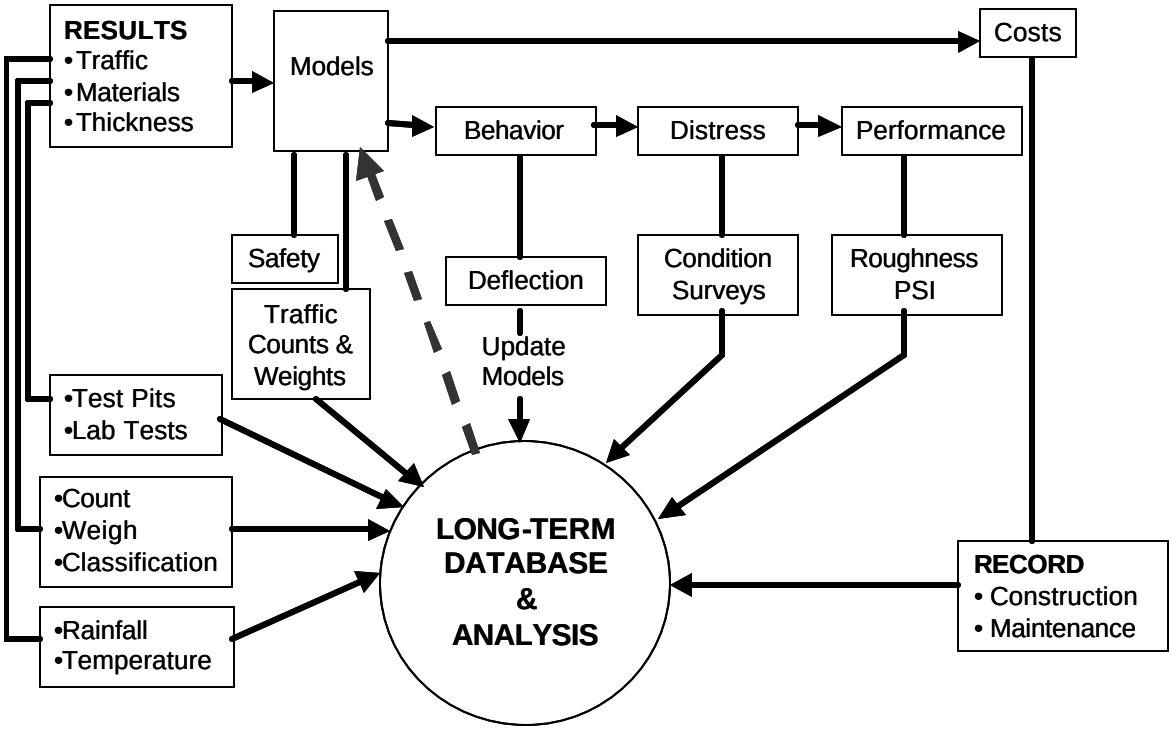

Figure 1.8 Flow Pattern of PMS Related Data

1.10.2 Network versus Project Level Data

The network versus project level discussion of pavement management is typical of most state and city PMS in current practice. In fact, however, most agencies including state DOT's place their emphasis on network-level pavement management data collection. Several states have been practicing network-level pavement management since the early 1980s. A few agencies do project level and project selection level data collection and design.

Many engineering applications will use both network-level data and project level information, whereby most performance data are available on the network level and most materials and construction data are kept in project files.

1.10.3 Topics for Engineering Applications

Engineering analysis of PMS related data can be performed for a wide range of subjects, provided the data are reliable and easily available. The topics that are relevant for a successful analysis are listed below.

- Database needs and elements. Sources of engineering data other than from the PMS database are:
  - Research data files
  - Construction records
  - Material test records
  - Additional field evaluations
  - Project plans
  - Additional structural evaluation and/or materials testing
  - Expert opinion
  - Maintenance management systems
- Performance and design model applications:
  - Performance models are essential at both network & project level;
  - Models should be developed from own PMS data, with AASHTO and other models as general back-up;
  - Performance models can be used to quantify benefits, to compare materials and techniques, to predict useful life, etc.
- Pavement (overlay) design evaluation and analysis using PMS data, in order to arrive at design improvements. The existing databases retain a wealth of experience for this purpose.
- Pavement preservation and rehabilitation strategies, mainly to:
  - Quantify the performance of various techniques and materials from PMS data and analysis;
  - Improve the future performance by improving these techniques and materials.
- Pavement maintenance performance. The current practice is:
  - Maintenance data are in most cases not fully incorporated in PMS;
  - With current data selection it is difficult to evaluate effectiveness;
  - Incompatibility of PMS and MMS
  - Maintenance modeling is hardly being used;
  - Problems in location referencing and recording data.
- Material performance evaluation. Some of the issues are:
  - Materials in pavement construction are truly diverse, new "miracle" materials are offered regularly;
  - Performance based specifications are not available for any material;
  - Analyze with PMS, identify material problems or good performance;
  - Improve use and specifications.
- Construction methods and specifications.
  - Analyzing existing data can lead to:
    - Improved construction techniques;
    - Improved quality; and
    - Improved specifications.
  - Most of these data can be found in materials and QC construction files. Information about this subject in the PMS database is often incomplete.

The examples given in this Notebook cover many of the topics listed above.

1.11 THE BENEFITS OF PMS FOR ADOT: A TYPICAL EXAMPLE OF ENGINEERING APPLICATIONS OF PMS DATABASES

1.11.1 ADOT's PMS Overview

The Arizona Department of Transportation (ADOT) has been maintaining a working pavement management system (PMS) since 1980. In 1980 this PMS system was state-of-the-art and it has provided valuable service to ADOT and the taxpayers.

Currently the total length of the pavement network for which ADOT carries responsibility is 7600 centerline miles. There are nine districts and a separate maintenance district for Phoenix. Arizona uses three climate zones: desert, intermediate, and mountains.

The pavement management branch has 11 employees and is headed by a pavement management engineer. Pavement management reports to the head of the Materials Group, together with the following branches:

- Pavement materials testing (mainly asphalt, no concrete)
- Structural materials testing (concrete et al)
- Pavement and overlay design (only asphalt, no concrete),
- Quality assurance, Geotechnical design and Geotechnical operations
- Four Regional Materials Laboratories.

The primary responsibility of the Pavement Management branch is to identify pavement preservation projects. This is done annually through the collection of pavement condition data at each mile point of the AZ highways (about 7,500 miles) with the help of the following:

- Surface distress data (cracking, rutting (since mid eighties), flushing, patching) measured through a visual survey of the first 1,000 sq. ft at each milepost,
- Skid resistance measured with a Mu-Meter,
- Roughness measured with a Mays Ride Meter, and since the mid nineties with a Profilometer in accumulated inches per mile,
- Structural properties measured with a Dynaflect, and since the early nineties with a Dynatest FWD, mainly for overlay design purposes.

Since the start of the PMS in 1980 ADOT uses a Markov chain model that forecasts the proportion of the road network that will change from one condition state to another during a given year. Originally this model only indicated the total sum of money required to improve the condition without specific project allocation. With the help of the gradually improved Network Optimization System (NOS) preservation projects are now selected on the basis of priority ratings (different weights for performance indicators like cracking, rutting, etc) and in this way money is allocated to each district in relation to their needs.

In 1980 the ADOT PMS preservation budget was approximately $ 35 million. This was based largely on the (low) maintenance budgets of previous years. To develop the network optimization, ADOT spent roughly $ 300,000 on

consulting services in 1979. Temporary staff was hired for a total of about 13 man-years to work on the PMS during its development. From 1981 the budget grew annually with some 10% to about $ 61 million in 1987, and since then the growth continued at a slightly slower pace (with fluctuations) to a current level of approximately $ 100,000. Since a few years all routine maintenance costs are excluded from this budget, in 1997 the maintenance budget was about $ 10 million.

1.11.2 Quantified Benefits of a PMS

The information in this section is based on a study carried out by TRDI for ADOT in 1998 [ADOT 99]. The proof of any PMS is its impact on the performance of the pavements, which comprise the roadway network. An assessment of this impact should include an evaluation of the effectiveness of the system to predict performance, effectiveness of various design strategies and materials, the improvement in the overall system condition, or an increase of pavement performance.

The problem is how to go about these analyses and how to best utilize the available data and the available enhancements. There may be missing data elements in this database as there are in any broad database. One of the problems to be resolved in this research is to generalize the analysis to account for such missing data and still produce effective results.

The objective of this exercise is to evaluate the impact of the ADOT PMS on pavement performance. To many pavement engineers the benefits of a pavement management system are self-evident, but often it is difficult to quantify these benefits in measurable terms. In this study the quantification of benefits could be done through statistical analysis of the ADOT PMS database for performance improvements, and relating these to budget and cost levels over the period considered.

1.11.3 Creating the Database

It is generally accepted that pavement management has been beneficial to most state highway agencies but a critical evaluation of those benefits and how they truly provide improved pavement performance, longer life, and lower maintenance cost is very important.

A number of possible approaches have been taken to evaluating PMS benefits in the past. One approach is to assemble a panel of experts to estimate the level of pavement performance, which was obtained before and which was observed after the pavement management system was installed. Several such studies have been carried out to estimate general effects of change. There is a strong chance of bias in such an approach since it is difficult for humans to remember what really existed 15 years in the past.

A second approach would make use of the data in the pavement management database to evaluate case study situations for various combinations of pavement variables. It is possible to compare a few selected case studies, but such case studies cannot cover the broad range of variables that must be evaluated. There is a danger of missing the true answer in this approach to the problem.

In the selected approach it was decided to make use of Visual Modeler [Hudson 95], developed by TRDI for the Pennsylvania DOT and the Federal Highway Administration. ADOT participated in a multi-state study to evaluate Visual Modeler, a functioning computer program that can take selected data from a pavement management database and produce performance prediction models for a variety of input conditions. The approach undertaken in this research was to use Visual Modeler for the ADOT PMS database and to assemble an analysis database for

a wide variety of pavement structures, environments, thicknesses, overlay types, and other factors, which exist within the state of Arizona. The work was started in 1998.

As the first step, the PMS database was evaluated for consistency, uniformity, and completeness. This evaluation showed a consistent available data set that could be analyzed effectively in Visual Modeler to show performance trends for the pavement. Specifically, we selected pavement performance indicators in the 1981/1983 timeframe as indicating the characteristics and performance of pavements in the "before PMS category". Ideally it might have been desirable to select pavements prior to 1980 but no significant PMS data is available prior to that time. However, during the first three years of PMS data collection the benefits of the pavement management process had not yet manifested themselves or their effect in improved pavement performance.

Initially it was intended to select the period 1995/97, the latest years available, for the "after PMS" pavement performance observations to provide the longest possible timeframe for comparisons. Analysis however, showed that data for the period 1996/97 had not been completely processed into the database; therefore a more realistic data set was selected for the period 1993/95 to represent the "after PMS" pavement performance status.

The data selection for both periods combined produced 40,321 data records or observations, which could be included in the analysis. A time series history regression of the 1981/83 data for pavement roughness and pavement age produced an equation and a line representing all data for this time period, an illustration of this with a limited number of data points is shown in Figure 1.9.

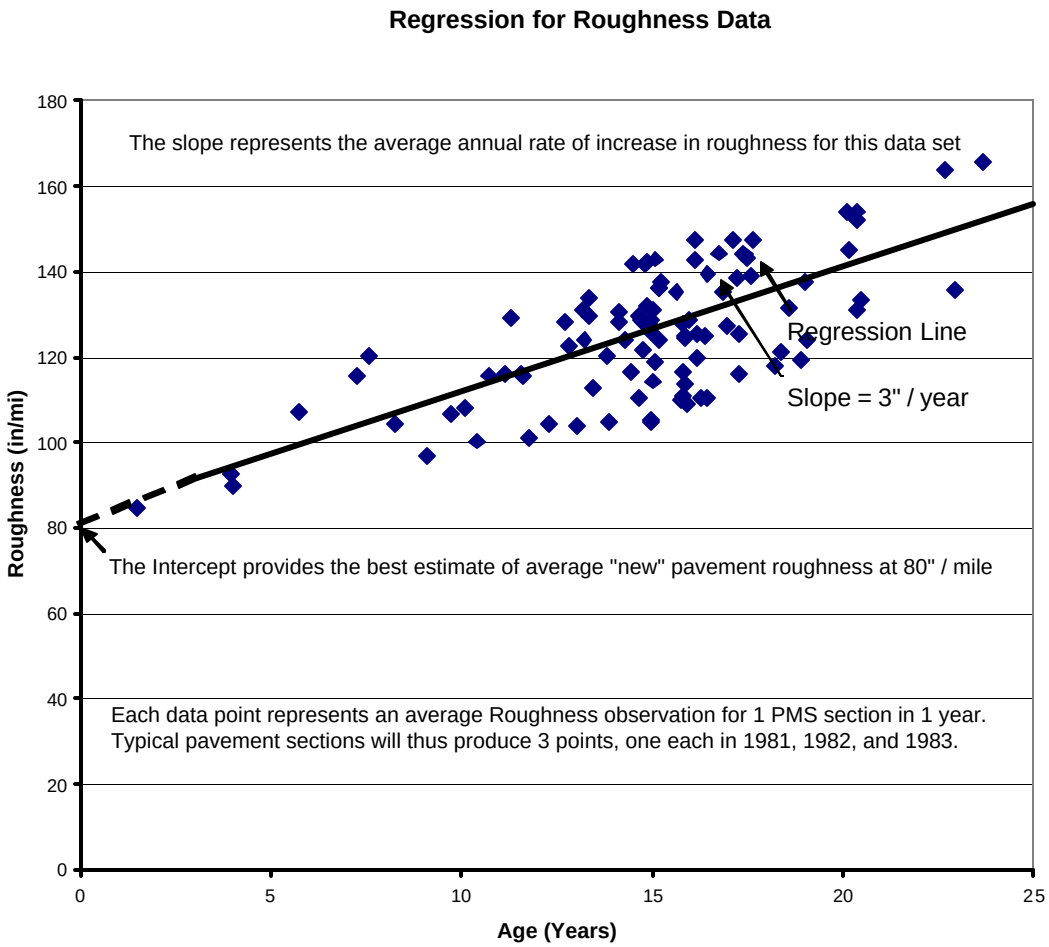

Figure 1.9 Illustration of the Time Series Regression for Roughness Data

A similar equation was developed for data for the time period 1993/1995. There were approximately 20,000 data points in each of these two data sets. Finally a complete regression was run which relates these two periods and

developed an estimated average initial roughness value for pavements for each period. The slope of this line describes the rate of change of roughness or the performance rate during these periods respectively. Since there are many different pavement sections, with different ages for any given measurement year in the database. There is a significant range of pavement age in the data set represented by about 7,000 records (data points) for each year for a total of 21,000 points for each three-year period.

While there is considerable variation in such a data set over a wide range of characteristics, traffic levels, environments, and other factors, the regression line for such a large data set can be unmistakably and our statistical analysis procedures can specify a significance level which defines the clarity with which the slope and intercept can be specified.

The analysis procedure is illustrated in the example in Figure 1.9. The example data set shown covers a group of pavements ranging in age from four years to 21 years for a range of roughnesses. The resulting regression analysis has an intercept or average initial roughness for age 0 (representing new construction) of 80 in/mi and a slope or increase in roughness of three inches per year of life. The reader is reminded that while only a limited number of points are shown here in order to keep from cluttering the plot, the actual analysis involves more than 20,000 points, for the analysis to define each period. Please note that in this example a linear relationship between Roughness and Age is defined. As you will see later, for the final analysis, a non-linear relationship was found to have a higher statistical significance.

Similar linear analyses were run for each of five performance indicators; roughness, cracking, flushing, friction, and patching, and two performance groups:

1. Road Structure Category (RSC) divided as follows
    a. All categories
    b. PCS New
    c. Asphalt Rehabilitation
    d. Non- structural (NS) rehabilitation and
    e. Asphalt New
2. Road class, interstate, non interstate, and all lumped together

The initial analyses show that roughness is by far the best indicator of performance and shows the best correlation for the data set. Cracking shows stable relationships, some of which are significant, but the relationships are not as robust as for roughness.

As a result of these preliminary analyses, roughness was selected as the primary performance indicator. Detailed regression analyses were run for six categories or types of pavement and road type. These analyses run used a regression model involving roughness, year group, and an interaction of age by year group, all as a function of age. The analyses were run for:

1. All pavement types in the state
2. All interstate pavements only
3. All rigid pavements only
4. All interstate rigid pavements only
5. All asphalt pavements only
6. All interstate asphalt pavement only

The details of all these results are published in reference [Hudson 01].

1.11.4 Summary of Analysis Results

It is important to note that all of the linear regression analyses run for roughness for all six of the road categories showed a strong significance of the important factors. There was consistency across all road types. Readers familiar with analysis of small data sets and large regression coefficient (R^2) may feel that the correlation in these analyses is low. A more detailed understanding of statistics shows that the significance of a correlation coefficient and therefore of a regression equation increases directly as the number of data points in the analysis increases.

With this many data points the assumption of predicting x from y where x=age and y=roughness, should not be a problem. [Ostle 88] There are indeed many examples of regression analyses on data from non-designed experiments. [Noffsinger 59] [Hanneman 71] [Peyton 78]
As shown in Table 1.1 the "P" value or the significance level for each of the factors for a data set of 40,000 data points is strongly significant at the 1% level, even for a linear regression.

Figure 1.10 shows the results for the linear regression, the dashed line representing the 1981/1983 performance trend and the solid line representing the 1993/1995 performance trend. Also shown are the tolerable roughness level of 93 inches per mile and the objectionable roughness level of 142 inches per mile, as specified by ADOT.

The intercept shows that average initial roughness for pavements in the 1981/83 period projects to be 68.3 inches per mile for the 1981/1983 period with a slope of 2.02 inches per mile per year. The 1993/95 data set shows an interceptor initial roughness of 64 inches per mile and a slope of 1.89 inches per mile per year.

This analysis shows that on average pavements were constructed slightly smoother during the 1993/95 period, after introduction and use of PMS, and that they remained smoother longer and performed better than pavements in the 1981/1983 time period (before PMS). A lower slope for roughness change shows that the pavement management process has produced improved pavement performance for ADOT pavements. The lower intercept or initial roughness shows that on the average overall, pavements are also being constructed smoother, perhaps as a result of better management and construction.

Figure 1.10 also shows that Pre-PMS pavements (1981/83) on average reached the maximum tolerable level at 12.2 years of age. For the Post-PMS (1993/95) period the age was 15.0 years.

It is also important to determine how good these estimates of life are. We, therefore, ran the 95% confidence interval for these equations. In figure 1.10, only the inside limit is plotted to simplify the plots. In the 81-83 Pre-PMS analysis, the 95% confidence interval is $\pm$ 0.5 years for a maximum life to tolerable roughness of 12.7 years.

For the 93-95 Full-PMS analysis, the 95% confidence interval is $\pm$ 0.3 years with a minimum predicted life of 14.7 years. Thus, the minimum predicted improvement is 14.7-12.7 or two years with 95% confidence. This would represent a 15.7% increase due to PMS.

Table1.1. Summary of Regression of Roughness for 1981 - 1983 (Pre-PMS) and 1993 - 1995 (Full PMS), All Roads, Age <30 Years

SUMMARY OUTPUT 81-83 (PRE-PMS)

Regression Statistics	
Multiple R	0.252177016
R Square	0.063593248
Adjusted R Square	0.063546635
Standard Error (Se)	40.00712408
Observations	20091

FORMULA

Roughness = a + b*(age)

RI = 68.3 + 2.02*(age)

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	2183632.012	2183632.012	1364.284001	5.2963E-289
Residual	20089	32153850.27	1600.569977		
Total	20090	34337482.29			

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept (a)	68.2972251	0.469852916	145.3587342	0
age2 (slope, b)	2.018358728	0.054644438	36.9362153	5.2963E-289

SUMMARY OUTPUT 93-95 (FULL PMS)

Regression Statistics	
Multiple R	0.384616649
R Square	0.147929966
Adjusted R Square	0.147887843
Standard Error (Se)	32.56025572
Observations	20230

RI = 64.8 + 1.89*(age)

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	3723140.502	3723140.502	3511.832644	0
Residual	20228	21445123.87	1060.170253		
Total	20229	25168264.37			

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept (a)	64.7689653	0.403588838	160.4825486	0
age2 (slope, b)	1.88585182	0.031822966	59.26071755	0

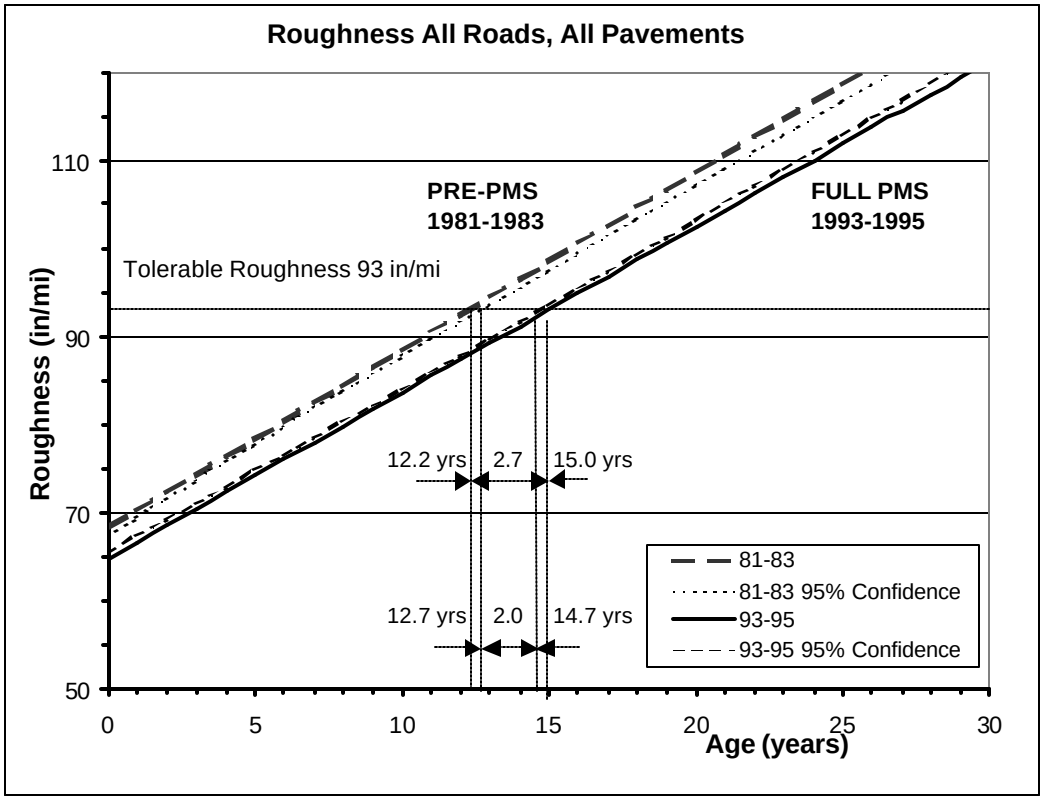

Figure 1.10 Roughness versus Age for the Periods 1981/1983 and 1995/1997

The significance of the relationship within one data set is important, but for a proper assessment of the benefits of PMS it is equally important that the two data sets are significantly different from each other. For a valid comparison between the two data sets the following requirements have to be satisfied:

- The distribution of roughness levels should be near to normal for both data sets,
- Check the variance levels of both sets, if they are similar one regression could be done for the combined set, if not separate regressions should be performed on each set.
- Determine confidence levels for each relationship and check possible overlaps.

As a first step the distribution of roughness data (frequency of discrete roughness values) was checked.

For the determination of the distribution of the roughness, the values for the roughness index (RI) were divided over intervals of 20 inches/mile covering the full range up to 500 inches/mile. This was done for both Pre-PMS and Full-PMS datasets. The results are given in Figure 1.11 for Pre-PMS (1981-1983) and Figure 1.13 for Full-PMS (1993-1995).

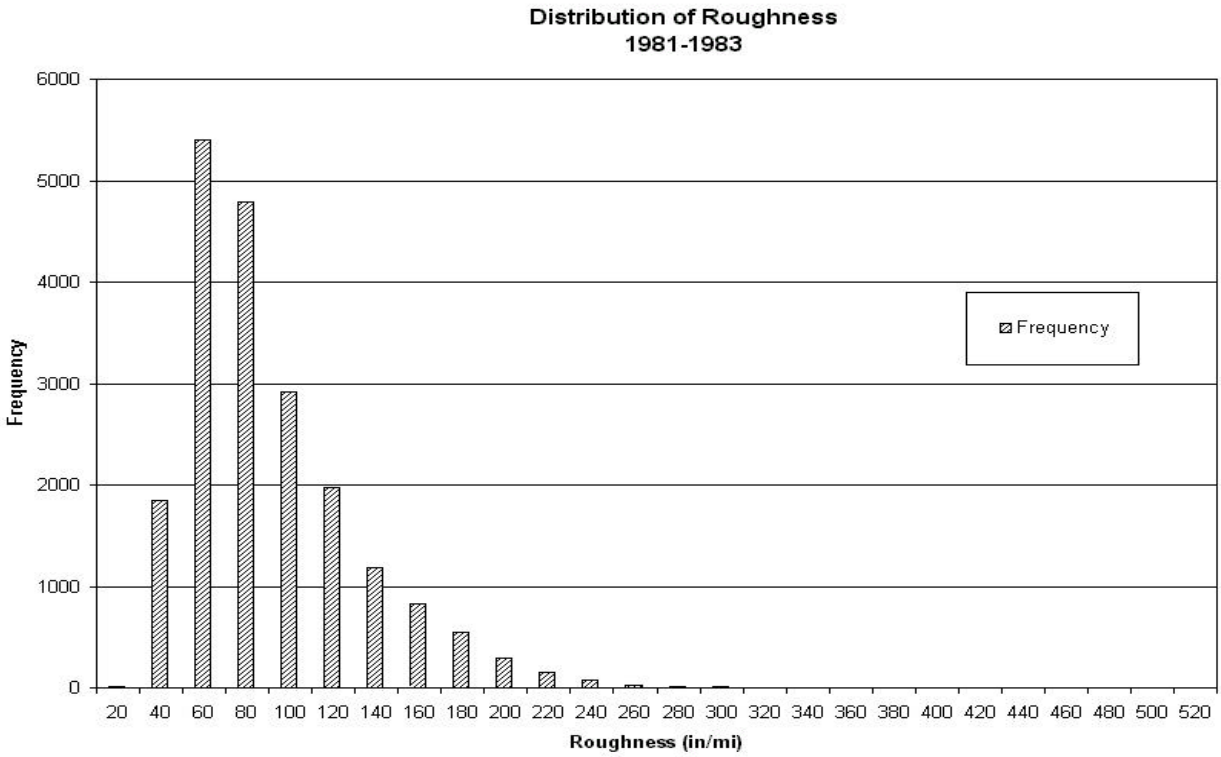

Figure 1.11 Distribution of Roughness during Pre-PMS period

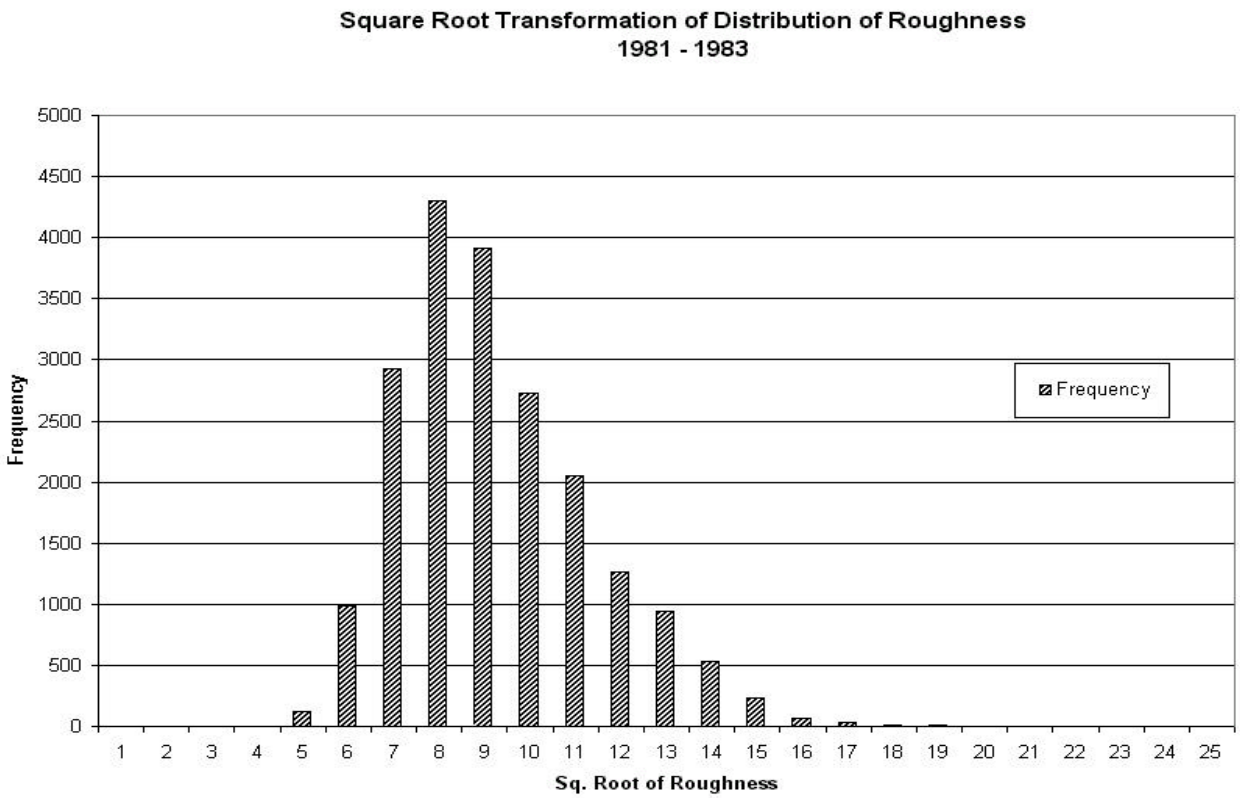

Figure 1.12 Distribution of Square Root of Roughness during Pre-PMS period

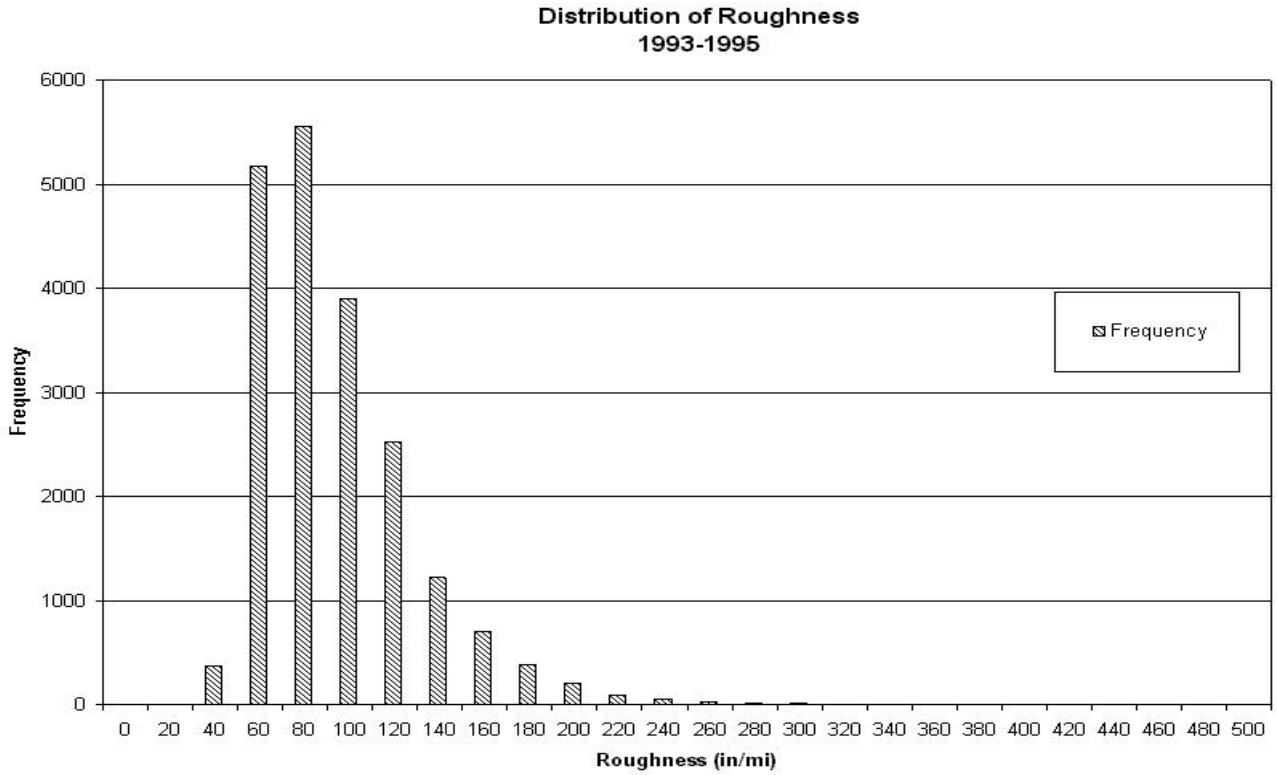

Figure 1.13 Distribution of Roughness during Full-PMS period

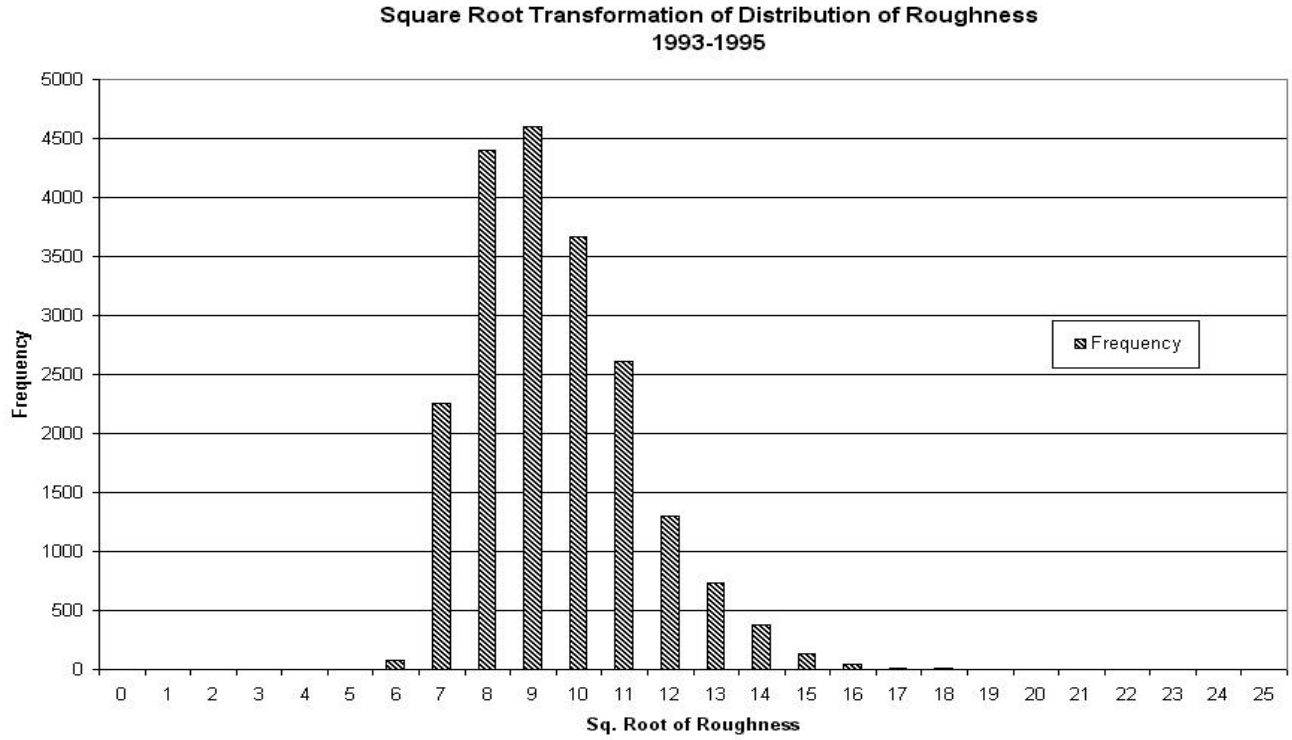

Figure 1.14 Distribution of Square Root of Roughness during Full-PMS period

Figures 1.11 and 1.13 show clearly that the distributions for both time periods are lob-sided, and not close to normal. Consequently it has been investigated whether a transformation of the roughness data could give better results, and it was found that the square root of the roughness values gave indeed a much-improved distribution. The frequencies for the square root transformation have been plotted in Figures 1.12 and 1.14. As can be seen this square root transformation of roughness data does indeed give a distribution pattern that is close to normal. Consequently it was decided to make the final comparison for the two data sets for Pre-PMS and Full-PMS with a relationship of the square root of the roughness versus time.

As indicated earlier the next requirement is to compare the variance (F-ratio) of the roughness values between the two datasets. From the initial Visual Modeler runs it was already known that the F-ratio for the Full-PMS period was more than twice as high as the one for the Pre-PMS period. A summary of the statistical outputs of the square root regressions for the Pre-PMS and Full-PMS periods is given in Table 1.2. The formula's resulting from the regression analyses for the two regressions are also given there. It can be seen that for both regression analyses, the F-Statistics are extremely large, indicating a highly significant relationship between the roughness (or transformed roughness) and age. In addition, the probabilities that either the slopes or intercepts are equal to zero (as indicated by the P-value statistics) are also extremely low. Figure 1.15 shows the actual final regression results, the upper lines representing the 1981/83-performance trend and the lower line representing the 1993/95-performance trend.

The next step is to calculate the confidence limits of the curves for both time periods. The following formula has been used for the 95% confidence bands for the prediction of the network mean roughness values at a given age (two times standard error of predicted mean value):

$$y = a + bx_0 \pm 2\left(\sqrt{S_e^2\left(\frac{1}{n}\right) + S_b^2\left(x_0 - \bar{x}\right)^2}\right)$$

in which:

y = roughness
a = intercept with y-axis (indication of initial roughness)
b = slope (age2 coefficient in Tables 3 and 4)
x_o = age
x = average age
S_e = standard error of estimate
S_b = standard error of slope coefficient
n = number of data points

The confidence limits have been calculated for both regression curves, the results are given in Figure 1.15. It can be seen that in all cases the confidence limits are small enough to avoid any overlap, which confirms that the curves for the two time periods are significantly different from one another. The quantification of PMS benefits is based on the data used in this figure.

Table 1.2 Summary of Regression of Roughness for both periods with Square Root Model

SUMMARY OUTPUT 81-83 SQUARE ROOT MODEL

Regression Statistics			**FORMULA**
Multiple R	0.25631784		Roughness={a + b*(age)}^2
R Square	0.06569883		
Adjusted R Square	0.06565233		**RI = {8.09 + 0.1048*(age)}^2**
Standard Error (Se)	2.04141767		
Observations	20091		

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	5886.983667	5886.98367	1412.632174	7.8876E-299
Residual	20089	83718.61908	4.16738609		
Total	20090	89605.60275			

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept (a)	8.09502437	0.023974881	337.646069	0
age2 (slope,b)	0.1047985	0.002788306	37.5849993	7.8876E-299

SUMMARY OUTPUT 93-95 SQUARE ROOT MODEL

Regression Statistics			**FORMULA**
Multiple R	0.401281		Roughness={a + b*(age)}^2
R Square	0.1610264		
Adjusted R Square	0.1609849		**RI = {7.98 + 0.0993*(age)}^2**
Standard Error (Se)	1.6308568		
Observations	20230		

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	10326.0307	10326.031	3882.413287	0
Residual	20028	53800.28697	2.6596938		
Total	20229	64126.31767			

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept (a)	7.9791174	0.020214694	394.71869	0
age2 (slope,b)	0.0993161	0.001593928	62.309014	0

Figure 1.15 also shows the maximum tolerable roughness level of 93 inches per mile as specified by ADOT. The intercept shows average initial roughness for pavements before PMS (1981/83 period projects) to be 65.6 inches per mile with an average slope or rate of increase of 1.96 inches per mile per year. The after PMS (1993/95) data set shows an intercept or average initial roughness

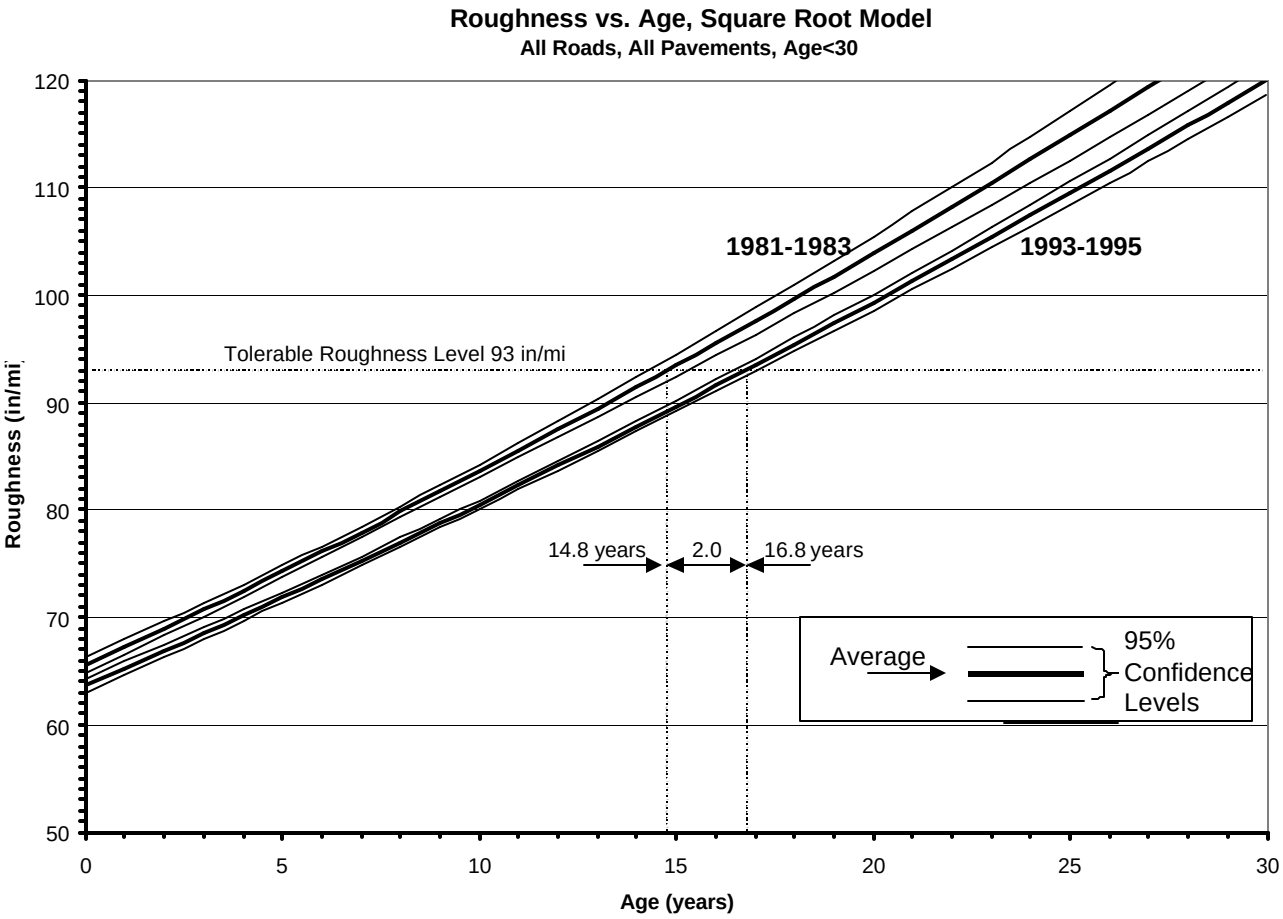

Figure 1.15 Roughness versus Age, Square Root model, Both Periods, with Confidence Limits

of 64 inches per mile and a slope of 1.86 inches per mile per year. This analysis shows that on average pavements were constructed slightly smoother during the 1993/95 period, after introduction and use of PMS, and that they remained smoother longer and performed better than pavements in the 1981/83 time period (before PMS). A lower slope for roughness change shows that the pavement management process has helped produce improved pavement performance for ADOT pavements. The lower intercept or initial roughness shows that on the average, pavements are also being constructed smoother under PMS, as a result of better management and construction practices.

Figure 1.15 shows that Pre-PMS pavements (1981/83) on average reached the maximum tolerable level at 14.8 years of age. For the Post-PMS (1993/95) period the age was 16.8 years. Generally pavements now last 2.0 years or 13.5% longer on the average since the advent of pavement management then they did prior to the development of PMS before reaching the tolerable roughness level. These numbers are useful for calculating quantitative benefits for pavement management.

Figure 1.16 shows the age distribution for all the roads used in the analyses and indicates that during the Pre-PMS period (1981/83) a large percentage of the roads were younger than 13 years in age, while the Post-PMS period (1993/95) pavements were reasonably distributed over a 23-year age range. This also indicates the roads are lasting longer under good pavement management.

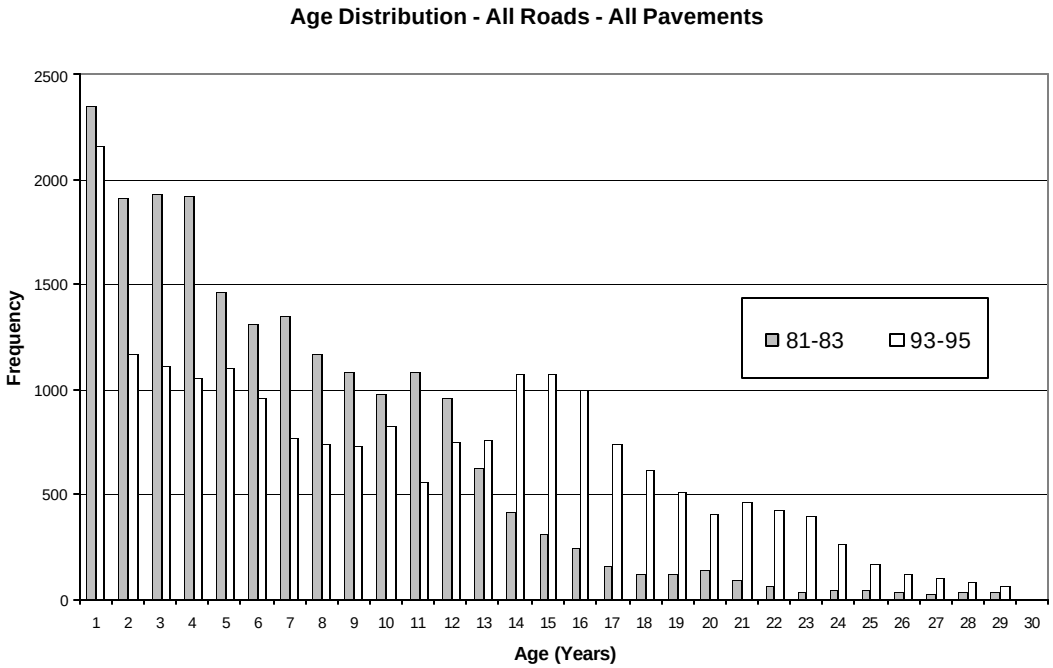

Figure 1.16 Age Distribution - All Roads - All Pavements

1.11.5 Benefits for Interstates Segment

One of the advantages of having a complete PMS database is the possibility to carry out an analysis on certain segments. Using Visual Modeler makes it easy to concentrate on the effects of materials, traffic, climate, etc. It remains important, however, to have a sufficient number of data points in order to arrive at a statistically significant solution.

As an example the benefits of having a PMS has been estimated for the road class "Interstates" in Arizona. In this case the statistical treatment has been simplified, a linear analysis was done, and no full analysis (in monetary terms) is presented.

The results of the regression analyses for Interstates are graphically presented in Figure 1.17. On average the Pre-PMS pavements for Interstates reached a "tolerable" roughness level after 18.9 years; the Full-PMS pavements are expected to reach that same level after 31.6 years. The increase in life for this criterion is 12.7 years, or 67%. While this is most positive and encouraging, it must be recognized that better construction practices, better materials, and better QC/QA may have had some part of the contribution. Table 1.3 defines the details of the statistical analyses.

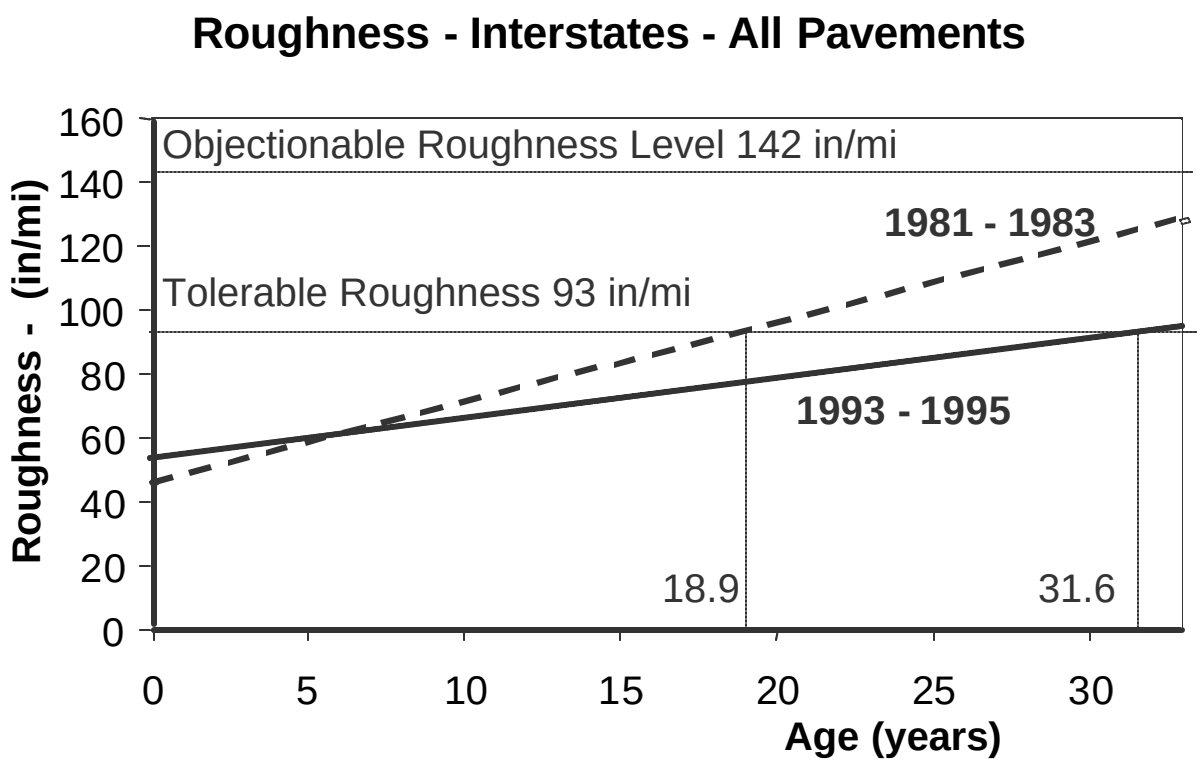

Figure 1.17 Regression Results for Interstate Pavements for both Periods

Table 1.3 Summary of Regressions for Roughness for Interstates only

SUMMARY OF REGRESSION OF ROUGHNESS FOR 81-83 & 93-95, INTERSTATES, ALL PAVEMENTS, AGE < 30 YRS

Regression Statistics	
Multiple R	0.375346
R Square	0.1408847
Adjusted R Square	0.1406948
Standard Error	26.06384
Observations	13580

Formula

81-83 RI = 46.02 + 2.48 *

93-95 RI = 54.49 + 1.22 *

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	1512379.684	504127	742.1006	0
Residual	13576	9222499.077	679.324		
Total	13579	10734878.76			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	46.027987	0.523118753	87.9876	0	45.00260191	47.053372
age2	2.4791882	0.061795191	40.1194	0	2.358061069	2.6003153
year_group	8.4686979	0.7248471	11.6834	2.19E-31	7.047897206	9.8894985
age_by_year	-1.2619787	0.079112541	-15.952	9.09E-57	-1.417050277	-1.1069072

1.11.6 Pavement Budget, Cost Levels and Benefits

Development Costs

In 1980 the ADOT PMS preservation budget was approx. $35 million. This was based largely on the (low) maintenance budgets of previous years. To develop the network optimization, ADOT spent roughly $300,000 on consulting services in 1979. Temporary staff was hired for a total of about 13 man-years to work on the PMS during its development, at a cost of approximately $400,000 or a total of $700,000 for development.
For general comparisons we can assume this cost is spread across the period of 1981 to 1996 or 16 years or about $43,700 per year average for development costs.

Operating Costs

In 1981 the annual labor cost of operating the PMS was roughly $275,000. This has increased to about $370,000 in 1996. To these costs should be added some $30,000 for travel, $60,000 for running/operating equipment, and some $65,000 for capital cost of equipment ($15,000 for FWD, $40,000 for profilometer, and $10,000 miscellaneous) for a total annual operating cost of $430,000 in 1981 rising to some $525,000 in 1996. Total annual cost including development was thus approximately $473,700 in 1981 rising to $568,700 in 1996. The total cost over this 16-year period is approximately $8,340,000,

Pavement Budgets

In 1981 the pavement preservation budget was $43 million and the budget for new and reconstructed pavements was $115 million for a total of $158 million. Budget distribution grew to $ 290 million total in 1996 as shown in Table 1.4. Routine maintenance costs are excluded from these figures.

Table 1.4 Budgets for Preservation & New Construction, 1981-1996 (Millions)

Fiscal Year	Preservation		New Re-Construction		Pavements Total	
	$$	Miles	$$	Miles	$$	Miles
1981	43	400	115	43	158	443
1982	28	278	133	45	161	423
1983	39	386	93	20	132	406
1984	56	544	183	25	239	569
1985	48	280	92	27	140	307
1986	48	290	145	24	193	314
1987	47	275	141	52	188	327
1988	67	296	150	90	217	386
1989	56	269	205	67	261	336
1990	78	371	221	92	299	463
1991	70	250	181	71	251	321
1992	88	308	74	35	162	343
1993	77	275	56	17	133	292
1994	60	209	69	22	129	231
1995	62	251	120	40	182	291
1996	68	222	222	53	290	275
TOTAL	**935**	**4,904**	**2,200**	**723**	**3,135**	**5,727**

There are many ways to quantify benefits or savings. In this case the calculated benefits are 13.5% of the total pavement budgets because pavements generally last 13.5% longer on average. The average total benefits are calculated at $423 million for 16 years. On the conservative side, the minimum benefits with 95% confidence are 8.6% or $270 million over 16 years.

Benefits and Costs

If the extra life benefits are compared to PMS costs we can make a reasonable judgment on benefit/cost ratios. Overall the average benefits totaled $423 million and total PMS costs approximately $8.3 million over 16 years. This gives an overall benefit/cost ratio of approximately 51 to 1. Using the minimum confidence interval values gives a saving of $270 million and a benefit/cost ratio of 33 to 1.

Associated with the improved level of performance obtained there will be an even greater savings in user costs. Thus, the total benefits could substantially higher. According to Paterson and World Bank experience, user benefits will be four to ten times the cost of road expenditures [Paterson 87]. Consequently, the overall benefit/cost ratio ranges from 200 to 1 up to 500 to 1 as a result of more effective expenditure of budgets.

Since it is possible that some of the observed benefits could arise from larger budgets or better maintenance techniques or any other reason we might discount the calculated benefit. Assuming only 50% of the benefits arise from PMS, the total savings are still $212 million and the overall benefit/cost ratio is at least 25 to 1.

In reality, it is highly likely that the increase in life with 2.0 years (at 95% confidence), represents the minimum expected benefits of $270 million or a 33 to 1 benefit cost ratio. With conservative user benefits of 4 to 1, the total benefits over 16 years are $1.1 billion to the citizens of Arizona.

This is clearly proof that pavement management, properly applied, can produce very large benefits to the using agency and to the public.

1.11.7 Alternative Approach to Estimate Benefits

An alternative approach to estimate the benefits of having a PMS is based on the following hypothesis: Implementing and maintaining a PMS will result in greater efficiency in budget expenditures, because increases in damage inflicted by heavier and more traffic loadings can be addressed by a smaller increase in maintenance budgets. The testing of this hypothesis could be carried out as follows:

> Select a base year (1981) and determine for each year from 1981 – 1997 the percentage of budget increase and the percentage of damage increase proportional to increased traffic loadings and or volume.
>
> If the damage increases faster than the budget, ADOT is using relatively less budget to counteract more damage. The ratio of damage to budget per year averaged across years and multiplied by the annual improvement budget reflects the efficiency of the pavement management process, expressed in a dollar amount.

The amount of damage inflicted on a pavement is proportional to increased traffic volume and loads, and the latter is directly related to ESAL's. In Module 9 of the earlier mentioned World Bank study [Patterson 87], some guidance is given about the magnitude of load damaging effects. Patterson concludes that a relative load damage power of four should be used to assess the effects of heavier loads on rutting, roughness and cracking. This is in keeping with the ESAL "fourth power law," which resulted from the AASHO Road Test.

From ADOT's PMS database it can be deduced that the average annual increase in ESAL's is about 4.23% over the period 1981-1995. Conservatively it can be stated that the damage caused by increased ESAL's is linearly proportional to this increase.

ADOT's annual budgets for rehabilitation since 1981, as provided by ADOT, are given in Table 1.4. The data for the total pavement budgets for the period 1981-1995 have been plotted as diamonds in Figure 1.18, together with the trend line for these budget numbers, which shows a slight average annual increase of 1.04%.

The average annual increase in ESAL's has been plotted in the same figure, starting in 1981 at the start of the average budget trend line. Since it has been assumed (conservatively) that the damage caused by increased ESAL's is linearly proportional to this increase, the ESAL line in this figure also provides an estimate of the annual increase in damage due to increased loads.

The efficiency of the pavement management process can be expressed as the ratio between annual damage and budget per year. For the period 1981-1995 that would mean that the triangular surface area between the dotted and the straight line in Figure 1.18 represents this efficiency. This benefit can be determined as:
½ * 14(yrs) * 50(dollars) = $350 million, which is a crude estimate of the savings provided by PMS in serving greater numbers of heavy traffic at approximately constant budgets. It is obvious that this dollar number will be much larger when the conservative assumption of linearity (between increased ESAL's and the damage caused by this increase) is replaced by one that more closely reflects the outcome of both AASHTO test findings and World Bank experience.

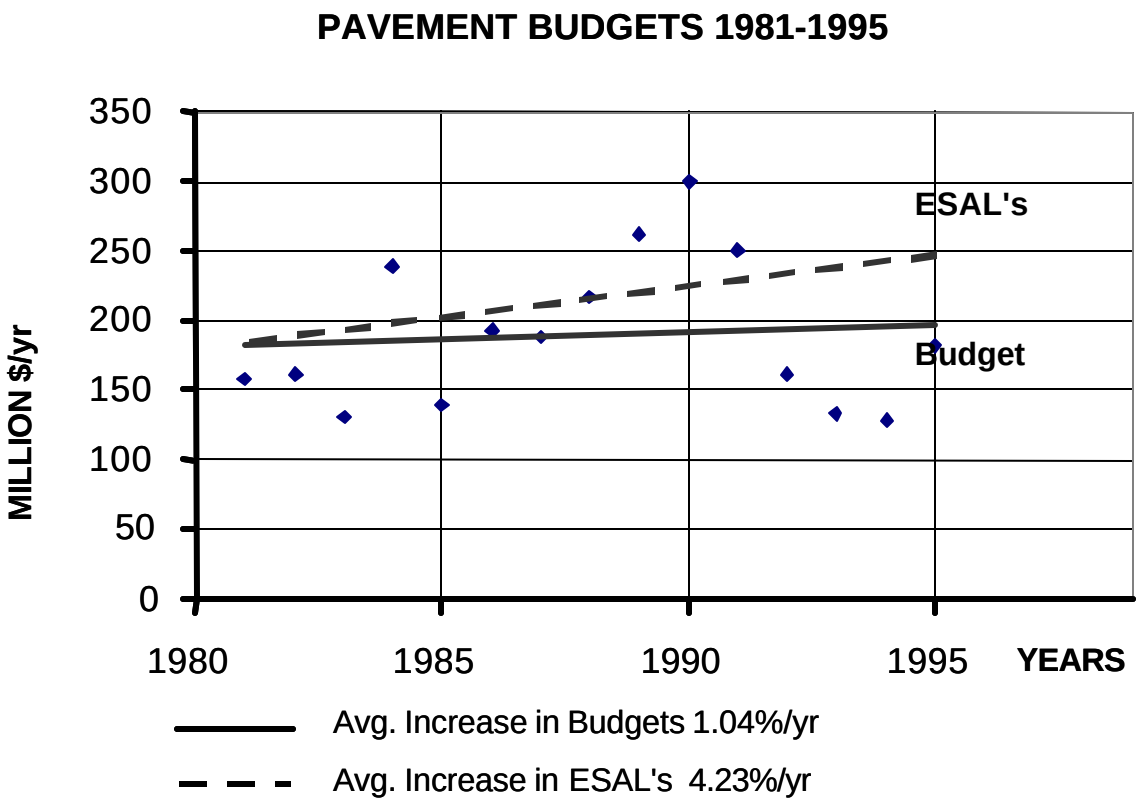

Figure 1.18 Pavement Budgets 1981-1995 and Increase in ESAL's

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2 DATABASE NEEDS AND ELEMENTS

2.1 CLASSES OF DATA REQUIRED

One objective of pavement management is to coordinate all activities required for providing pavement structures in a cost-effective manner. These activities, for virtually all divisions of the highway agency, have impacts of varying degrees on a comprehensive pavement management system. Support of these activities requires a broad database involving every division within the agency. The database should include pavement condition and performance, among the various items.

While the focus of many existing pavement management systems (PMS) is on condition and performance of the surface and the structure, a comprehensive PMS uses data from a variety of sources. The classes of data needed include the following [Haas 91]:

4. Section Description
5. Performance Related Data
6. Historic Related Data
7. Policy Related Data
8. Geometry Related Data
9. Environment Related Data
10. Cost Related Data

Figure 2.1 lists the foregoing classes and their components, and whether they are primarily applicable to maintenance or rehabilitation, or both.

All but the policy and cost related data classes provide background information required for the analysis and modeling of pavement performance. Comprehensive pavement management requires that data from each source be readily accessible to the pavement management staff. In large agencies, such as state transportation departments, each class of data may be the responsibility of a different section. Hence, there is a need for effective coordination and cooperation. In smaller agencies, a staff of one or two engineers and technicians may handle these functions. However, organizing, acquiring, and recording the data in a systematic and accessible manner is necessary in all cases.

Section Description	R + M
Performance Related Data	
11. Roughness	R
12. Surface distress	R + M
13. Deflection	R
14. Friction	R + M
15. Layer material properties	R
Historic related Data	
16. Maintenance history	R + M
17. Construction history	R + M
18. Traffic	R + M
19. Accidents	R + M
Policy Related Data	
20. Budget	R + M
21. Available Alternatives (maintenance & rehabilitation)	R + M
Geometry Related Data	
22. Section dimensions	R
23. Curvature	R
24. Cross slope	R
25. Grade	R
26. Shoulder / curb	R + M
Environment Related Data	
27. Drainage	R + M
28. Climate (temperature, rainfall, freezing)	R
Cost Related Data	
29. Construction costs	R
30. Maintenance costs	R + M
31. Rehabilitation costs	R
32. User costs	R

R: data used primarily for rehabilitation
M: data used primarily for maintenance
R+M data for both uses

Figure 2.1 Major Classes and Component Types of Pavement Data [Haas 91]

The database is a central feature of a PMS as shown in Figure 2.2. The database serves as the repository of the information required to support virtually all engineering analyses and decisions in addition to those concerning maintenance and rehabilitation shown in Figure 2.1. Moreover, the quality of the database will dictate the value of the PMS and the engineering applications that can be accomplished with it.

It has been common practice to maintain these data elements in a disjointed fashion. For example, the construction division may maintain construction records; planning may keep traffic records; and operations may keep maintenance data. Under a PMS, the database function should be centralized so that all the concerned divisions have ready access to the needed data and duplication is reduced.

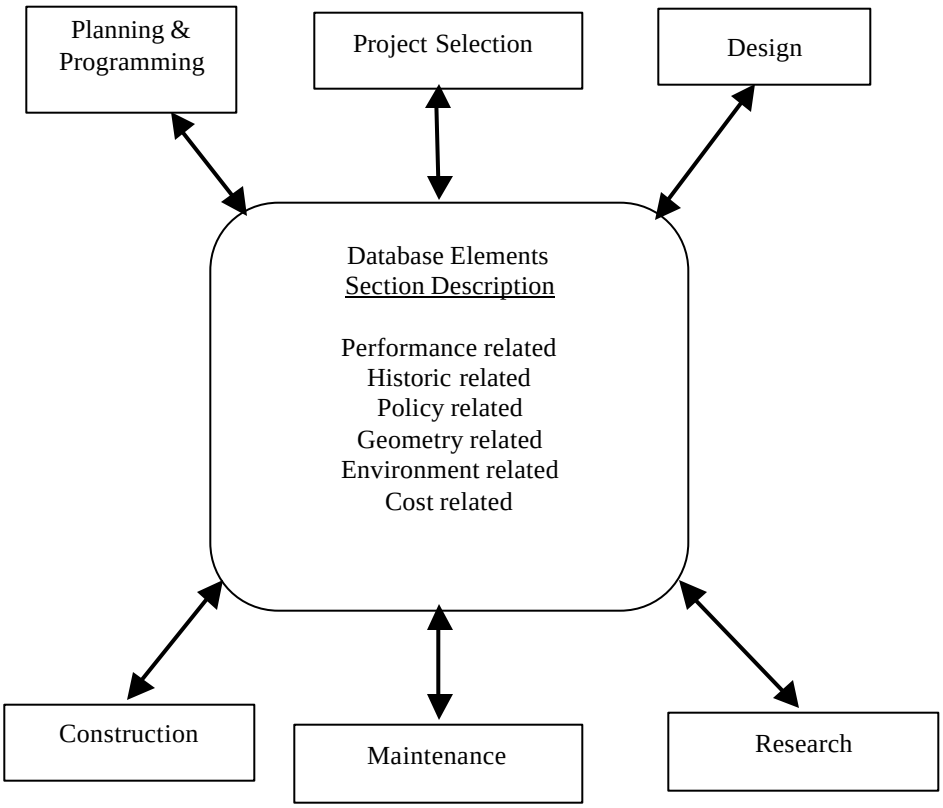

Figure 2.2 Database as a Central Feature of the PMS and Engineering Applications

2.1.1 Uses of Pavement Management Data

Data for pavement management is used at the network and project levels and can be used for engineering applications. The network level data is used in the engineering analysis component of priority programs that are implemented, plus budgeting and financial planning. Project-level data is used for the engineering associated with specific sections or projects such as design and rehabilitation.

Some of the typical network and project level uses for the types of pavement data identified in Figure 2.1 are listed in Table 2.1, which is based on [Haas 94].

Table 2.1 Typical Uses of Pavement Management Data Including Engineering Applications [Haas 94]

Data Item	Network Level	Project Level
1. PERFORMANCE RELATED		
Roughness	a) Describe present status b) Predict future status (deterioration curves of roughness vs. time or loads) c) Basis for priority analysis and programming	a) Quality assurance (as-built quality of new surface) b) Create deterioration curves c) Estimate overlay quantities
Surface Distress	a) Describe present status b) Predict future status (deterioration curves) c) Identify current and future needs d) Maintenance priority programming e) Determine effectiveness of alternative treatments	a) Selection of maintenance treatment b) Identify needed spot improvements c) Determine maintenance quantity estimates d) Determine effectiveness of alternative treatments
Surface Friction	a) Describe present status b) Predict future status c) Priority programming d) Determine effectiveness of alternative treatments	a) Identify spot or section rehabilitation requirements b) Determine effectiveness of alternative treatments
Deflection	a) Describe present status b) Predict future status c) Identify structural inadequacies d) Priority programming of rehabilitation e) Determine seasonal load restrictions	a) Input to overlay design b) Determine as-built (deterioration curves) structural adequacy c) Estimate remaining service life d) Estimate remaining load restrictions
Layer Material Properties	a) Estimate section-to-section variability b) Develop basis for improved design standards	a) Input to overlay design b) Provide as-built records

Data Item	Network Level	Project Level
2. HISTORIC RELATED		
Maintenance History	a) Maintenance Programming b) Evaluate maintenance effectiveness c) Determine cost-effectiveness of alternative designs and treatments	a) Identify problem sections b) Estimate maintenance effectiveness
Construction History QA/QC and Acceptance Testing	a) Evaluate construction effectiveness b) Determine cost-effectiveness of alternative designs and construction practices c) Determine need for improved quality assurance procedures	a) Provide as-built records b) Provide feedback to design c) Quantify as-built material properties
Traffic History	a) Priority programming b) Input to estimate general performance/distress trends	a) Input for pavement design b) Identify traffic handling methods c) Estimate remaining service life
Accident History	a) Develop countermeasures b) Priority programming	a) Identify high-risk sites b) Develop countermeasures
3. POLICY RELATED		
Budget	a) Priority programming b) Selection of management strategies	a) Determine cost limitations
Available Alternatives	a) Selection of management strategies b) Priority programming	a) Economic evaluation b) Life-cycle cost comparisons
4. GEOMETRY RELATED		
Section Dimensions	a) Develop general policy or standards	a) Determine section constraints
Curvature	a) Develop general policy or standards	a) Determine section constraints b) Assess safety

Data Item	Network Level	Project Level
Cross Slope	a) Develop general policy or standards	a) Assess drainage b) Assess safety
Grade	a) Develop general policy or standards	a) Assess drainage b) Assess safety
Shoulders/Curbs	a) Develop general policy or standards	a) Assess safety b) Assess drainage
5. ENVIRONMENT RELATED		
Drainage	a) Evaluate general network performance	a) Evaluate section performance
Climate	a) Evaluate general network performance	a) Evaluate section performance
6. COST RELATED		
New Construction Costs	a) Priority programming b) Selection of network investment strategies	a) Economic evaluation b) Selection of strategy
Maintenance Costs	a) Priority programming b) Selection of network maintenance strategies	a) Evaluation of maintenance effectiveness b) Selection of maintenance sections
Rehabilitation Costs	a) Priority programming b) Selection of network rehabilitation strategies	a) Economic evaluation b) Selection of rehabilitation strategies
User Costs	a) Priority programming b) Selection of management strategies	a) Economic evaluation b) Selection of mitigation strategies

2.2 THE IMPORTANCE OF CONSTRUCTION AND MAINTENANCE HISTORY DATA

In order to fulfill its purpose, a PMS must follow through from planning and programming design to implementation, including construction and periodic maintenance. Construction, of either a new pavement or rehabilitation of an existing pavement, converts a design recommendation into physical reality. Loss of performance, eventually leading to the need for rehabilitation, is identified within the ongoing process of data collection and evaluation. Such evaluation of pavement performance is also used for determining the current status of the pavement network.

Pavement data collected over time provides the basis for developing, updating and assessing pavement performance models used in planning and programming and in design. Essential to such model development is data on the construction and maintenance of the pavement. Pavement construction data includes information on the as-built quality of the materials, such as the results of concrete flexural strength tests and asphalt concrete densities. Large variability of construction quality will result in poor performance, compared to pavements with uniform quality. Pavement maintenance data includes records of all maintenance activities that can affect the performance of the pavement such as crack sealing, patching, and surface seals. A high level of maintenance makes it possible to extend the life of the pavement beyond the expected design life.

2.3 THE IMPORTANCE OF PERFORMANCE RELATED PAVEMENT EVALUATION

Evaluation of all the pavement data items in Figure 2.1 is important. Of key concern is the data related to pavement condition particularly that related to the highway users. Users rate the pavement either consciously or subconsciously every vehicle ride.

The major purpose of performance related pavement evaluation is to determine the current condition of the pavement structure. Four key measures can be used to characterize or define the condition of the pavement (see Figure 2.1):

1. Roughness (as related to serviceability or ride comfort)
2. Surface distress
3. Deflection (as related to structural adequacy)
4. Surface friction (as related to safety)

The distinction between distress, roughness, structural adequacy, and surface friction of the pavement is particularly important. Distress is the physical deterioration of the pavement surface, such as potholes, cracking, and rutting, and it is generally but not necessarily, visible. Roughness is derived from the longitudinal profile of the pavement surface and affects ride comfort or quality. Under the premise that pavements are constructed for the users, then roughness is the primary operating characteristic of pavements that affects users, and roughness; therefore, defines the serviceability or functional response of the pavement. Structural adequacy is the ability of the pavement to carry loads without resulting in undue distress. The safety of the pavement surface is primarily related to the surface friction or skid resistance of the pavement, but it can also be affected by severe rutting or potholes.

These four measures or characterizations, along with maintenance and user costs can be viewed as representing the "outputs" of the pavement, that is, they are the variables that can be measured to determine whether or not the pavement is behaving satisfactorily. These outputs would originally be predicted at the design stage and then periodically evaluated while the pavement is in service. The service life of the pavement would be reached when any one of the measures reaches a minimum (or maximum, depending on the measure) acceptable level. At that point, if sufficient funds are available, rehabilitation would be carried out and a new service cycle would begin.

2.4 CONSISTENCY IN PAVEMENT EVALUATION DATA

When evaluating existing pavements, consistent and repeatable condition data is an essential requirement. Many pavement evaluation schemes rely on the judgment and opinion of a human rater. While this may provide useful insight into condition, such evaluations lack uniformity and generally lose meaning over time as the attitude and ability of the rater changes and/or new personnel are added. Whenever possible, evaluations should be performed objectively using calibrated instruments.

The primary need of an engineering evaluation of pavement condition is consistency of the database across time and space. Frequently, agencies with large pavement networks rely on several teams for evaluating the pavements. Without detailed instructions and training in the evaluation process, there will be inconsistencies among ratings that invalidate the data. The problem is even more acute when the data are examined across time to evaluate pavement performance.

Thus, engineering evaluation of pavements requires a well-documented set of practices and procedures, plus good training. For example, in the measurement and evaluation of pavement distress, the most practical method of survey for many agencies may be with trained observers. In these cases, the observers should be given explicit instructions on survey methods of performing the survey. New equipment, which can objectively measure pavement distress, is becoming more common and widely accepted. Improvements in this automated distress evaluation technology have great potential to stabilize data consistency.

In order to improve PMS data consistency on a nationwide basis, the FHWA recently completed a major research project that developed network-level PMS data collection protocols in the area of cracking, rutting, faulting and roughness measurements for pavements. To achieve uniformity, protocol development considered available technology, practices within critical state DOTs, the extensive LTPP studies [LTPP 93], and adjustments necessary to facilitate data collection by modern and emerging automated data collection equipment technology. The results are being carefully examined by AASHTO (American Association State Highway and Transport Officials) as Proposed AASHTO Provisional Standards (PAPS) for application at the state level.

This project concentrated on developing standards for collecting pavement condition data at the network level. Network PMS constitutes the largest data collection area in pavement management as well as the most costly due to its sheer volume. Planning and programming for annual fund expenditures, optimization and minimization of costs, or maximization of user benefits all require good knowledge of pavement condition. Performance models are essential to good pavement management and can only be built from a comprehensive pavement condition data history over time.

Measurement standards will help ensure consistency of data across time and space, provide reliable means to report and summarize condition measurements, and identify condition measures that are cost effective for network level decision making. Realistic standards of accuracy for condition data will provide a basis for agencies to evaluate vendor data collection procedures and equipment so the accuracy is consistent with the management functions the data is intended to support. Quality assurance guidelines assure that the ongoing collection of condition data meets the required standards of precision and bias. The PAPS will aid the states in creating network-wide needs estimates, predicting pavement performance, and making sound network level decisions.

PAPS for cracking have standard procedures for both automated and manual data collection techniques. This is because automated technology in only now evolving into a viable means for a state to collect network level data and wide acceptance of automated methods will take some time. PAPS for faulting, roughness, and rut depth all are strictly oriented towards automated data collection at the network level. This is because automated technology has been proven and been in use for the past several years. Most states are already collecting or contracting for such automated data collection and would not consider returning to manual data collection. Thus, manual data collection for these three items was considered outdated and PAPS were oriented only for automated collection.

2.5 INVENTORY DATA NEEDS

The management process must have an inventory of the existing facilities. Depending on the requirements of the agency, the level of detail in the inventory will vary. However, all pavement management systems require at least a rudimentary inventory of the pavement network. This inventory is essential for the development of a PMS. The inventory database provides the pavement manager an accounting of the extent of the network, the types of pavements, their geometry, traffic levels, environmental conditions, etc. Essentially, it incorporates information on what exists now, plus past information on such items as maintenance and construction performed.

Developing the inventory of the pavement system can be a time-consuming task. The basic steps in the inventory include the selection of the variables or data elements that will be used in the inventory, definition of the pavement section description or referencing method, and assembling and recording the data into the database. Figure 2.1, which lists all the major classes and component types of data, including inventory data, can provide initial guidance for the selection.

2.6 TYPES OF INVENTORY DATA

The inventory of the network generally refers to the permanent features of the highway or airfield pavements. Because a wide variety of data elements are candidates for inclusion, a compromise between the level of detail desired and practical cost considerations for collecting and managing the data is required. Some large agencies compile extensive and detailed inventories while other agencies operate with minimum data.

There are several major classes of inventory data to be considered in pavement management, and they would, for many agencies, include the following:

33. Section reference and description
34. Geometry
35. Pavement structure
36. Costs
37. Environment (weather) and drainage
38. Traffic

The geometry of the pavement refers to its classification and physical features of the pavements. Pavement structure defines the thickness and material types of the layers. Cost data includes construction, maintenance, rehabilitation and user costs. Environment data identifies those local conditions that can influence pavement performance. Traffic data changes annually but it relates to the other data items and is, therefore, frequently incorporated into inventory data as a matter of convenience, and relevance. However this does not minimize its importance, as traffic volumes and loading are critical factors in understanding pavement performance.

2.6.1 Selection of Pavement Management Sections

Highway agencies frequently have multiple methods of referencing the location of pavement sections. Historically, different sections or divisions in the agency have data collection and use needs that are not totally compatible with the needs of other divisions. For example, the planning section may use traffic control sections for the collection and storage of traffic data; the construction section may use a construction project numbering scheme; and operations may use a route milepost method for scheduling maintenance operations. A PMS needs to coordinate these functions. The PMS must use a permanent referencing system and the location methods used by all other divisions will be changed or cross-referenced. Therefore, one of the first tasks required for the development of a PMS is to establish a common referencing method to identify pavement sections within the network. This can be one of the existing methods in the agency. Dynamic cross-referencing of multiple methods to the standard PMS method may also be accomplished in order to simplify communication across division.

2.6.2 Geometry Inventory

The geometry inventory defines the physical characteristics or features of the pavement sections. It can include, for each pavement section, the following:

39. Location reference and length of section
40. Classification (i.e., freeway or interstate, arterial collector, arterial, local)
41. Number of lanes, widths, shoulder type and width, grade, cross slope, curvature, presence and dimensions of curbs

Geometric characteristics of the pavement, such as number of lanes and widths, shoulder type and widths, grade, cross slope, and curvature can be included in the inventory for basic planning information and to indicate whether the existing geometry meets current standards. These data are generally used in planning major rehabilitation projects to determine if reconstruction is required based on geometric consideration.

2.6.3 Pavement Structure Inventory

The pavement structure inventory is basically a record of the construction history of the project. For some agencies the only pavement structure data available is the type of surface. This is inadequate for a good PMS. At the other extreme are systems that contain complete construction details of the pavement construction history. A database of this type would include the thickness, type of material, source of materials (aggregates, asphalt, cement, etc.), and year of construction for each layer in the pavement structure. In addition, results of construction quality control tests can provide such data on the quality of the materials such as the densities of the compacted layers, compressive strength of concrete, Marshall stability of the asphalt concrete layers, etc. Summary statistics, such as the mean, standard deviation and coefficient of variation, can be used to capture the material quality data.

The pavement structure inventory should also have records of major maintenance or reconstruction projects, including type of treatment and layer thickness.

2.6.4 Cost Data

The cost inventory should include data on the cost of new construction, maintenance, and rehabilitation. It may also include user costs. Construction and rehabilitation costs can be compiled from records, estimates and surveys of recently completed projects. These costs should be updated on a regular basis, at least annually. If the agency has implemented a maintenance management system, average maintenance costs can be determined by analyzing the data records. Otherwise, maintenance costs must be estimated based on the expected performance of the maintenance crews and the condition of the pavements. User costs are estimated based on the traffic volumes, condition of the pavements and models of vehicle operating costs. Obtaining and recording adequate maintenance and rehabilitation costs in the PMS is emphasized for engineering applications because these costs are critical in life cycle cost analyses.

2.6.5 Environmental (Weather) and Drainage Data

Environmental conditions can have a serious effect on the performance of pavements. When these conditions vary significantly across the geography of the agency's jurisdiction, a record of the local environmental conditions can assist the pavement manager in predicting performance and in the selection of pavement rehabilitation strategies. There are several measures that can be used as an index of environmental conditions, such as the Thornthwaite index, freeze-thaw cycles, freezing index, seasonal rainfall, or a regional factor developed by the agency.

Drainage and shoulder characteristics can have a direct impact on pavement performance, and will also affect the selection and cost of maintenance and rehabilitation treatments. While there are many techniques of ensuring adequate or good drainage, the data characterizing drainage is usually in terms either of a porosity or permeability value; or subjectively recorded in terms of good, fair or poor; or alternatively as working or not working.

2.6.6 Traffic Inventory

Traffic data are required in pavement management for the prediction of performance and the assignment of priorities during the selection of rehabilitation projects. For the selection of projects, a measure of traffic volumes is required. For highway agencies the average annual daily traffic (AADT), with a breakdown into percent passenger vehicles and percent trucks, is a common measure of the total traffic on the section.

Performance modeling on the other hand requires an estimate of the heavy vehicle traffic that generates the majority of the distress. For highway pavements, the usual measure is the total 18 kip equivalent single axle loads ($ESAL_{18}$). This can be used to estimate the quantity of vehicle loads relative to the damage caused to the pavements. This is the typical measure of load that the pavement has carried or is expected to carry. Traffic growth rates should be included for both the AADT and the $ESAL_{18}$.

2.7 PMS DATA FUNCTIONS

2.7.1 Historical Database

The function of a database is to provide information for decisions at the various levels of a PMS. Less detailed information is needed for models at the network or program level than for models at the project level. In order to develop the wide variety of models required, including those for deterioration or performance prediction, agencies should develop long term historical databases.

Long-term and continuous monitoring of deterioration (roughness, surface distress, deflection, etc.) is needed to determine the relative damage attributable to traffic and environmental factors and to predict pavement performance. The National Highway Cost Allocation Study, completed in 1982, has examined the data available for determining causal relationships among traffic use, the environment, and maintenance costs. The FHWA Long-Term Monitoring (LTM) Program was established as a forerunner of LTPP in an attempt to assess the problems of building a national database that can develop improved pavement damage relationships [FHWA 82].

The Long-Term Pavement Performance (LTPP) study of the Strategic Highway Research Program (SHRP), now under FHWA, is currently underway and should provide a comprehensive database. This database can be used to build performance models over a long period of time because it should contain traffic and materials data as well as condition information.

2.7.2 Information Flows

A total PMS functions from the details of the project level to the highest administrative level. At each level, the decisions required various types and amounts of data, but the flow of information or sequence of actions has a high degree of similarity.

This similarity of information flow was originally illustrated in a NCHRP project [Hudson 79], which identified three basic subsystems, information, analysis, and implementation (See Figure 2.3). The concept is that in carrying out the various activities and making a decision, pertinent information is gathered and the consequences of the available choices are analyzed in light of this information. Based on this analysis and on other non-quantifiable considerations (perhaps political) and constraints, a decision is made. Once made, the decision is implemented, and the results of the decisions are recorded in the database so they are available to other management levels.

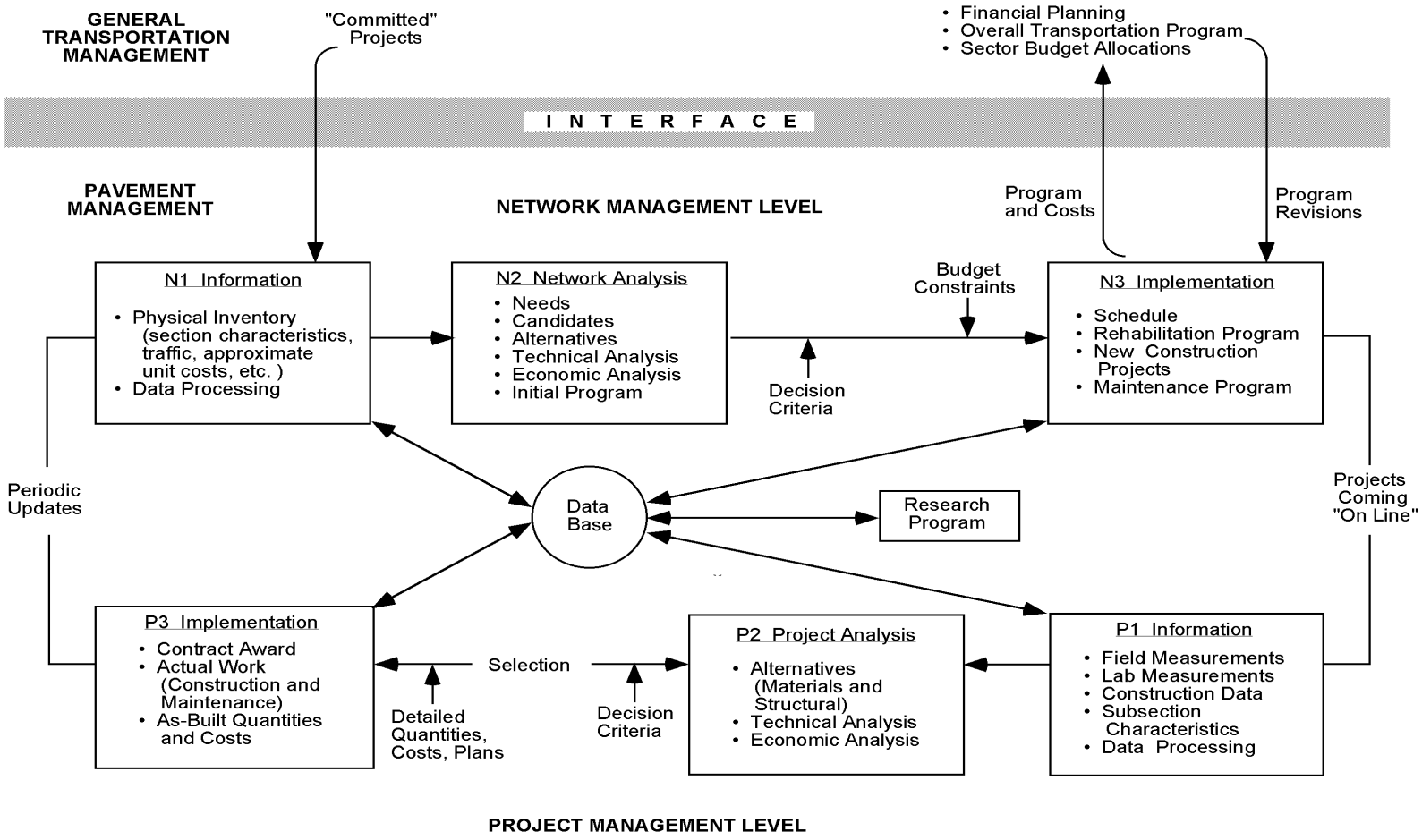

Figure 2.3 Information Flows in a Pavement Management System [Hudson 79]

Although budget constraints, decision criteria and other components of Figure 2.3 are not shown as subsystems, they are, of course, essential to the overall function of the management system. It was considered more useful to separately identify them since they are applied to the outputs of the subsystems. The activities listed within the subsystems of Figure 2.3 are meant to be illustrative and not all-inclusive.

The interface of the network level of Figure 2.3 with the higher level includes "committed" projects that come forward and are submitted in the optimized or prioritized program for review and approval. Any such program, and its associated costs, would likely go forward to the higher level of management as a recommendation, be evaluated with respect to the overall transportation policy and programs as well as the sector (i.e., highway, airport) budget allocation, and then be suitably modified if any program revisions are required.

The subsystems shown in Figure 2.3, and key management activities that are applied to the outputs of these subsystems, are discussed in more detail as follows.

2.7.3 Information Subsystems for the Network Level

This subsystem involves the collection of data that provides the basic foundation for network analysis and can be used for many engineering applications. The essential activities and types of data collected for this subsystem include the following:

1. Determination of what attributes of the pavement should be measured and/or what types of information should be acquired, and the methods, equipment, etc., needed.
2. Identification of homogeneous sections or links in the network.
3. Geometric and other inventory characteristics of the sections.
4. Traffic measurements or estimates, accidents, etc., for each section.
5. Field measurements for structural capacity, ride quality, surface condition, skid resistance, etc., on a sample or mass inventory basis and to a degree of accuracy and/or frequency appropriate to the class of road involved, agency resources, etc.
6. Estimate of approximate unit costs for new construction, rehabilitation construction, and maintenance.
7. Identification of available resources (materials, contractor "capacity," physical plant, etc.).
8. Determination of criteria for maximum roughness and surface distress, and minimum structural adequacy and surface friction.
9. Identification of "committed" improvements or projects from the overall highway, airport, and/or transportation program.
10. Data from as-built projects and maintenance.
11. Data processing for input to network analysis subsystem and for transmittal to the database.

2.7.4 Periodic Updates of Information

Periodic updating of the information subsystem of Figure 2.3 is shown separately even though such updating may be conducted within the subsystem itself. In any case, what is important is the recognition of updating as a key activity and that it applies not only to periodic physical measurements but also to recording changes to the inventory as each project is completed.

2.7.5 Network Analysis Subsystem

The essential function of the network analysis subsystem is to consider the pavement improvement and/or maintenance needs and to arrive at a program of rehabilitation, new construction, and maintenance. This is accomplished through the following activities:

1. Identification of needs and "candidates" for improvement. Future needs identification requires the engineering application of deterioration or performance prediction models.
2. Generation of alternatives for each candidate project or maintenance section.
3. Selection of program analysis period, discount rate, minimum ride quality levels, etc., for engineering and economic analysis; also, identification of what the basis will be for deciding on the final prioritized program (i.e., solely economic or partially economic and partially nonquantitative).
4. Engineering analysis of each alternative in terms of estimating performance.
5. Economic analysis of each alternative in terms of calculating life-cycle costs, and benefits or cost-effectiveness.
6. Development of initial program for new construction, rehabilitation and maintenance.

2.7.6 Project Level Information Subsystem

This subsystem involves the collection and processing of more detailed data, appropriate to the size and type of project, so that the project level engineering analysis and subsequent implementation may proceed. The types of data and component activities may include the following:

1. Identification of homogeneous subsections within the project or section length.
2. Field measurements for or estimates of:
    42. Geometrics (lane widths, layer thicknesses, etc.)
    43. Traffic volumes and equivalent axle loads
    44. Structural adequacy, roughness, surface distress, surface friction, etc.,
3. Laboratory measurements to determine material properties.
4. Acquisition or estimates of unit costs of materials, construction, etc.
5. Identification of criteria or standards for maximum roughness, maximum surface distress, etc.
6. Collection of climatic or environmental data.
7. Collection of available data on construction and maintenance variability.
8. Data processing for use in the project analysis subsystem and for transmittal to data files.

2.7.7 Project Level Analysis Subsystem

The project analysis subsystem of Figure 2.3 primarily involves design where new construction or rehabilitation projects are concerned. However, the terminology and concepts used also allow for maintenance treatment alternatives to be analyzed. A list of activities for this subsystem would include the following:

1. Generation of current and future materials and layer thickness combination alternatives, plus maintenance treatment alternatives.
2. Selection of life-cycle analysis period, discount rate, etc., for technical and economic analysis.
3. Technical analysis of alternatives in terms of predicting distress and performance.
4. Economic analysis of alternatives to determine life-cycle costs and benefits.

2.8 ENGINEERING APPLICATIONS DATABASE

All of the data in an existing pavement management database is, of course, a candidate for use in engineering applications. However, it must be remembered that the data is not necessarily in the format or condition to be used directly. In current terminology, it is necessary to “mine the data” to get additional knowledge from it. In other

cases, if the database for the network-level pavement management system has been carefully setup, then it may be directly analyzed for engineering applications.

Data mining means reviewing the existing data, taking a new look at subsets of the data and determining what information might be available that has previously been overlooked.

One of the important steps is plotting the data. As you plot the data in various categories, if a pattern emerges it should be examined. There might be a strong pattern but lots of scatter. In this case, it is necessary to sit down and list other factors that could affect the results and result in scatter.

As an example, many years ago the Kansas highway department used a simple pavement design process. Prior to the AASHO Road Test the method increased pavement thickness as a direct function of subgrade strength. Weaker subgrades had thicker pavements similar to the CBR design method shown in Figure 2.4 [Yoder 75].

According to discussions with KDOT personnel, this method was used for several years and the general failure curve was as shown in Figure 2.5. Finally it was observed that the design method was functioning differently in various parts of the state as indicated in the plot in Figure 2.6. It was recognized that there were two patterns in the data. The eastern half of the state as illustrated by ■'s, and the western half illustrated by **o**'s in Figure 2.6. It was also known by the Kansas staff that the western half of the state was considerably dryer than the eastern half. As a result, a regional factor was incorporated into the design method and pavement thickness as in the eastern half of the state was increased by two to five inches. This type of analysis is carried out as a combination of statistical analysis and engineering judgment.

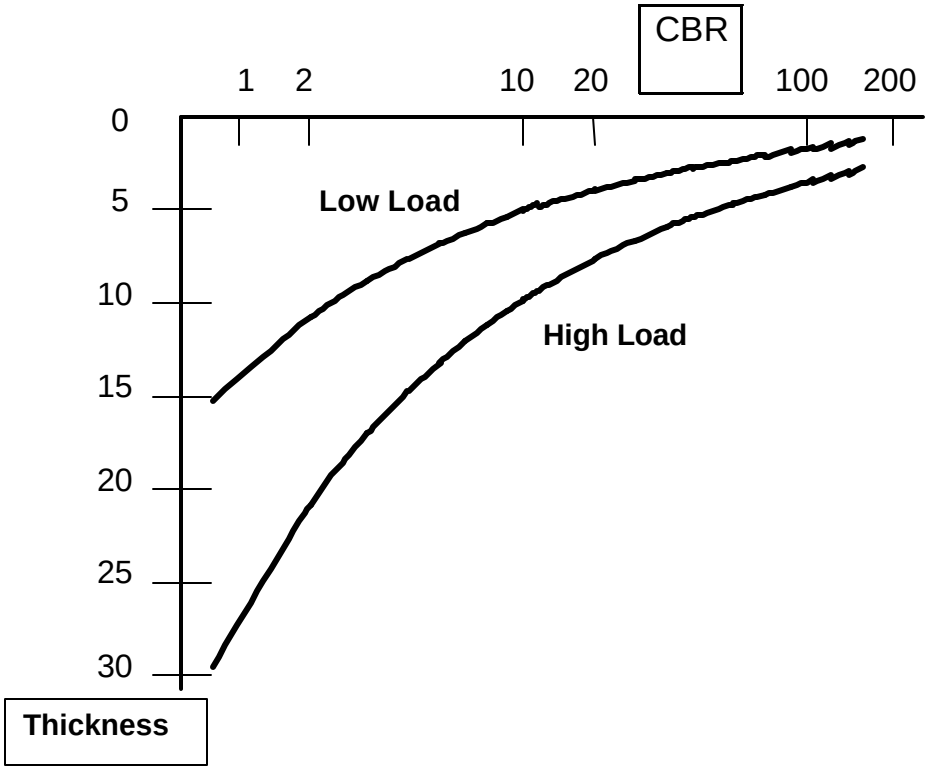

Figure 2.4 CBR Design Concept

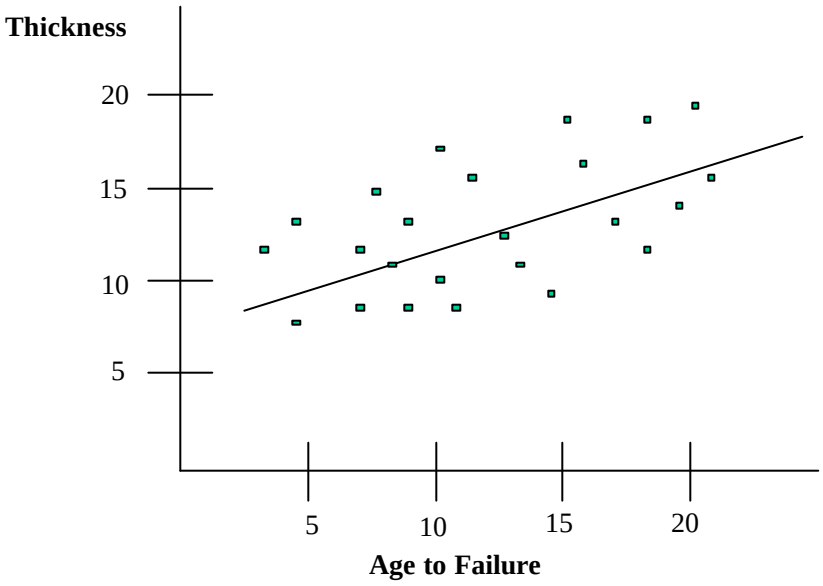

Figure 2.5 Plot of Pavement Failures (circa 1945-1960)

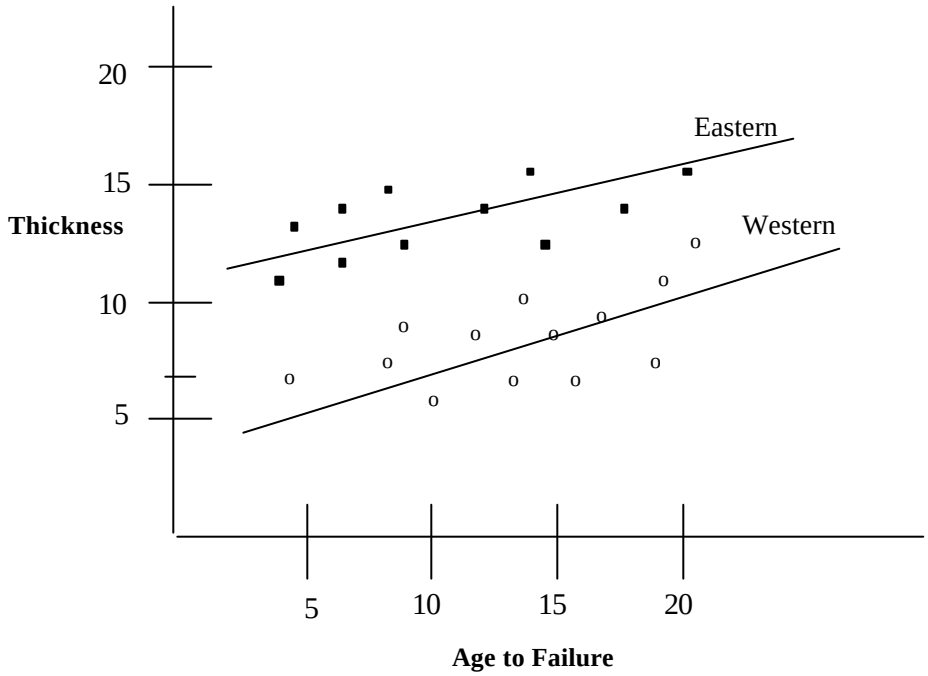

Figure 2.6 Graphic Analysis of Failure Data

2.8.1 Using a Pavement Management Database to Begin Engineering Analysis

The first step in analyzing any problem, particularly engineering problems, is to clearly identify the question to be answered. The typical network level pavement management database may not contain data with enough detail to fully analyze why a particular type of construction is performing poorly but it can help to define the problem and detail its scope. The pavement management database is useful in an engineering study as the first source of information. A well-designed pavement management database will provide easy access to a wealth of information about all aspects of pavements in the agency.

For a pavement management database to serve as the base source for information for an engineering study it should meet the following requirements:

- It should allow the user to access all the information in the database easily and clearly in a variety of formats,
- It should contain sufficient data checking abilities so the user can maintain the database with accurate and complete data,
- It should have an interface that is designed for primary users who are engineers not computer programmers
- Customized queries of information from the database should be easily obtained.
- Control of data updates with appropriate security to ensure data integrity.

These requirements will help to insure that the data in the database is reliable and complete, both of which are necessary to begin an engineering study.

Examples of engineering studies based upon pavement management data are discussed throughout this course. This section presents examples from two pavement management databases that show how the database is used as the source of information for engineering studies.

Montana DOT Condition Database Review

The Montana DOT pavement management database uses a graphical interface that allows the engineers simple point and click access to their pavement management data. An excellent example of the pavement management database serving as the source of information for an engineering study is presented in Module 8. This section shows the interface to the Montana DOT database and an example data quality verification tool used by the department.

The condition data review window is shown in Figure 2.7. This is the first place the engineers will go when beginning to review pavement performance issues or questions. This window shows the performance index data from the Montana database in both graphical and tabular formats. The engineer can “zoom” in on any section in the network to view the condition data on the graph, and print a report of the data for any year in the database. When investigating performance of pavements this will be the first step in any analysis.

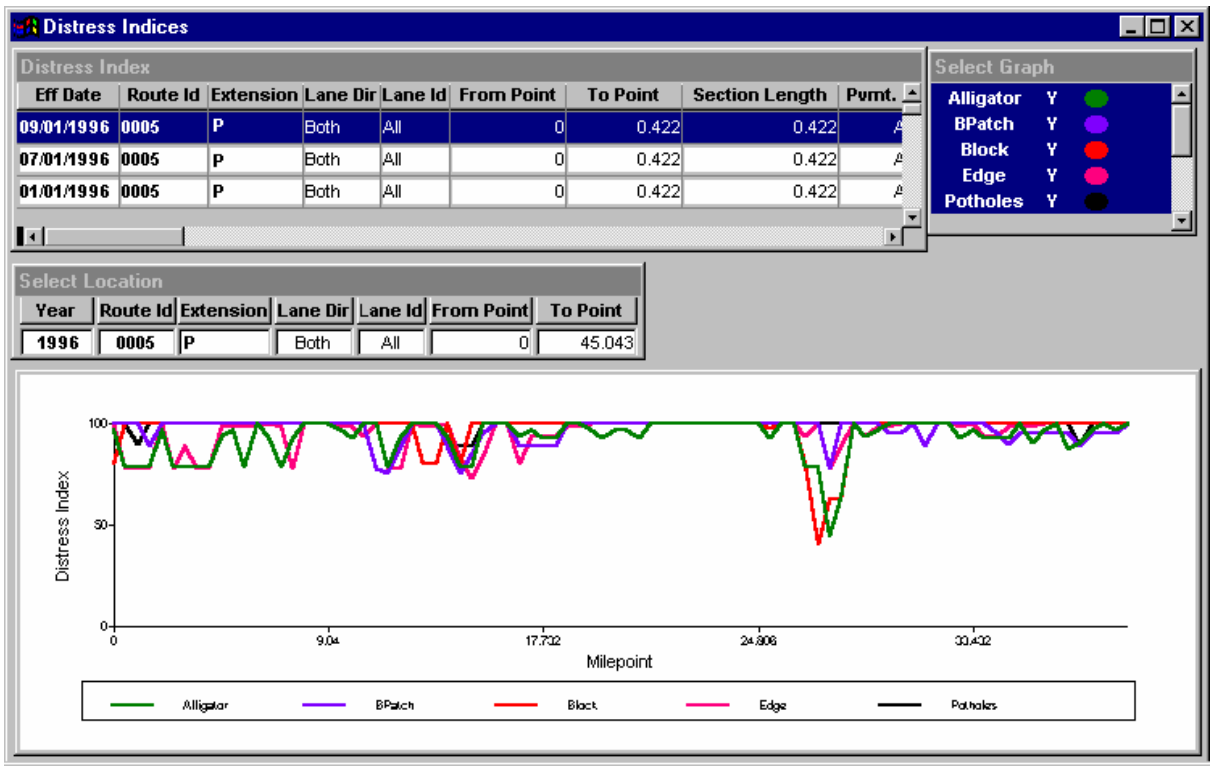

Figure 2.7 Montana Condition Review Window

Montana also uses their database to perform data quality checking to ensure their data is complete. A simple report is produced periodically that helps them to identify missing or incomplete structural data. The report looks at the condition of each section of the network from year to year. If the condition of the section has improved by more than a user defined percentage (typically 10%) and there is no corresponding record of rehabilitation activity in the structural history files, the report will identify that section as having possible missing structural data. An example of this report is shown in Figure 2.8. The engineers can then inquire with local agency employees to determine if pavement rehabilitation was performed and if so of what type so the database can be updated appropriately. If no activity was done the engineers will check the condition data to see if unexpected errors have occurred in the pavement rating. In addition to quality checks made during the pavement condition surveys this also helps to ensure quality condition data. These are both crucial steps in using the pavement management database for any type of analysis. For the database to provide reliable information it must be thoroughly checked on a regular basis for accuracy and completeness of the data.

Pennsylvania Structural History Review

The pavement management database allows the Pennsylvania Department of Transportation easy access and review capabilities for their structural history data. This window is shown in Figure 2.9. On this window the PennDOT engineers can review the data in the database for any section in the network and also view structural cross sections along the road or a project. Each material in the database is shown in different colors, which allows them to identify base, subbase and surfacing layers easily along with their relative thicknesses. Before any engineering analysis begins the pavement structure should be identified. Such a tool allows the PMS user to access that information quickly and easily and produce reports with the exact information needed to begin analyzing a problem.

MONTANA DEPARTMENT OF TRANSPORTATION

PAVEMENT MANAGEMENT UNIT **AUGUST 25, 1997**

Overall Pavement Index Exception Report

Route Id	Suffix	From	To	Year	OPI	% Change
0005	P	56.201	58.567	1996 1997	72.4 83.6	15.4 ---
0005	P	46.330	47.543	1996 1997	65.3 73.2	28.0 ---
0017	S	10.112	11.211	1996 1997	84.0 91	8.3 ---
0017	S	13.432	14.569	1996 1997	44.4 77.9	75.8 ---

Figure 2.8 Improvement Exception Report from Montana Department of Transportation

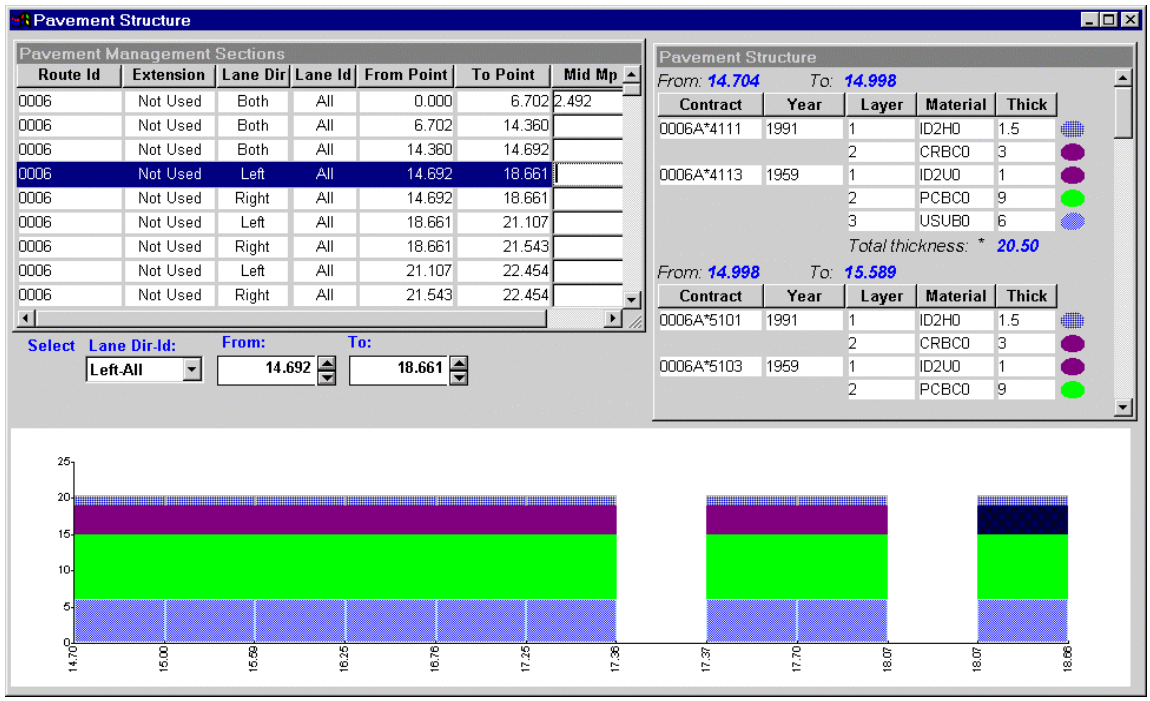

Figure 2.9 PennDOT Structural History Review Window

2.8.2 Data Integration and Centralization

Pavement Management Data is the foundation from which all pavement management information and decision support is derived. The accuracy and completeness of required and desirable data and the transformation of that data into useful information is paramount to the success of the PMS. Because the data is such a critical piece of the pavement management process, it is critical that the PMS database be linked with databases from other functions in order to access and use key information from any pertinent or interrelated component of the agency. Only with such

broad data accessibility can pavement management truly glean all of the information available and necessary to manage all aspects of providing cost-effective pavements.

The integration and centralization of data, together with the accessibility of databases is absolutely essential for a successful analysis of such data. In most cases the PMS database is indeed centrally available in electronic format, but PMS related data such as material properties, construction information and QA/QC data may be managed by groups other than pavement management, stored in other locations, or not available in electronic format. This can lead to serious delays if such data are required for an engineering analysis.

One objective of a well-designed database is to catalog and index all available data throughout the agency and allow easy access by the PMS. It should allow users to analyze that data in any appropriate way to produce information for decision support both at the pavement management technical level and for strategic planning and management at the executive level within an overall asset management approach. Thus, it is important to distinguish the difference between data integration and systems integration. They are different layers in both pavement management and an overall asset management framework. We will therefore focus our discussions on how raw data can and should be integrated at the base level to feed information and decision support at the technical level. The output from the technical level in turn feeds asset management at the strategic level. Within the intermediate technical level, there may also be several levels of detail such as project design level, project selection level, and network level decision-making. System integration extends across these 3 general levels including basic data, decision support, and strategic analysis.

The following issues related to this subject will be discussed next:

- Integrated data;
- Integrated systems;
- Integration methods and tools;
- Analysis of database;
- Statistical analyses.

Integrated Data

Integrated data implies that raw data (with appropriate prescreening and cleaning) can be accessed and utilized by any authorized personnel throughout the agency who is responsible for any aspect of managing the pavement network. It is not necessary to create integration by putting all required data in one comprehensive database. This is because such an approach often leads to problems in managing such a large database. A good approach is to have separate databases for various components, each managed by an owner who is responsible for its upkeep, placed on a centralized server and linked through referential keys for access by the potential users.

Pavement Management is typically classified in a conceptual view as “stove pipe” which implies that it is self contained and not related to other areas within an agency. In fact, pavements extend over many aspects of a managing agency. This is because pavements typically constitute 60% or more of the total value of assets, which the agency was created to manage. Therefore the question of data integration is more fundamental than allowed by the pavement management stove pipe view. Data integration is a two way process between pavement management and all other functions that the agency has to perform to truly manage the pavement and other asset investments entrusted to the agency’s care. Pavement management data must work as integral components within a broader asset management database in order to provide vital information through the systems integration process discussed in the next section.

At the data integration level the definition of the assets to be managed is a core issue. The basis of a definition is some type of basic referencing and naming system to completely identify locations and interrelationships of each of the assets relative to know points on the earth. Since 95% of an agency’s assets are constituted by features along a roadway (including pavements, bridges, signs and other road side appurtenances such as lighting, signals, drainage, right-of-way, etc.), a method of linear referencing, relative to the centerline of the roadway, is appropriate. Such a method would be used as a basis to tie all of the information relative to each of these features together. Within each of the features, significant amount of detail may be collected by each component (“stove pipe”) system such as the PMS, Bridge Management System, Sign Management System, Drainage Management System, etc.

Linear Referencing System

The basis of a linear referencing system is location on the roadway. How that is defined by each agency varies widely throughout the country. NCHRP project 20-27(2) is focusing on the basic definition of linear referencing and provides a good foundation for agencies to build or improve on their existing referencing methodology's [Opiela 97]. In general, the lowest level of referencing should be handled spatially in a two-dimensional x-y coordinate space. The technology for easily handling this type of special information is readily available through GIS (Geographical Information Systems) tools, which are readily available to be integrated and used with PMS and in the overall asset management function.

The basic location of each asset is accurately located on an interactive map, which completely defines the roadway network and all features to be managed, relative to each other and their known locations on earth. The conversion to a specific linear referencing system suitable for referring to locations on a particular agency's roadway network becomes a matter of preference. NCHRP Research Results Digest Number 218 accurately describes several available and suitable methodologies for linear referencing, which can be accommodated and utilized by PMS and other roadway related assets [Opiela 97].

Features Inventory

A features inventory uses the linear referencing as a data key in order to identify and locate all features to be managed. Within such a framework, the pavement merely becomes one of the many features in the asset inventory portfolio. Pavement features may be broken up into shoulders and main lanes and even divided into base and surface layers as individual features. However, the detailed definition of each feature would be specific to the owner/manager/maintainer of each individual asset. The obvious example here is that the PMS contains all pavement condition data over the years and basic functional information about each of the pavement components identified within the features inventory. Similar details are applicable for bridges in which bridge detail data is housed within the bridge management system as well as other component management systems for signs, signals, drainage and other right-of-way features.

The features inventory provides a basic understanding of the magnitude and status of all assets to be managed. With this centralized concept the interface between the PMS and the Linear Referencing System becomes a streamlined set of keys within the detailed pavement management database. The interaction between pavement management and asset management goes through this features inventory identification and keyed cross-referencing method to identify its location relative to the basic spatial standard established under the Linear Referencing System.

Feature Details

The details for each feature identified in the Features Inventory are contained in each component system. For example, the PMS has a great deal of data which is necessary to feed the various decision support and information provision components of the system and to fulfill it's roll in feeding the larger asset management process. Such an integrated data concept allows for the access of detailed information by any party within the department. However, appropriate security should be provided within the information system to allow only authorized access and use of the expensive to collect and delicate to maintain data, which feeds the information engine of the asset management process. At this level, data quality, integrity, accuracy, and completeness become the responsibility of a specific system owner, in this case the PMS group. Communication between and within groups is critical in the overall system design and development process.

Integrated Systems

The primary concept associated with integrated systems is that information can be shared to maximize availability of important information to decision makers responsible for managing the assets of an agency. At the data integration level discussed above, the raw information is related and made suitably accessible throughout the agency for decision support analysis. At the systems integration level the user interface and reporting functions become the important components in sharing the results of the detailed analysis among various organizational units within an agency. Again, as with the raw data at the data integration level, the flow of information is a two-way issue. Pavement management receives reports and information packets from other management areas which affect their

decisions and operations and the PMS provides results and information from its analysis to other management areas and to upper level administrators and decision makers responsible for dividing limited budgets among various asset categories owned by an agency. In addition, system integration involves the interaction between each component system and the core linear referencing and features inventory system. This ensures the integrity of the data that is used to provide the information to the user interface and reporting functions. System integration is a coordinated approach to organizing and interrelating the data and resulting information used and produced by the various components of an overall asset management system. It is important to recognize pavement management as only one piece of the larger picture. It is equally important to fully recognize that pavement management is a vital component of asset management because of its coverage of a majority of the value of the assets managed and because it is the most mature model for the management of asset components.

System integration requires design and analysis of all aspects of system scope. Design of database interactions and relationships relative to the central linear referencing and features inventory as well as the other technical details within a component system and between other systems that manage other asset components is important. Good design based on a sound foundation of linear referencing is a key to success in providing for adequate exchange of data and information within the overall integrated systems concept.

The human component to systems integration cannot be ignored. Ultimately, people feed and operate any management system. Computer software and databases are the focus of this module thus primary discussions have centered on considerations and relative design of databases and systems to facilitate integration and exchange of data and information. However, in approaching the systems integration question the larger concept of the human components and influences upon the system processes must be considered and incorporated into the design of any computer software. The design process must also ensure that fair and reasonable policies and procedures, which facilitate the management process and maximize potential benefits for minimum costs, are in place along with computer tools to facilitate each management role. The management roles themselves are a foundation from which the policies and procedures must be developed. Unfortunately, there is a significant influence of "political factors" which make the human interaction factors highly variable and thus unpredictable aspect to address in this course. In general, it is important to coordinate development efforts among all affected parties and ensure that everyone's needs are met to the maximum extent possible.

Pavement condition information is used for many types of analyses. Accurate linking of location to the condition information over time is of paramount importance. Particularly in the case of experimenting with new materials or techniques routine performance measurements should be conducted on the relevant sections in lanes or at a location where the experimental section is constructed. Often only average roughness level is reported for several different trial sections, which does not allow for performance comparisons between sections. In such cases it is important to have good communications between the performance evaluation group and the group responsible for designing and placing the trial sections. Such a joint commitment is equally important for work programs, and also here an integrated data system can be a great help.

An area where integration of systems is also required is the establishment of maintenance needs and the effects of maintenance activities. It is obvious that major savings are possible if maintenance people have access to performance measurements to plan their efforts, and, if they can obtain information about planned rehabilitation activities. On the other hand, the PMS analyst should know when and where and what type of major maintenance work (e.g. a chip seal) has been done when evaluating and interpreting performance data obtained from recently maintained sections.

Integration Methods and Tools

The sharing of integrated data across an organization has to be carefully planned, designed, developed, and tested. The administrative procedures for accessing such data should be simple and straightforward but with adequate security to protect the valuable data. When the organization is spread over several locations a suitable network has to be established. The linear referencing system discussed above acts to coordinate all information relative to location.

The integrated database should be made available on a server in a central location with strong technical support provided. For use over a wide area network, fast network lines are strongly recommended. Alternatively, architectures such as terminal emu lation services, distributed databases covering only certain regions, replicated data bases, or others can be made into a workable integration solution.

In most state agencies, database and system integration will be controlled by or at least strongly influenced by computer and information services divisions. These divisions typically provide the hardware infrastructure and the technical support structure required for such enterprise-wide systems to succeed. It is important for the owners of the PMS establish a good working relationship with their information services division in order to evolve into an integrated system. Many states by the end of 2000 are still basically operating stand-alone or "stove pipe" pavement management systems. They may have taken first steps at integration with cumbersome download/upload or import/export routines from their systems. However, true integration, as described in this course will take a broad spirit of cooperation and a great deal of human interaction effort to accomplish.

Analysis of Database and Data Mining

In some cases, the PMS database and the PMS-related databases may not be designed for engineering applications although they may have engineering database components. Sometimes an entire set of needed data items may be missing such as rainfall in the Kansas example presented later. In such a situation, it is possible to use a surrogate variable, in this case the geography in Kansas as a substitute for rainfall. If that proves to be useful or rainfall shows to be significant, then it is possible to gather additional data and strengthen the analysis in subsequent years.

At other times there may be gaps in the database that must be filled. That is, a few key points must contain all of the data items needed to make the trends being observed clear. It may then be necessary to go back to the construction records, back to the traffic section, or wherever necessary to fill missing data elements in the database for subsequent analysis.

In still other cases, you may have a good indicator of a relationship, but you need to sharpen up the estimates, or you have to cleanse the data in order to provide useful results. For example, it might be that the current database has traffic information only on the basis of interstate highways (high), primary highways (medium), and secondary highways (low). If a preliminary analysis or plot shows a relationship based on traffic but the relationship is not sharp then it is possible to go back to the traffic section and get more precise traffic estimates that can be used to improve the quality of the analysis. At all times it will be required to evaluate the suitability of a database in order to determine whether it is fit for the purpose at hand.

Data mining is the process of cross-referencing, querying, and interrelating an agency-wide database in an analytical manner in order to extract valuable information. Modern computer tools for querying and reporting from corporate databases are available to greatly facilitate this process. However, such analyses also require a great deal of human thought and effort in experiment design, statistical analysis, results review and comparison, and revaluation. Then, reiteration of the whole process continues trying to look for the always-possible improvement in results.

The pavement management database and the related construction and materials databases generally contain a gold mine of information. That information is not perfect and may not be complete but it can be used. Subsequent examples in the following Modules give good examples of this process from several states.

Statistical Analyses

Performance data in PMS databases normally show a lot of scatter. Consequently, when carrying out an engineering analysis using such data, the results need proper scrutiny to assess whether they are statistically significant. Statistical analyses can help to quantify the significance of a conclusion or a trend. The reader is referred to professional textbooks, such as Statistics in Research [Ostle 74], Design of Experiments for Industrial Engineers [Anderson 82], and Practical Business Statistics [Siegel 90].

This section will briefly summarize the techniques used for deterioration models, and will explain some of the terms and concepts used in statistical analysis of these techniques.

Two techniques are most often used for deterioration models and road performance curves:

- Deterministic techniques, commonly based on regression analysis. Various criteria can be used to get the best possible fit: linear, square root, or other regressions. The least square technique can be used to obtain the curve with the smallest sum of squared vertical prediction errors.

- Probabilistic techniques, involving transition probability matrices. The Markov transition process is a time-dependent stochastic description of event development. Transition probability is defined as the probability that a variable (e.g. an IRI), starting in one condition state (e.g. very good) will be in another condition state (e.g. good) after a specified time. The matrix is formed by condition state versus time.

In order to be able to judge the significance of regression results several techniques are available, but in most cases the results are expressed using one or more of the following parameters:

- Percentile: expressing ranks as % from 0 to 100;
- Mean or average: sum of the scores divided by their number;
- Mode: most frequent score in distribution;
- Median: midpoint in distribution, the point above & below which half of the scores fall;
- Variance: mean of squared deviation scores about mean of distribution;
- ANOVA: Analysis of Variance - statistical hypothesis testing based on the examination of different sources of variability in a complex situation;
- F-test: ratio of two variance measures used to perform each hypothesis test;
- Standard deviation: the square root of the variance;
- Coefficient of variation: Standard deviation divided by the average;
- R-square: coefficient of determination (0: no relation, 100: all points in line).

2.9 REFERENCES TO MODULE 2

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3 SUPERPAVE PERFORMANCE MONITORING USING PMS DATA

3.1 INTRODUCTION

3.1.1 General

Pavement management is a powerful tool for use by state DOTs, cities, counties and other transportation agencies. Pavements make up more than 60% of the assets and asset value of a transportation agency. A good pavement management system [Haas 78, Haas 94] is valuable at both the network and the project level and when properly applied, can be used for various engineering analyses. All 50 state DOTs have a functioning pavement management system although some are more robust than others [TRDF 94]. Approximately one third of the states have very strong pavement management systems. One-third have active network level pavement management. The final third are in various stages of implementing and using a simple PMS. There is also strong emphasis on the use of Pavement Management in the AASHTO Joint Task Force on Pavements.

Since the main use of pavement management in many state DOTs is applied at the network level for planning, programming and budget distributions, the more detailed use of the data for project applications and engineering evaluation is sometimes neglected. The purpose of this study and this report is: to determine the status of pavement management systems and databases, to emphasize the benefits of recording and systematically enhancing the PMS and related data with needed information, and in general upgrading the quality standards for data collection to fulfill burgeoning engineering objectives in state DOTs. It is simply good business to evaluate various pavement structures, materials and new concepts using these pavement management databases. To accomplish this it is necessary to collect, process, store, retrieve, and analyze the required data. In doing this, it is essential to maintain good data quality as well as good data collection standards, within states and across state DOTs. With proper focus this can be done at minimum cost.

3.1.2 Objectives

The main objective of this project carried out by TRDI and some specialist consultants[*]) was to examine how existing pavement management data and materials related data in various state DOTs can be used to evaluate the performance of new materials and concepts and to validate new design methods. In particular, Superpave is used as an ideal example and is carefully examined. This study is not big enough or long enough to evaluate or validate Superpave per se. Rather we examine the process and obtain consensus on what data states would need to collect to adequately evaluate Superpave. Similar studies could be done for any other new design or materials concept such as the 2002 Pavement Design Guide. No evaluation or judgment is made about Superpave, rather the concepts and details of using pavement management and related data are clearly illustrated.

A second objective of the study was to determine what PMS data and other related data and factors the state DOTs collect on a common basis, which could be combined and used for multi-state data analysis. A third objective is to determine what can be done to make data from several state DOTs compatible and usable to others to the point that a common analysis could be made for the broader benefit of all state DOTs and thus the Federal Highway Administration.

[*]) The TRDI Project Team included Dr. W. Ronald Hudson, Project leader (pavement management), Prof. Carl L. Monismith (Superpave), Dr. Charles E. Dougan (States/AASHTO), and W. (Pim) Visser, Project Engineer/Coordinator.

3.1.3 Methodology, Activities and Time Frame

FHWA and the project team visited DOTs in Maryland, Indiana, Florida, Arizona and, at a later stage, Washington State to discuss the aspects of their active pavement management system and the status of their use and record keeping for Superpave materials.
Prior to the visits to these states the team developed a list of questions and a candidate list of appropriate data that were needed for a Superpave evaluation and which hopefully could be collected in a face-to-face visit and through subsequent follow-up mailings. This list covers three areas:

1. General information about the Pavement Management System and the PMS database being used in each state;
2. Details of the Superpave Projects carried out and planned in each state;
3. Extent of availability of detailed Superpave information and performance data in each PMS database or in other databases.

In each case, the state DOT provided representatives from their material's group, as well as their pavement management staff, to meet with the project team. In some cases representatives of the research staff or associated research universities were included in the discussions since they had knowledge of various Superpave activities. Each state DOT was requested, where possible, to provide examples of their pavement management system output, a brief description of their pavement management data collection and data storage efforts, and if possible an example of their PMS database output. They were also requested to provide a copy of any electronic Superpave information available as well as to be prepared to discuss their Superpave data collection and storage efforts.

From the observations made during the discussions with the state DOTs it became clear at an early stage during the project that a routine analysis of the performance of Superpave, or any other material or technique for that matter, can only be done effectively when the relevant data is made available in electronic format. That, in turn, led to the concept to recommend the use of a structure that is summarized in Figure 3.1 and discussed more fully in Section 3.6.2: the use of a PMS database and a Materials & Construction database, both electronic, with links to a Performance Analysis database, from which any desirable performance analysis could be performed.

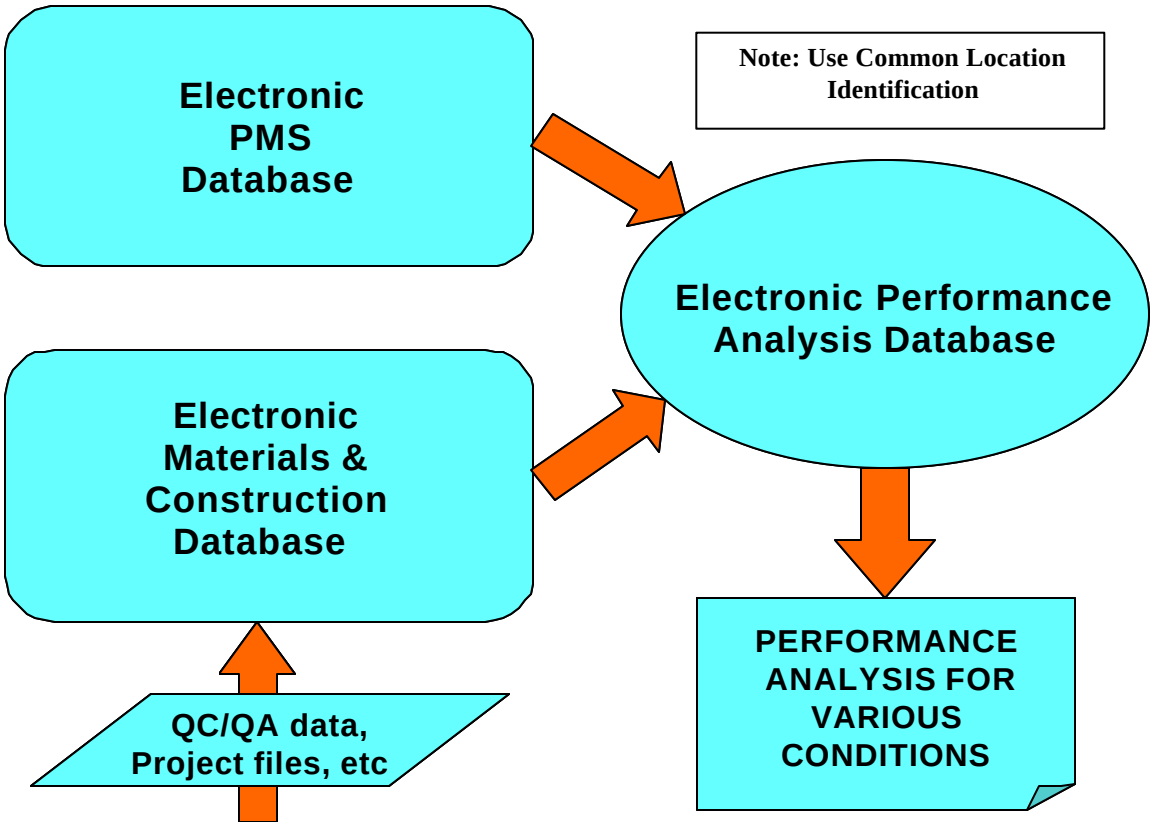

Figure 3.1 Links from PMS Database and Materials & Construction Database for Performance Evaluation

This report summarizes the information gained in these state DOT visits, explores the details that our professional experience shows to be necessary for performance monitoring and evaluation, and gives recommendations for the realization of a systematic methodology for using PMS and related data to monitor the performance of Superpave and other materials and techniques.

3.2 CHARACTERISTICS OF PMS IN FIVE STATES

Generally PMS databases reflect three levels of data collection: network, project and research.
The most general and the most widely used is network level data collection . In most cases state DOTs collect only the minimum required data and in some cases even less than is considered minimally required as discussed below.

1. The project level involves a much more detailed data collection effort and one, which is not currently collected for inclusion in a PMS. It involves detailed design and as-built data on individual pavement sections including thickness, material strengths, variability, stiffness and ultimately forensics analysis of pavement failures.

The third pavement management level involves research and evaluation and in some cases includes forensic information. To be most valuable the research level database will involve several sections or projects where the set

of sections or projects combine to form a factorial or set of sections that can be compared for evaluation of material specifications, construction methods and design. All DOTs in the five states visited have these kinds of data. It is used every time a new method is employed or a new construction procedure is tested. Unfortunately the data seldom get entered electronically into the organized pavement management database.

3.2.1 Network PMS Databases in Actual Practice

All PMS databases contain at least the following information: location of the pavement section (county, district, road nr., mile post, lane, direction), type of pavement, age, traffic (AADT, CESAL) and performance indicators for ride, cracking, rutting and friction, together with the year of testing. More sophisticated databases have additional data on the existing pavement (last rehab, project number, layer materials and thickness), environmental data (regional factor, climate) and more extensive performance data for patching, flushing and maintenance work. The majority of PMS databases, however, give little or no information on types of materials, layer thicknesses and construction details. Available traffic information is often unreliable. Nearly all databases differ in the way detailed data are recorded. Some agencies record AADT per direction, others take the average of two directions. The way AADT is divided over lanes can also differ. The direction can be recorded as a compass direction (N, S etc), or increasing/decreasing with mile posts (I or D). The distress data are normally converted into a performance index or rating, but the way this is done differs by state. The criteria used by states for judging the indicators also differ widely.

An overview of the performance indices used in the five states is given in Table 3.1. It can be seen that there is some diversity in the actual performance indices reported by state DOTs at the present time. The same is true for the definition of the condition limits. It must also be remembered that each state has individual needs within their agency, which will dictate their database characteristics. However, great benefit can accrue nationwide if the minimum desired network level data is recorded by every DOT. States are encouraged to review and modify their data collection efforts in this regard. It is also highly desirable that the data be collected uniformly nationwide. Currently AASHTO is evaluating data collection protocols, which would be very useful for this purpose [FHWA 97].

Table 3.1 Performance Indices in Five States

	Ride	Rutting	Cracking
Maryland	IRI (inch/mile) Condition limits: Very Good, Good, Fair, Mediocre, Poor*)	Max depth for both wheel paths recorded in inch	Not reported
Indiana	IRI (inch/mile) Condition limits: Excellent, Good, Fair, Poor*)	Average depth in wheel paths recorded in inch. >0.25" is Poor	PCR includes all cracking types, Condition limits: Excellent, Good, Fair, Poor*)
Florida	IRI (inch/mile) Converted to RN with 0-10 scale (10 is best possible condition) *)	Average depth in both wheel paths, converted to 0-10 scale *)	Crack rating in & outside wheel path converted to 0-10 scale *)
Arizona	Roughness (inch/mile), Condition limits: Satisfactory, Tolerable, Objectionable*)	Average depth in both wheel paths, Limits: Low, Medium, High*)	% cracking of 1000 ft area at mile point, Limits: Low, Medium, High*)
Washington	IRI (in/mile or cm/km), Condition limits: Very Good, Good, Poor, Very Poor*)	PRC, average depth in both wheel paths, Limits: Very Good, Good, Poor, Very Poor*)	PSC includes all cracking types and patching, Limits: Very Good, Good, Poor, Very Poor*)

*) See also Section 6 for additional information on distress ratings in the five states.

3.2.2 Actual Project Level Data Collection in Example States

The way data are collected, stored and analyzed varies from state to state, but in most cases the following four organizational groups are involved:

1. The Pavement Management group is responsible for the PMS database and its maintenance, for collecting performance data and for analyzing and interpreting the data in this database. This group is also largely responsible for the allocation of funds, since they provide data for projects that use various classifications of money (rehabilitation, maintenance, etc). Data on traffic loads and distribution are normally obtained from a separate Traffic group in the DOT, but they are often not available in a timely fashion. Data on distress, ride and friction are entered into the PMS database, data on deflection measurements are normally kept in a separate database for the benefit of the Pavement Design group. Data on maintenance activities are normally not recorded in a PMS. The date of carrying out performance measurements is recorded by year only, consequently it is unknown whether maintenance or rehabilitation work was done before or after the measurements.

2. The Pavement Design group is responsible for the design of new road pavements and for overlays and rehabilitation design. This group makes use of data supplied by PMS (performance, location, climate, traffic, age, etc), data from deflection or other structural testing (for existing pavement and/or sub grade), and data on materials to be used from the Materials and Testing group. With this last group there is often a lively exchange of information to arrive at the best possible choice of materials for a particular design.
   In some DOTs the design of new Portland cement concrete pavements is done by a different group, responsible for all concrete structures. The rehabilitation of these roads is normally the responsibility of the Pavement Design Group.

3. The Materials and Testing group is responsible for the selection, design and testing of all pavement materials. Most of the data used by this group are generated from laboratory and field testing, these tests are normally carried out prior to construction (on raw materials and for mix design), during construction (check on mixes from plant and paver) and immediately after construction (mostly on cores). This group works closely together with the Construction group prior to and during the execution of the project.

4. The Construction group is responsible for the realization of the project. This group uses data from both the Pavement Design and the Materials and Testing group. An important part of the data is related to Quality Assurance (QA) and Quality Control (QC) requirements. QC testing is frequently contracted out to the contractor or an independent laboratory. In most cases the QA testing is done in-house by the Materials and Testing group. On-site density tests are carried out extensively, and in most cases cores are taken to check volumetric properties. These cores are hardly ever used to check actual layer thickness, consequently this important parameter is often neglected. A modern trend is to measure and record thermal images of the pavement prior to compaction to check cold spots and segregation. The date that the pavement is opened for traffic is often not recorded.

For the evaluation or monitoring of the performance of materials like Superpave it is imperative to have access to data generated by the four groups mentioned above. There are three main reasons why such data are often difficult to obtain:

a) Each of the groups described above has a clear, but different responsibility, and this is reflected in the way they generate and store their data.
b) The time lapse between initial planning for a project and the actual construction can be several years, and this puts an additional burden on keeping all relevant data organized and accessible.
c) In many cases major changes are incorporated, particularly for a longer time frame. These changes in design, materials or construction techniques are not always recorded in the documentation of all four groups, and this makes it more difficult to get accurate data after the project is completed.

There have been attempts by DOT's to develop a databank with project information, based on e.g. a survey questionnaire, but in practice it often appeared difficult to collect the data after the project was completed. There are a few exceptions, ADOT for instance has a reasonably updated project database, but even here not all relevant data needed for a proper evaluation is available.

3.2.3 Actual Research Level Databases

Currently most state DOTs do not record research data in their PMS or a PMS related database because of lack of resources. It is not feasible to assume that project or research engineers who work full time in the design, construction and observation of projects or who have already completed those projects and have data available in hard copy, will have time to enter the data in a database. It is necessary to set up a data collection and processing activity, which will make the data available to the electronic database. The activity must include the pavement management section to ensure that the data are compatible in every way.

An important example of data that fit this category but which have never been properly used or entered into most state databases are the LTPP data. In many cases the state DOTs could take the individual LTPP sections and supplement them with measurements in their own state on other sections to make a broader applicable factorial, which could be analyzed for state benefits.

Little effort has been made in most state DOTs to store research level data in a form keyed to the pavement management database. There are many reasons for that, which is not relevant at this point. What is relevant is that state DOTs should be made aware of the benefits of keying such data to the PMS database. The value of research could be doubled if the data could be maintained permanently so that mistakes are not repeated.

Another major benefit of keying these data into the PMS database is that multiple states could share the data with each other. Electronic storage is critical to such databases. For example, if ten states stored their research data for 30 test sections each into their own PMS database, they could exchange data electronically among the states and a much broader database of 300 section could be analyzed. Most engineers do not fully understand the powerful analytical capabilities of large sample statistics. Even in the face of large variability, strong trends can be obtained when data sets can be combined into one large one.

It is important for data entered into the database, to make reasonable estimates of the traffic load history of each section. Even if weight data are not available, it is vital to make a timely estimate of equivalent single axle loads

based on a reasonable evaluation of available load, traffic, and classification data. If attempts are made to reconstruct the data five to 15 years later, then the value of the estimates are degraded.

In summary the characteristics of the PMS database are extremely important. Great benefits can accrue to state DOTs if those data sets can be broadened to include project and research level data on a regular basis, and if the data can be combined among states for additional strength. Subsequent sections of this report will deal with Superpave data as a specific example and the importance of gathering these data while they are fresh.

3.3 DOT REPORTING TECHNIQUES ON MATERIALS AND CONTRUCTION

Current DOT practice is normally to plan, design, bid, and build projects. Included in these functions are the materials requirements and the QC/QA project acceptance requirements. Each function is treated as a separate entity, i.e., the design plans and specifications govern the project in a general fashion. Intermediate laboratory tests to develop project mix parameters are the "property" of the materials group, tied to the assigned project number. In a like manner, the day-to-day materials tests and QC/QA testing performed by construction inspectors are delegated to the project file, in the materials area and project office, or sometimes by the contractor. Usually duplicate materials testing data are provided to the project inspectors for payment and audit purposes. In most cases, data on materials, mix design, and construction details are recorded on paper, so this information is stored in flat files by different groups and is often difficult to access after the project is completed.

The pavement design plan for a particular rehabilitation project can normally be found in an inter-department communication, or memorandum, from the pavement design engineer to the Manager of Preliminary Engineering Studies Section, the project engineer or coordinator, or the design engineer. Such a memorandum is routinely copied to the materials engineer. In some cases, further correspondence follows between these parties with various revisions until a final design recommendation is made. In most cases this will be accompanied by one or more drawings with the cross-sections of the pavements in question. Apart from the design for the main pavement section, design details can be given for shoulders, ramps, overpasses, and temporary crossovers. This correspondence normally refers to the name of the road and the mile posts of the section, to a design number, and to a contract number. Often a project number is used also. Copies of these memoranda are filed at various different locations within a DOT. The time lapse between the first design memorandum and the final version can be from one to several years, since there can be long delays for a project between preliminary design and final construction.

Data on the laboratory mix design are developed by the materials group, they are tied to a design number and a project number. Most of the historic laboratory data are stored in flat files, and consequently are difficult to access. Currently there is a trend to make such data available in electronic format, either with the help of a commercial system such as LIM (Laboratory Information Management), or through custom made spreadsheets on a PC. In that way it is easier to analyze and to share the data, but the fact that such data are often still residing on a PC or a dedicated server makes it difficult for other interested parties in the DOT to access such data.

The routine testing data and the quality control and assurance data can come from several sources. There is a trend of increased testing by contractors or specialized consultants, although quality assurance is mostly still carried out by DOTs. Such data were usually made available in paper files until recently, when attempts were made to streamline data storage and transfer by using electronic systems. AASHTO introduced two programs for this purpose, Site Manager and BAMS (Bid Analysis and Monitoring System), but both have the limitation that they can not be integrated easily with other databases, such as PMS. Site Manager also has the drawback that it does not have any sort of filter or query capabilities.

Knowledge of and easy access to materials and construction data is essential if the long term performance of pavements is to be analyzed for constructive purposes. Such data should be linked by the uniform identifiers of date, project number, route and cumulative mileage to any follow-up performance data obtained after construction is completed. Routine measurements as well as specialized tests such as FWD should be tied together also by route and milepost to create a more complete picture of historical performance and expected performance trends by

various classes of pavement. In this respect it should be stressed that records should be kept of all required data. Some DOTs do not measure as-laid thickness routinely, but this information is essential for analysis.

Arizona DOT has for several years maintained a separate electronic database for projects, which is called Projmod. This file contains the following information: Route, lane, direction, mileposts (begin and end), material category applied and its thickness (as designed, for up to six layers), project name, and date. The file does not give actual thickness and material properties (designed or actual), but it is a good start for any engineering analysis.

In the ideal situation all relevant materials, construction and performance data would be available in electronic format and a full integration and sharing of such data could be realized. There would be major advantages if all required data are indeed entered electronically:

- Data entry would only have to be done once,
- Less room for mistakes,
- Available immediately,
- Corrective actions are possible in a timely manner,
- Automation possible (link to GPS during construction, field testing, etc)

The web-based system developed by the University of Washington in cooperation with Washington State DOT offers an excellent methodology for both data warehousing, inspection, linking and analysis. Details of this system and examples of its capabilities are given in Appendix F.

3.4 CONCEPTS FOR LINKING PMS AND MATERIALS AND CONTSTRUCTION DATABASES TO PERFORMANCE EVALUATION

3.4.1 Current Situation

Typically, when engineers and analysts think of performing an evaluation or analysis, they think of the final data set needed to make the analysis, in other words they think of the "combined performance evaluation database". While that is the ultimate goal, it is not a feasible starting point, since such a database must be constructed from other available data sources first.

PMS Database

It's important to remember that the average PMS database is very large in terms of number of sections or subsections entered. In Arizona DOT, for example, there are more than 7,000 sections and in Texas there are more than 70,000 sections. On the other hand, for a particular performance evaluation module, the number of sections to be analyzed may range from a few to several hundreds at the most. Keeping this size differential in perspective should make it clear that it is impossible to put all the detailed materials data into the very large PMS database. Rather, separate databases for each individual purpose can be maintained or assembled and those linked to create the performance evaluation database or module. These facts are born out by the knowledge that early in the development of pavement management, it was thought that all project details could be assembled into a database. In each case the process bogged down. Only when states like Washington and Arizona [Kulkarni 82] started with the idea of keeping simple performance data at the network level, did pavement management move forward rapidly.

For the purposes of material evaluation and this project, reality starts where the states currently exist. Good PMS databases for network level use are simple and contain relatively few data elements because excessive detail can bog down the process. The data elements used in five states that were visited in this project are given in Section 6.

Most pavement management databases do not include details about actual pavement thickness and material type below the surface layers. Time and resources have not permitted coring and searching permanent record files to supply this missing data.

Similarly, reliable traffic load data are often not available in the PMS database. For the present, it is recommended that the traffic data be obtained and entered into an analysis file. However, in some state DOTs a traffic data file is available and this information can be provided into the PMS database. In most cases, it is desirable that the traffic and load data element be added to the PMS database. Originally this element would contain data only for those sections being considered for evaluation of Superpave, other new design, or materials concept. As these data are collected and processed, it may become clearer to the DOT staff that it is important to add this information to the entire PMS process. If this is not possible, then a supplementary data element called the "Detailed Data" should be developed.

Depending on the materials or conditions being evaluated, other classes of data not available in the PMS database may be needed. This may involve detailed climatic or history, structural evaluation such as repeated FWD measurements, and in the cases of some materials more detailed distress data related to the purpose of the material. For example, it may be necessary to evaluate rutting more extensively for certain types of mix designs than for others. Therefore, rutting or detailed cracking while combined as an index in the PMS database, should remain separate for those sections being evaluated for Superpave.

It will also be valuable if auxiliary short-term performance information such as, wheel track testing, Stabilometer values or other heavy vehicle simulator or other short-term estimators of performance could be stored in a permanent database.

Materials and Construction Databases

As discussed earlier, much of the required data necessary for the performance monitoring of individual design and construction concepts exist in the materials and construction databases, not in the PMS database. Two major problems exist. The sheer magnitude of the design and construction information exceeds anything that can be entered into a routine PMS database. Thus, the data, if stored, must be summarized on a statistical basis keyed to individual locations, including average value, standard deviation and number of samples. Probably more important is the fact that the data exist in a separate database obtained for a different purpose. First of all, the design data are probably not in a database, but on a set of hard copy files or hand written notes that define the design parameters and the mixed design characteristics obtained for the materials and the pavement structure for the individual section. Secondly, as-constructed pavement data exist in a separate construction file. In some cases, and hopefully in the future, this will exist in a file such as Site Manager or in another suitable electronic data subsystem. These will make the process somewhat easier, but it must be remembered that data collected for one purpose, often have limitations for a second purpose. The transposition of these data must be done carefully, effectively, and in a timely manner.

Another major problem with linking data is the source of the data, which may be some other section of the DOT, the files of a contractor, a materials testing company working for the contractor, or a field materials laboratory. Other complications occur when field changes are initiated due to unexpected conditions such as weather, varied subsoil conditions, or existing condition of old pavements where rehabilitation or reconstruction is involved. This latter condition is further compromised and complicated when the section is to be built under existing traffic. Other complicating factors occur when the section is a widening of an existing pavement where drainage patterns are interrupted.

In these subsidiary data sets, the data may be keyed to local field identification and location, which must be transferred to a permanent mile point GPS location system. This must be done carefully with cross checking if it is to be useful. Data improperly located in an electronic data format are useless and have a negative impact since purportedly it represents conditions incorrectly. It is better to have missing data then bad data.

Only with special effort made at an early date will the linking of the data take place. There are many examples where important past research is not accessible for subsequent analysis and in fact the data have been effectively lost.

Summary of Main Needs

It is important in summary that the data from a materials and/or construction database be appropriately and effectively linked to the PMS database at an early date within the efforts of the history of the data. No major effort should be made to retrieve all possible project level or research level data for a twenty-year period. However, in many state DOTs it would be desirable to initiate a retrospective effort to enter such data for the past year and to setup the database so that the data could be effectively entered in the future. In this way, important data would not be lost. This is especially important for the Superpave study which serves as a primary example in this report. Superpave sections have been constructed for the past several years and because of the critical focus on this element nationwide, such data should still be available in most state DOTs and as soon as possible should be entered into an electronic database keyed to the PMS database.

3.4.2 Concept for Linking Databases

It is proposed to use the concept of linking two central electronic databases, one for Materials/Construction data and one for PMS data. Relevant data from these two databases should be made available and transferred electronically to a third Performance Analysis database, which should contain all required information. A possible structure for this methodology is given in Figure 3.2 below.

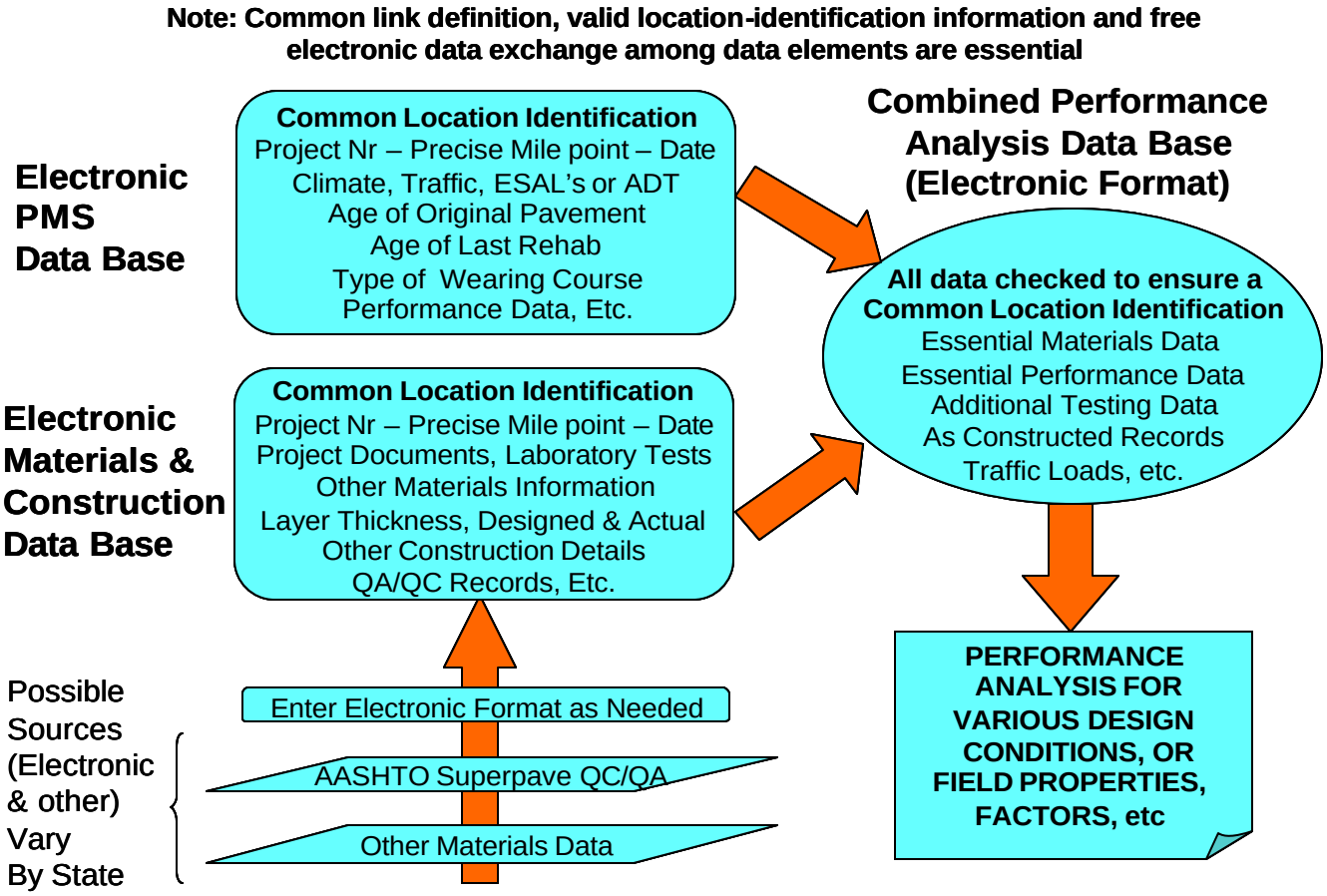

Figure 3.2 Example of a Structure to Link Materials and Construction Data to PMS Data

The key to linking databases, as shown in Figure 3.2, is to have precise unambiguous location identification and date/time information. Only in this way can it be assured that the data are comparable. For example, in the case of multi-lane roadways involving bi-directional interstate highway lanes and frontage roads, the new material may be used in only one or two lanes or in one roadway direction. This identification must be unambiguous over time. When the material is first placed, it is clear in everyone's memory, which is which, but four or five years later or after personal changes, when a long-term analysis is needed, this becomes more difficult. Unambiguous locations can be provided by GPS measurements and these are relatively economical at the present time. However, they must be tied to traditional location identification information such as project number, mile point, lane, direction, date, etc.

3.4.3 Practical Implementation of Data Linking

While the concept of data linking is relatively obvious to all, it became clear in this project, by visiting five DOTs, that additional resources and energy will be required to actually capture the information in usable form. As long as the data elements remain in paper files, field logbooks or other auxiliary locations, there will be a very difficult transition process.

The good news is that several states have already started to put materials and construction data in electronic format. An effort should be made in other states to transfer the data from hard copy files into electronic form quickly, soon after construction. This would introduce an additional check into the system and there is a good chance that the quality of data can be improved. With a properly designed system much of the detailed design, materials and construction data, only have to be entered into the system once. But this initial entry information should be done carefully and uniformly.

The new web-based database with analysis and evaluation capabilities as developed by the University of Washington shows a lot of promise as a system for practical implementation of linking. Data can be entered with Excel, Access or comparable files. The University of Washington system offers a range of evaluation, analysis and reporting possibilities, but it is also possible to download the raw data and perform these analyses in another way at your own location. Appendix B gives further details.

Administratively it will be necessary to assign someone for data processing and storage. In computer parlance a "Data Administrator" will be needed. If the data in each of the two or three subdata-elements remain independent of each other, there may be a gap in the compatibility of the data for future analysis. Rather than create a separate position or slot for this data administrator, it is likely that someone charged with data responsibility in materials, can start or Pavement Management should be responsible. If the Pavement Management Section is charged with the responsibility of evaluation and analysis, then the best place for data administration is PMS. However, this data administrator must be aware of the needs and workload imposed on the materials section or other sources for the detailed information. As in most other undertakings good relations and coordination between the sub-elements are required.

Ideally, the pavement management or asset management section within a DOT would recognize the multiple goals of a good pavement or asset management system: Network level, Project level and Research and Evaluation. In this case, recognition can be extended beyond Superpave analysis to other new concepts such as an evaluation of the results of using the AASHTO 2002 Guide, new concrete pavement technologies, etc.

In the specific case of Superpave, someone intensely interested in the performance of Superpave can become the data administrator and insure that the materials and auxiliary data are collected and then interfaced with the PMS database to obtain from that source the additional information needed for the ongoing analysis.

Since Superpave performance has a time history dimension to the data evaluation follow-up, the data required for analysis should be collected and evaluated as early as possible and the process should be continued on an annual basis for five to 10 years to better define the performance histories. We must remember from the AASHO Road Test [Hudson 68b] and other long term studies, that only abnormal failures are observed in the first two or three years. True performance based on long term observations usually take a minimum of five years and more often 10 to15 years of observation. For true success, some administration unit should be charged with the data activities.

3.5 SPECIFIC REQUIREMENTS FOR LINKING DATABASES (WITH SUPERPAVE AS EXAMPLE)

3.5.1 General Requirements

1. All data must be available in electronic format in systems with search and filter capabilities. Most PMS databases conform to this requirement, but materials, design, testing, construction and QC/QA data are often stored in flat files.
2. The electronic evaluation system, linking the PMS database with the Materials/Construction/QC databases, should be manageable and user friendly, it is recommended that universal software be used so that the output can be easily reported, exchanged and compared;
3. Each database should have an "owner" who is responsible for the timely upkeep and the quality of data. These "owners" should work in an organizational structure that facilitates open communications among them.

3.5.2 Performance Evaluation Data

1. The performance evaluation data must be linked to the correct materials and construction data, using a common denominator such as project number and exact mile point (or GPS coordinates). This requires three checks:
    a. Check the exact location (mile point and lane) of the material in question. This will only be possible when records are kept of the placement of each batch or lot as produced by the hot mix plant;
    b. Check the exact location of the testing of in-place properties such as density/degree of compaction and thickness.
    c. Make sure that the location where performance data are collected does indeed correspond with the location of the material being studied.
2. Use the appropriate performance indicators for the distress or performance being studied. For Superpave this could mean comparisons of rutting, cracking, and/or ride quality when studying effects of mix or binder type, compaction effort, traffic loadings, temperature, age, types of specifications, etc. Other examples are given in Section 8.
3. Make sure that the performance indicators used are properly defined, standardized and consistently applied.

3.5.3 Environmental and Traffic Data

1. Incorporate relevant climate data (temperature, rainfall, etc);
2. Check proper drainage;
3. Make sure reliable information is used for actual traffic volume and loads.

3.5.4 Materials Data

1. Limit information for materials properties, design and testing to data that is essential to an individual DOT;
2. Make proper categorization of (Superpave) mix type, such as coarse or fine, binder type (grade bumping), etc;
3. Check whether mix design was done according to specifications and appropriate performance testing.

3.5.5 Construction and QC/QA Data

1. Information is required on total pavement structure, including subgrade, and actual layer thicknesses and strength;
2. Check that actual thicknesses conform to the pavement design specifications;
3. Check whether as-placed materials properties, including stiffness and degree of compaction conform to specifications.

3.5.6 Examples for Superpave Mix Performance Monitoring

Examples are given below of data from the PMS, Materials and Construction databases that are relevant for the monitoring of the performance of Superpave:

From PMS Database

1. Performance Data, such as ride (IRI, etc), rutting (identify contributing layer), cracking (fatigue, low temperature, reflective, etc), surface deterioration (raveling etc) and skid resistance;
2. Location (mile point, lane) and project number;
3. Traffic loadings (ESALS) and climate data;
4. Age of Superpave pavement.

From Materials Database

1. Asphalt/binder test data & PG classification (e.g. PG 64-22);
2. Aggregate test data, such as coarse and fine aggregate angularity (CAA, FAA) and gradation (coarse, fine, control points, restricted zone);
3. Other (sand equivalent, etc);
4. Laboratory mix test data, such as Gyratory test data, volumetric properties, water sensitivity test data and mix performance test data like Hveem "S" value, Marshall test, Rut test, Stiffness (E*, phase angle), creep test, repeated load test, shear test, axial load test, etc.

From Construction and QC/QA Data

1. Location, mile point, lane, and project number;
2. Asphalt mix composition as placed (grading, binder content and grade);
3. Voids content (VIM, VMA, VFA) and degree of compaction;
4. Actual layer thicknesses (e.g. from cores, or non destructive testing);
5. Other relevant construction information (rain, delays, etc).

Examples of Parameters to Investigate

The four examples given below illustrate how the process of linking the PMS data to the Materials database can be used to evaluate the Superpave mix technology, which includes the binder specification, aggregate requirements, volumetric mix design and accelerated performance tests of mixes if available.

1. Low temperature cracking: plot degree of transverse cracking as a function of time for different Superpave binder grades in different climates. These plots, together with an examination of field construction data, may indicate changed aging requirements in MP-1 binder specs;
2. Grade bumping: compare rut depth as a function of ESAL's for two grades, e.g. PG 64-22, and a grade bumped to PG 70-22. The latter is expected to show less influence of higher ESAL values.
3. Effect of fine aggregate angularity (FAA): check the observed rutting for several levels of increased FAA levels for similar traffic, environment and mix design.
4. Effect of aggregate gradation (coarse vs. fine) on roughness: Check roughness levels for three levels of traffic loading for two types of Superpave mixes, one with fine, and the other with coarse aggregate, and make sure that all other conditions are comparable. One would expect the coarse mix to show higher roughness values, this effect might increase with higher ESAL values.

These four examples are illustrated in more detail in Appendix A.

3.6 EXAMPLES OF PERFORMANCE PLOTS OF SUPERPAVE PROVIDED BY VARIOUS STATE DOT'S

3.6.1 General

Although there were differences among the organizations and procedures of each DOT, they all shared a similar approach in reporting data for Superpave. All five pavement management systems could identify the location of Superpave sections, together with performance data for a few years (currently one to four years), but did not give any details on type of mix, as-constructed thickness, construction details, or any quality assurance (QA) or quality control (QC) data. It also was universal that materials, construction, and QA/QC data, which are crucial for a proper analysis of a material like Superpave, were kept in files in different sub-organizations, such as "Materials" and "Construction". In some DOTs the materials files were in electronic format, often existing of tailor made Excel files, but construction data were mostly stored in flat files. This practice makes it extremely time consuming to retrieve data from flat files in various locations, particularly when the project is several years old.

The state DOT's of Maryland, Indiana, Florida, Arizona and Washington have submitted data on several Superpave projects carried out in their states. TRDI has selected those projects with performance data for at least two years. That selection resulted in a total of 30 projects: three from Florida, seven from Indiana, three from Arizona, thirteen from Maryland and four from Washington. Superpave has been used in these five states for hundreds of projects, some of them as early as 1995. TRDI collected information on a total of 56 projects, but 26 of these projects had inadequate data to link performance to materials properties for at least two consecutive years. These projects were either too recent, or their location was such that no performance measurements were available (e.g. in the left lane of a multilane highway, a short section between mile points, etc).

3.6.2 Superpave in Maryland

Maryland State Highway Administration (MDSHA) started designing and constructing Superpave mixes in 1995, and since 1998 virtually all asphalt mixes for new construction and rehabilitation use Superpave. The introduction of Superpave was facilitated by the fact that the previous asphalt mix compositions in Maryland were already similar to those in Superpave. As of 2000 there were almost 200 projects constructed using Superpave in Maryland.

MDSHA supplied data for 22 Superpave projects, constructed between 1995 and 2000. For 13 of these projects the performance data were available for more than one year. MDSHA provided the following summary for the 22 Superpave they furnished to us:

"The following data have been retrieved and are presented:

Project limits and location
Date project was opened to traffic
General comments on the pavement condition before the treatment was applied
Design inputs – life of design and resilient modulus of subgrade
Date of last major rehabilitation
Pavement layer data – thickness, mix type, ESAL category, PG binder, tonnage
Network level ride and friction data for past three years

Additional information is available in project design and construction files. The following comments are provided with regard to these data:
The projects selected were among the oldest Superpave projects in MD to provide as many years of performance data as possible. Unfortunately, the approach to design these projects at the time was considerably different that the current approach. Many of the projects were designed years before construction and changed to Superpave mixes at the time of construction. As a result, little to no information is documented on the selection of the various Superpave mixes. However, all projects in MD that were designed after June, 1999 do include a very detailed

approach to assessing existing conditions and selected appropriate designs (the attached projects were all designed prior to June, 1999).
MDSHA has developed and implemented completely new pavement design guidelines that were implemented in the middle of 1999. These guidelines include very detailed condition surveys as well as updated design analysis procedures. More recent projects include more information on the design approach, distress quantities, repair quantities, effective structural condition and traffic data. These projects are no more than one year old and are available for consideration in this evaluation as well.
Limited construction and materials details are included. Information such as time of year of construction, detailed mix properties and quality control data will certainly be of use in this evaluation. This information requires more effort to access."

Two additional points about the Maryland projects are worth noting: They use a special wearing course of stone-matrix asphalt (SMA) extensively for major roads in Maryland. Consequently, in many cases Superpave mixes are not exposed to traffic. For this study MDSHA generally selected and provided project data where a Superpave mix was used as wearing course. Unfortunately no data for cracking were available for the Maryland projects.

Pavement evaluation ratings, and the criteria used with these ratings, are the following:

- IRI (International Roughness Index), expressed in accumulated inches per mile for two lanes into each direction. The criteria are Very good (<60 in/mi), Good (61-94 in/mi), Fair (95-119 in/mi for Interstates, 95-170 in/mi for other roads), Mediocre (120-170 in/mi for Interstates, 171-220 in/mi for other roads) and Poor (>170 in/mi for Interstates, >220 in/mi for other roads)
- Rutting, expressed as the maximum rut depth in inches in each section in both wheel paths, for two lanes into each direction, measured with the ARAN vehicle.

3.6.3 Superpave in Indiana

Superpave has been used since 1995, and from 1997 onwards all contracts over 4000 Mg of base or intermediate course or 2400 Mg (mega grams) of wearing course use Superpave. The INDOT staff tries to follow Superpave specifications without modifications. Mixes are designed in conformance with the Superpave method (code 401), but use is also made of a generic Superpave recipe (default design, code 402). Up to 15% of RAP (recycled asphalt pavement) is used on shoulders and in binder courses without a change in asphalt grade. RAP is not used in wearing courses by INDOT.

More than 100 Superpave projects have been carried out so far. A major effort would be required to convert all materials and construction information on these projects into electronic format, assuming that all data is still available. Our Project Team requested performance data for a limited number of sites, and INDOT personnel agreed to make the required information available in electronic format for seven projects with Superpave-designed mixes constructed since 1997. The mixes for these projects are designed in conformance with the Superpave method (code 401), not with a generic Superpave recipe (default design, code 402).

The ride is expressed as IRI and measured in inches per mile. The following criteria are used: excellent (60-100), good (100-150), fair (150-200) and poor (>200).
Rutting is a measure of the average depth in the wheel paths, rutting above 0.25" is considered severe.

INDOT uses PCR (Pavement Condition Rating) as a measure of pavement surface distress. The PCR includes transverse, longitudinal and block cracking. The distresses are rated for severity and quantity, and "deduct points" are determined, which are subtracted from 100 to determine the PCR. The ratings for PCR are: excellent (100-90), good (90-80), fair (80-70) and poor (<70).

The plots for the performance indicators show a discontinuity between the years 1997 and 1998. There are two possible causes for these variations:

1. First, INDOT has serious reservations about the quality of these data, collected by a contractor in 1997. They therefore changed contractors to collect performance data in 1998 and 1999.

2. In addition the exact date of completion was difficult to establish for some of the projects. It is possible, therefore, that some of the performance data in 1997 were actually taken on the pavement prior to its rehabilitation and laying the Superpave.

3.6.4 Superpave in Florida

So far about 700 Superpave projects have been completed in Florida. Since 1998 all mix designs conform to Superpave criteria. It is normal practice for FDOT to apply a friction course as top layer for all mixes, including Superpave mixes used on Interstates. FDOT gave the Project Team a list with of 24 projects for the years 1997-2000, where the major structural component was a Superpave mix. Out of these 24, three projects could be selected based on two criteria:

1. At least two years of performance data must be available,
2. Performance, materials and construction data were available and relatively complete.

Details for these three projects are given in Appendix D.

The pavement distress is recorded as PMS data and expressed in crack, ride, and rut ratings. The tabular data also cover segment number, county, year of contract, direction, mile points, road number, rating year, age, WPA (Work Program Administrator) data, and contractor. Statistical information on the performance data is included in the PMS database.

The distress ratings, and the criteria used for these ratings, are the following:

- IRI (International Roughness Index), converted since 1991 into a Ride Rating, reported on a 0-10 scale to the nearest integer value, with 10 as best condition;
- Rutting, a measure of the average rut depth in both wheel paths, converted to a Rut Rating and reported on a 0-10 point scale to the nearest integer value, with 10 as best condition;
- Cracking, assessed as percent confined to wheelpaths (CW) and % outside of wheelpaths (CO), a crack rating is a combination of CW and CO, derived from established distress rating scales, and reported on a 0-10 point scale to the nearest integer value, with 10 as best condition. Crack type is reported as C=combination, B=block and A=alligator.

Segments having a Ride, Rut or Crack Rating of six or below are eligible for resurfacing.

Cracking is reported, but crack sealing is not done as a routine maintenance activity because experience in Florida suggests that sealing of cracks does not extend pavement life.

Material properties of the related Superpave mixes and traffic data can be accessed electronically (through a tie-in to bid files), and design thicknesses should be found in paper files. No information is readily available about actual thicknesses of the Superpave layers, but we were informed that these depend on the aggregate size, as a general rule for SP 9.5 mm, for SP 12.5 mm, and for SP 19 mm the thickness would be 1½”-2”, 2”-3” and 3”-3½” respectively.

On average about 20% RAP (reclaimed asphalt pavement) is added to Superpave mixes (30% to Marshall mixes); about 60% of all Superpave projects in Florida contain RAP.

3.6.5 Superpave in Arizona

From 1993 until 1996 ADOT’s Materials Group in cooperation with the FHWA designed and built a number of Superpave test sections. For the hot mix design use was made of the SHRP gyratory compactor. Towards the end of 1996 it was decided that ADOT would move from a test section phase to a pilot project phase of evaluation and implementation of the SHRP gyratory method of Superpave mix design.

ADOT has made an overview of 21 Superpave projects completed in 1997 and 1998 in various parts of Arizona. The total length of these projects is about 122 center line miles on Interstates and other main roads.

For each project the following information is recorded: Project Number, Tracs No., Project Name, Contractor, Year built, Mile points, ESAL's, Elevation, Design Air Temp., PG Grade, Mix Size, Gyratory compactor testing details, Surface Course, Overlay, and Mill/Replace. In all projects the final surface course consisted of ½" thick rubber-modified porous asphalt friction course (AR-ACFC).

ADOT's overview is very comprehensive, but does not contain as-laid layer thicknesses, or any performance data. After checking with the ADOT PMS group it appeared that three projects could be identified with at least two years of performance data (data for the year 2000 were not yet entered into the PMS database). For the three projects with performance data, information about construction data was extracted from Pavement Design Summaries and from Superpave Project Tracking Survey Questionnaires (both in flat files, and not available for all projects).

Distress is recorded in ADOTs PMS for cracking, flushing, patching, rutting, friction and roughness in the right lanes only. For cracking, rutting and roughness the following is relevant:

- Cracking is measured through a visual survey of the first 83 ft at each mile post, covering roughly 1,000 sq. ft. and expressed as percentage of that area. The following criteria are used: Low (<10%), Medium (10-30%) and High (>30%).
- Rutting (measured since mid eighties with a four foot straight edge), determined from several short sections in the wheel path spread evenly over the entire mile after each mile post, averaged per section and expressed in accumulated inches. The following criteria are used: Low (<0.25 in), Medium (0.26-0.50 in) and High (>0.5 in).
- Ride is expressed as Roughness, measured after each mile post for the entire mile with a Mays Ride Meter, and since the mid nineties with a Profilometer, expressed in accumulated inches per mile. The following criteria are used: Satisfactory (<93 in/mi), Tolerable (94-142 in/mi) and Objectionable (>143 in/mi).

3.6.6 Superpave in Washington State

WSDOT began building Superpave projects in 1996, and since 1999 almost all their asphalt hot mixes are Superpave. Performance grade binder implementation started in 1999. In total some 40 Superpave projects have been completed so far, four projects were done in 1997, nine in 1998, 10 in 1999 and the remainder since then. Hot mix design uses all SHRP specifications and mix design techniques.

All materials and construction data are collected and stored in Excel files. The Civil and Environmental Engineering Department of the University of Washington (UW) has, in a joint effort with WSDOT and NCAT, recently developed a fully integrated website that contains relevant data of Superpave contracts, including performance data. This new development was possible because nearly all essential data on materials and construction are available electronically. In addition a major effort has been made to link these data to performance measurements from WSDOT's Pavement Management System (PMS). The data are available on a series of web pages (see Appendix B) from which they can be easily organized, downloaded and analyzed. Although this development is still in a pilot stage, it represents a major step in fulfilling the objectives of the project that is the subject of this report.

The project team received an Excel file with details of 12 Superpave projects, including PMS performance data and materials/construction data. Of these projects, four contained performance data for at least two years.

Most Superpave mixes are based on a 12.5 mm grading and binders have been used ranging between PG 58-22 and PG 70-34.

The website with the hot mix database gives detailed project information for each contract. Currently the website shows the following data:

- Project information with location, dates of construction, total tonnage, and type of mix and binder,
- Mix Design information with details of gradation, binder content, voids, density, gyratory compactor results, TSR, Sand Equivalency, and performance test data for creep and Hveem stability,
- Construction details with lot number, date, gradation, binder content, voids, density, gyratory compactor results, and RAP %,
- Performance data for five years (before and after construction), PSC, rutting (PRC) and IRI and distress details. All performance data have been linked to materials and construction data.
- Various graphing windows for plotting several relevant data for the lot numbers in each project.

The weather conditions in Washington State can be harsh, and consequently extensive use is made of studded tires during the winter season. WSDOT experience shows that friction courses do not stand up to the abrasive action of studded tires; consequently the Superpave mixes are directly exposed to this wear. Experiments with wearing courses of stone mastic asphalt and/or gap graded mixes with larger size aggregates are being considered.

Performance data are collected on an annual or bi-annual basis in the outside lane (for a multi-lane highway) in two directions for the following parameters:

- Structural Condition, based on surface distress data (fatigue, longitudinal and transverse cracking, flushing, and patching) measured since 1999 with a Pavement Condition Van at highway speeds, using continuous video imaging of the pavement surface, ahead view, shoulder view, and vertical views. From these distresses a Pavement Structural Condition (PSC) is calculated on a scale from 0 to 100. The range 100-75 is Very Good, 75-50 is Good, 50-25 is Poor and below 25 is Very Poor.
- Rutting Condition, measured continuously in the wheel path with the help of lasers on a cross beam on the Pavement Condition Van, expressed in a Pavement Rutting Condition (PRC) and calculated in average rut (in mm) over the previous mile. The condition limits are: Very Good when < or = ¼", Good when > ¼" and < or =1/3", Poor when > 1/3" and < or = ½" and Very Poor when > ½".
- Roughness Condition defined by the International Roughness Index (IRI), measured continuously with laser sensors mounted on the Pavement Condition Van, expressed in accumulated inches per mile. The condition limits are: Very Good when < or = 95 in/mi, Good when > 95 and < or = 220 in/mi, Poor when > 220 and < or = 320 in/mi and Very Poor when > 320 in/m.

3.6.7 Examples of Performance Plots

Figures 3.3 through 3.5 give example curves of performance plots for ride, rutting and cracking for Superpave projects. One example for each performance indicator is presented.

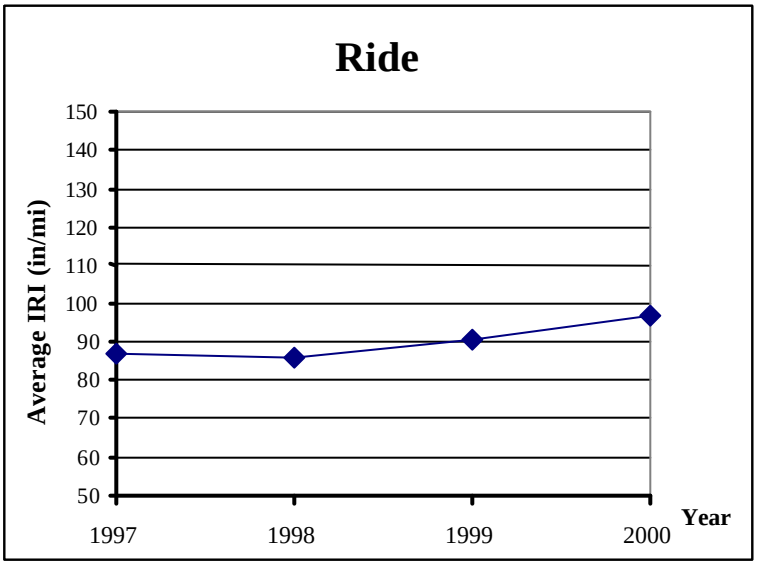

Figure 3.3 Example of Performance Curve for Ride

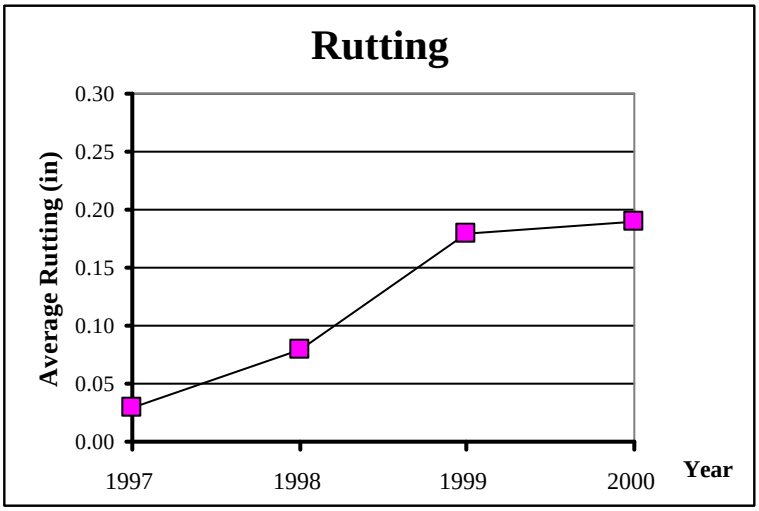

Figure 3.4 Example of Performance Curve for Rutting

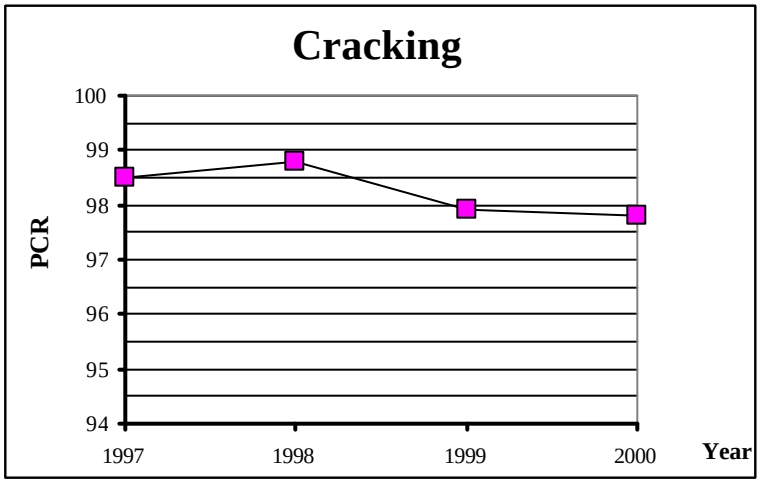

Figure 3.5 Example of Performance Curve for Cracking

These graphs are presented for illustration purposes only; it is not possible to draw any conclusions about the performance of Superpave. For a full analysis many more projects would be needed, each one with more complete data about traffic loading, actual thickness, existing pavement condition (pavement layers, subgrade, etc.), environmental conditions, etc.

3.7 PHASE 2 – PATHFINDER STUDY IN MARLYAND

3.7.1 General

The results of the study described in the previous sections were presented and discussed during a project review meeting with several FHWA representatives in Washington DC.

As a follow up TRDI received a Purchase Order to extend the study for Phase 2 with the following statement of work:

> *"Additional data elements for measuring pavement performance have been identified by TRDI and recommended to be linked with the pavement management database. Based on this recommendation, TRDI will return to Maryland and take information that is currently in flat files or any other inappropriate form and put it in electronic format. TRDI will then electronically link the databases (i.e. materials, PMS, construction, and quality control) in such a way that engineers can readily access the data from all the databases for the purpose of analyzing pavement performance at the network level for long enough periods of time so it can cover a complete life cycle.*
>
> *Specifically, TRDI will return to Maryland to take data needed to track the performance of Superpave projects and move from flat files to an electronic format and then link the databases (materials, construction, QC and pavement management) as an example to show the states how this should be done. Only one state, Maryland, will be revisited and a report written to document the process".*

As a first step, the Project Team met with representatives of Maryland SHA to discuss the following:

1. Database to be used for linking performance/materials properties/construction details (mix design and testing, construction, performance, maintenance)
2. Amount of Data Transfer for Path Finder Study (period, types & number of projects)

3. Format of electronic database (AASHTO Superpave, Excel, Web-site)
4. Process of Data Linking (including analysis and reports)
5. Efficiency and Reliability of Data Collection, Transfer and Linking process (how much time and how much detail are needed)
6. Example(s) of actual analysis of Superpave performance (in final report)
7. Actual staff arrangements

MDSHA prepared a list with proposed data fields, data availability and comments for QC/QA data, Mix design information, Pavement design information, and Pavement Management data, together with a suggested list of reports that should cover the effects of various parameters on rutting, reflective cracking, ride quality and cracking. The Project Team proposed to add a few data fields in the areas of existing pavement structure, mix temperatures during spreading and compaction, performance data of laboratory- and actual mixes, and date when pavement section is opened to traffic.

The Maryland PMS does not yet contain data on maintenance, but it was agreed that the linking system should provide for the inclusion of maintenance activities in the future.

The recommended data fields are given in Table 3.2, and Table 3.3 lists the types of analysis and evaluation that would be desirable.

Table 3.2 Recommended Data Fields to Include in Pilot Study

Key to Data Type Column : E=Electronic, PF=Project File (paper), N/A=Not available

Data Field	**Data Type**	**QC/QA**	**Mix Design**	**Pavem. Design**	**PMS**
AADT	E				v
Aggregate Consensus Properties	PF		v		
Aggregate Source	PF		v		
Aggregate Type	PF		v		
Ambient Temperature	PF	v			
Asphalt Content	E	v	v		
Asphalt Producer	E	v			
Binder Test Results	E		v		
Binder Type	E		v		
Compaction Level (N Design)	E		v		
Cracking Condition	E				v
Cumulative ESALs	PF			v	
Daily Paving Location (location of lots)	PF	v			
Date when pavement section was opened to traffic	N/A				
Density (field compaction) from cores & nuc gauge	E	v			
Density at N Max	PF		v		
Design Constraints	PF			v	
Design Life	PF			v	
Existing pavement layer thicknesses	PF			v	
Friction Condition	E				v
Gradation	E	v	v	v	
HMA Unit Price	PF	v			
Layer Type	E				4
Load Spectra	N/A			v	
Location of Project	E				v
Longitudinal Joint Condit ion	N/A				v
Max Gravity	E	v	v		
Milling/Grinding Recommended	PF			v	
Mix Design Number	E	v			
Mix Pay Factor	E	v			
Mix Temperature prior to compaction	PF	v			
Mix Temperature after compaction	PF	v			
Mix Type	E		v		
No and Type of Layers Recommended	PF			v	
Patching Recommended	PF			v	
Pavement Type	E				v
Paver Type	PF	v			
Paving Contractor Name	PF	v			
Paving Time of Day (Day or Night)	PF	v			
Performance Test Results	N/A	v	v		
Pictures	N/A	v			
Pre-overlay Condition	PF			v	
QA Sample Location	PF	v			
RAP Quality	PF		v		
RAP Quantity	E		v		
Rating of Joint Construction	N/A	v			
Rating of Segregation	N/A	v			
Raveling Condition	N/A				v
Reflective Cracking Condition	N/A				v

Ride (profilograph results, profiler results in 2002)	PF	v			
Ride Condition	E				v
Ride Pay Factor	PF	v			
Rolling Pattern	N/A	v			
Rutting Condition	E				v
Section (open or closed, manholes, entrances)	PF			v	
Soil Strength	PF				v
Soil Type	E				v
Special Features (joint tape, saw seal, etc.)	PF	v			
Structural Capacity	PF				v
T-283	PF		v		
Thickness of new layer(s)	N/A	v			
Tonnage	PF	v			
Transfer Device (Y/N)	PF	v			
Type of Improvement Designed	PF			v	
Type of Rollers	PF	v			
Volumetrics	E	v	v		
Weather Conditions	PF	v			
Wedge and Level Recommended	PF			v	
Why Binder was Bumped	PF				v

Table 3.3 Analysis and Evaluation Reports

Effect of the following items on Rutting: ***Mix Type (gradation and compaction level)*** ***Fine Aggregate Angularity*** ***As Built Density*** ***Asphalt Content*** ***Percent Passing #200 Sieve*** ***Cumulative ESALs*** ***Condition and Structure of Existing Pavement*** ***Thickness***	**Effect of the following items on ride quality** ***Constructed Ride Quality*** ***Use of Material Transfer Device*** ***Condition of Underlying Pavement (ride and distress)*** ***Mix Type (gradation and compaction level)*** ***Reflective Cracking*** ***Day or Night Paving***
Effect of the following items on reflective cracking ***Binder Type and Grade (use of polymer)*** ***Type of Patch at Joint*** ***Special Features (joint tape, etc.)*** ***Thickness of Overlay*** ***Asphalt Content*** ***Condition of Underlying Pavement (load transfer%)***	**Effect of the following items on cracking** ***Constructed Density*** ***Binder Type and Grade*** ***Thickness of Overlay*** ***Asphalt Content*** ***Condition of Underlying Pavement*** ***Mix Type (gradation and compaction level)***

3.7.2 Collecting Data and Conversion to Electronic Format

In Maryland only the Superpave projects designed after June 1999 have sufficient data to make the performance analysis meaningful. MDSHA therefore selected 7 Superpave projects carried out since 1999, and made an effort to collect all available data listed in Table 3.2 from their QA/QC, Mix design, Pavement design and Pavement management records, and to import these data into an electronic database. Apart from the data that were not available (N/A in Table 3.2) they were unable to retrieve data on Aggregate Properties and on Binder Test Results. Other data, mostly those in paper files, were hard to retrieve. An overview of the sources of data and the difficulty to retrieve them is given in Table 3.4. For the seven Superpave projects it took 2 staff members about 4 weeks working full time to retrieve the data and to enter them into an electronic database. They used an Access database about 45 MB in size. Electronic data storage will in the future improve this process and speed it up considerably.

Table 3.4 Degree of Difficulty in Collecting the Data Listed in Table 3.2

SUBJECT	SOURCE	DIFFICULTY TO RETRIEVE
Inventory Information	Design & Constr. Proj.Files	Medium
Daily Paving Information	Construction Project Files	Hard
Project Paving Information	Construction Project Files	Hard
Density QC and QA	QC/QA Database	Medium
Mix Design	QC/QA Database	Easy
Mix QC and QA	QC/QA Database	Easy
Aggregate Properties	MD AASHTOware Program	Unable
Binder Test Results	MD Binder Database	Unable
Ride QC and QA	Construction Project Files	Hard
Pavement Design&Recommend.	MD Pavement Design Files	Medium
Pavement Management Data	MD PMS Database	Easy (no cracking data yet)
Pre-Overlay Pavement Condition	MD Pavement Design Files	Medium
Pre-Overlay Pavement Layers	MD Construction History DB	Easy
Project Condition Ratings	None	Hard (collected in the field)
Pictures	None (not currently stored)	Hard

3.7.3 Setting up Database in modified UW Web-site System

As mentioned in Section 6.6, the University of Washington (UW), in cooperation with Washington State DOT and NCAT, recently developed a fully integrated website that contains relevant data of Superpave contracts, including performance data [White 02]. This new development was possible because nearly all essential data on materials and construction are available electronically. In addition a major effort has been made to link these data to performance measurements from WSDOT's Pavement Management System (PMS). The data is available on a website (see Appendix B) from which they can be easily organized, downloaded and analyzed.

In the initial phase of the study the Project Team considered the use of a modified version of the QC/QA module of the Superpave/AASHTOware data set [AASHTO 00] for collecting and storing the required Superpave data. After further review and knowledge of the University of Washington system, two reasons emerged to support why it proposed to use the web based system developed by UW for the Maryland Database with Superpave projects:

1. AASHTO recently decided to stop promoting and servicing their Superpave/AASHTOware data set package.
2. Several of the data fields listed in Table 3.2 are not included in this AASHTOware package.

Dr. Joe Mahoney and Mr. George White of the Civil & Environmental Engineering Department of UW offered their help in making their web-site system available for the seven Superpave projects from MDSHA. In order to do this George White undertook the following:

- Modify and extend the system to:
    - Accommodate all data fields given in Table 3.2
    - Extend the functionality of the system to include the analyses and plots listed in Table 3.3.
- Checking the MDSHA database and entering the data in the web-site system.

The total effort of George White for the above mentioned activities was about one man-month. Most of this effort went into the last point, clarifying the meaning and inter-relationship of the data, getting additional data, adjusting units, converting systems, etc.

UW received most of the data from MDSHA in the form of a substantial database with complete data for most of the fields. Still, for 10 fields no data were available at all, and for several other fields there were insufficient data. Consequently no analysis could be done for cracking or other distresses, for the effects of mix temperature, the influence of the use of a MTV, the influence of day or night paving, the effect of different roller patterns or roller

types, or the effects of actual versus designed layer thickness. It appeared that data with exact locations of lots and sub-lots were also missing, but it was agreed for one section only to divide the lots evenly over that section so that, for demonstration purposes only, the correlation between a lot and its performance could be shown. Another limitation of the data for the seven Superpave projects is that there is only one or two years of performance data, so it is difficult to create meaningful plots of performance over time. Some of the older Superpave projects do have several years of performance, but these projects have insufficient data on materials and construction.

3.7.4 Demonstration of Capabilities of Web-site System of UW

The final presentation of the current project took place at the 2002 TRB Conference in Washington DC. The purpose of the meeting was to give the FHWA and representatives of industry, universities, committees and state DOTs an overview of the progress made and a demonstration of the web-based evaluation system developed by the University of Washington. Representatives of the FHWA, TRB, AASHTO, Maryland SHA, Washington State DOT, NAPA, Asphalt Institute, NCAT, Battelle and the Universities of Maryland and Washington attended the meeting.
It was stressed that the demo should be judged on capabilities, rather than the interim results.

The animated discussion after the meeting can be summarized as follows:

- It was clear that the system is very flexible, and that it is able to accommodate the different requirements of various agencies, but that there will be a need for some degree of consistency for data and the use of these data to make broader comparisons possible. One additional advantage of the system is that the information can be refreshed and updated at any moment, so it is possible to work on the basis of the latest available information. The use of "real time" QC/QA data during construction could be very beneficial.
- The FHWA is very interested to pursue the presented concepts, for instance by assisting with a Pooled Fund study in which various states can participate, but in the short term there is a lack of funds,
- Several representatives agreed that the concept would be a very useful tool to secure required pavement performance data for use in the AASHTO 2002 Pavement Design Guide and to evaluate the performance of pavements designed with this Guide,
- Industry representatives indicated their willingness to cooperate by promoting the use of data in electronic format. They considered it advantageous to have one complete set of data that could be used by all parties,
- All participants welcomed and approved of the concept, and it was realized that it is important now to capitalize on the considerable momentum that has been built up by the current promoters of the concept.

3.8 OTHER EXAMPLES OF USING PMS DATA IN PERFORMANCE MONITORING

In Sections 3.6 and 3.7 examples were provided illustrating how the linking of PMS and materials and construction data could be used to evaluate specific facets of Superpave technology. It is important to note that Superpave is only one example, although an important one, of what can be done with engineering analysis of PMS and related data. It was chosen as the primary example because the increasing use of this new technology for mix design and evaluation requires that performance information be obtained to test the performance of Superpave technology and whether modifications are necessary to enhance its usefulness.

The purpose of this section is to illustrate some other examples and to briefly point out the kinds of changes or additions that would be needed to PMS data collection procedures for a particular subset of pavement sections. For asphalt concrete (AC) pavements brief examples are given here for the performance and evaluation of pavement structures and overlays designed according to the forthcoming *AASHTO 2002 Guide*, and for the performance evaluation of porous friction courses, surface treatments, and asphalt treated permeable base (ATPB). For Portland cement concrete (PCC) pavements examples are given on the effect of aggregate type, the effect of maintenance procedures such as joint sealing and the effect of load transfer devices.

It must be emphasized that the basic PMS database with good location and referencing identification is critical to all of these evaluations. However, in each case a supplementary data set is required which depends upon the details of the materials, construction, or design techniques being evaluated. Each concept requires that some detailed materials or construction data be recorded for individual test sections and in some cases it is important that a small amount of additional performance information be recorded for these same sections. No modifications are needed to the basic data structure of the PMS database. However, additional sub-files keyed to the appropriate location identification should be set up with required query functions so that routine data may be extracted from the PMS database and assembled with the individual data subsets for analysis. Likewise, the data subsets can be imported into a PMS database for overall analysis as appropriate.

3.8.1 2002 Pavement Design Guide

The proposed mechanistic-empirical 2002 *Pavement Design Guide* presents a unique opportunity for the use of PMS data. New mechanistic concepts will be employed that have not been tested. Work should be done in 2001 to set up plans for a database with nationwide potential to initialize data for sections designed according to the 2002 Guide. State DOTs can be encouraged to set up the required data set and start collecting data using standard data collection protocols. In this way a broad, dependable performance database could be built.

Linking of PMS and materials and construction databases will be essential to the successful application/implementation of the 2002 Guide for pavement design and rehabilitation. Clear records should be made of design parameters, calculation, and predicted layer thickness and material properties. These should be followed with accurate records of asphalt thickness and properties.

To evaluate the performance models on which the 2002 Guide is based will also require a detailed materials database. Some of the elements of this database are:

- Material characteristics of the various components including subgrade, determined either in the laboratory and in-situ (e.g. by back calculation from FWD measurements) or both;
- QC/QA construction data for the various layers to assess both variability and reliability of design estimates and to assess as-built properties; and
- Other as-designed records.

To verify the models in the 2002 Guide, it will be important to ensure that good traffic/load data are recorded annually. Weigh-in-motion data will be desirable but as a minimum, good load spectrum and ESAL estimates must be recorded on an annual and seasonal basis. Annual performance measures should include distress, roughness and deflection data. The type and location of maintenance activities must also be recorded.

Comparisons of estimated and measured performance both as a function of traffic and environment will permit calibration and necessary modifications to be made to the performance models, thereby improving confidence of the transportation community in the use of the 2002 Guide.

3.8.2 Asphalt Pavement Examples

Porous Friction Courses

Porous friction courses of open – graded asphalt mix have been used for the last several years as a surfacing on both asphalt and concrete payments to carryout one or more of the following functions:

- Reduce the potential of hydroplaning, and consequently minimize loss of skid resistance at higher speeds;
- Reduce tire splash and spray, thus improving visibility to the road users; and,
- Reduce vehicle noise.

These purposes are fulfilled through increased pavement macro-texture and improved contact between tires and the pavement surface under conditions of heavy traffic and rainy weather. The function of the material depends upon the open pores created remaining open so that surface water will quickly drain, permitting the tires to continually contact the aggregate. The performance of such mixes is influenced by the composition of both the aggregate and the asphalt, the aggregate gradation, and by construction practices such as compaction, thickness, etc. There is no doubt that a properly placed porous course fulfills its goals. However, there is evidence that in a short time many porous friction courses fill up with debris and lose their effectiveness under road traffic.

Typically PMS performance indicators in the database include surface distress, ride quality and skid resistance. Additional indicators would need to be incorporated to study porous friction courses to measure noise levels and surface water permeability for the affected section. These additional data factors could be appended to a subsidiary data set keyed to pavement location since they will not routinely be collected on all PMS sections.

The materials database should include the following factors as a minimum:

1. Mix design, aggregate gradation, shape, and durability characteristics, both as-designed and as-constructed;
2. Asphalt/binder type and amount, both as-designed and as-constructed;
3. Density and void information as-constructed;
4. Layer thickness as-designed and as-constructed; and preferably,
5. Permeability measurements, both laboratory determined, and as-constructed.

As little as three or four years of these data combined with PMS data could be used to evaluate performance and could lead to a determination of the benefits and actual life of such porous friction courses. It could also lead to improvements in material selection and mix design procedures as well as construction practices, which would insure the desired functional performance of this type of free draining surface course.

Surface Treatments and Seal Coats

This category includes spray applications of asphalt covered by a single layer of aggregate to improve surface characteristics of the pavement structure. This type of construction encompasses fog seals (asphalt alone or a softening or recycling agent), chip seals, and slurry seals. Currently in the United States, application is limited to relatively lower volume roads and to maintenance of existing pavements under low and medium traffic volumes. In other countries such as New Zealand [Seal 87], Australia [AAPA 98], and South Africa [Emery 94] many miles of single and double surface treated roads are used to carry primary rural traffic.

Surface distress such as cracking and patching in a PMS database are obliterated when the surface is covered up with a layer of asphalt and stone chips or slurry seal. This creates a discontinuity in the PMS performance data. However, roughness data remain continuous and can be used for performance evaluation. For this type of pavement it is also necessary to add aggregate loss and bleeding as additional distress factors since they are important modes of failure.

The subsidiary materials and construction database for chip seals as an example should include:

1. Details of existing surface before application (smoothness, degree of cracking, cracking and patching, presence of bleeding, or raveling, etc.);
2. Type of treatment, single or double application, etc;
3. Asphalt/binder data – emulsion or cutback, grade, modification if incorporated in binder;
4. Aggregate data – gradation, shape, polishing tendencies, adhesion characteristics;
5. Application rates for binder (gallons/yd^2) and aggregate (lbs/yd^2);
6. Construction control data, curing time prior to traffic;
7. Environmental data (temperature, possible rain fall or humidity during construction procedures); and,
8. Laboratory test data (e.g., wheel tracking or abrasion test results).

These data should be placed in a database (subset) for surface treatments to be studied, and then keyed to the PMS database by location identifier. Performance evaluation would use both the PMS and the auxiliary database and should lead to improved chip seal performance predictions. In turn this will improve materials requirements and construction practices as well as add to the understanding of traffic conditions and surface conditions under which these procedures perform acceptably.

Asphalt Treated Permeable Bases (ATPB)

These mixes generally consist of a uniformly-graded relatively large size aggregate and a paving grade asphalt. They are usually placed in the pavement section directly beneath the HMA surfacing to intercept water entering from the pavement surface. An alternative application is to place this permeable material near the subgrade surface to intercept water, which might enter the pavement from subsurface sources. Some evidence suggests that ATPB may not be as effective as originally envisioned when it was adopted by many state DOTs to reduce the effect of surface water infiltration on pavement performance. The reduced effectiveness may result from lack of maintenance of the necessary side drains required for proper functioning of the drainage layer and/or infiltration of fines from the untreated base and subbase layers caused by heavy traffic and lack of a suitable filter layer.

Pavement performance data linked with materials data have the potential to define the efficacy of the use of the ATPB. In addition to the usual type of performance information obtained for the PMS database, data records relative to the performance of side drains are extremely important and must be recorded.

For the materials database, mix design and construction QC/QA data are important. In addition, special design features such as the use of filter fabrics or soil filters should be recorded as well as the location of the ATPB.

3.8.3 Example Uses of PMS Data for PCC Pavements

PMS data can also be used for performance evaluation of PCC pavements and various design and construction characteristics [Dossey 94, McCullough 95]. In most state DOTs PCC pavements make up a small but very important portion of the pavement network. For purposes of evaluation, a separate PCC PMS data set may be desirable and is often maintained because design factors, distress types, and maintenance methods are different from A/C pavements.

Continuously Reinforced Concrete Pavements

The first example relates to the performance of continuously reinforced concrete pavements (CRCP). Many states have banned the use of CRCP but the poor performance blamed for this situation may be due more to bad construction than to the pavement type itself. A nationwide study of PMS data might clarify this situation.

For this study, it would be necessary to pull PMS data records of pavement sections by pavement type and to identify CRCP sections and examine their performance life. Construction records would then need to be obtained for all performing sections for analysis and comparison. TXDOT, as one example, already has set up a performance analysis database, which is maintained for concrete pavements. Other state DOTs could do the same.

Effect of Aggregate Type

Based on work done in Texas [Dossey 94, McCullough 95] it is now widely believed that aggregate type and its coefficient of expansion can have a major effect on concrete pavement distress and performance. In Texas, pavement built with siliceous gravel aggregates show earlier failure than those with limestone aggregates.

In this study it would be necessary to examine the PMS database and define aggregate in some way perhaps by coring for direct observations. A direct comparison could then be made of pavement serviceability and distress history as a function of aggregate type and other variables [Hankins 91].

Other Concrete Pavement Studies

Many other concrete pavement studies could also be made using the PMS database. The effect of joint and crack sealing could be examined by studying recorded maintenance history versus PMS distress history. The effect of load transfer could be examined by comparing roughness history for pavements with and without dowels or CTB bases.

Retrofitting of existing un-dowelled, plain, jointed concrete pavements with dowels (dowel-bar retrofit) is now underway in a number of states. The effectiveness of this technique on pavement performance can be accomplished in those states, e.g. Washington, with well established PMS databases.

These are just a few examples of the many possibilities to use this methodology. Apart from monitoring various materials for structural and functional treatments one might also consider the following categories:

- Different construction techniques for laying, compacting, recycling, etc.
- Different types of contracts with incentives, warranties, etc.
- Maintenance techniques, including preventive maintenance.

3.9 CONCLUSIONS, FINDINGS, AND RECOMMENDATIONS

The project results show that it is possible for state DOTs to assemble a database that can be used to evaluate the performance of Superpave and other design- and new materials concepts. The project was not large or long enough to make a thorough evaluation, but it did determine the feasibility of the concept and its applicability among five states.

The many details and variables involved in a new methodology such as Superpave also requires recording data on design, construction, and performance for individual projects in addition to the ordinary PMS data. These details may extend normal pavement management activities in some states, but the results are well worth the effort. As the data are collected over five to ten years, results and updates of performance comparisons will provide substantial benefits and validation of the method. Each year the analysis of the growing database will produce definite results.

The key to linking databases for performance, materials and construction is to have precise and common location identification and date/time information. Only in this way can it be assured that the data are comparable. For example, in the case of multi-lane roadways involving bi-directional interstate highway lanes and frontage roads, the new material may be used in only one or two lanes or in one roadway direction. When the material is first placed, it is clear in everyone's memory, which is which, but four or five years later or after personal changes, when a long-term analysis is needed, this becomes more difficult. Unambiguous locations can be provided by GPS measurements and these are relatively economical and easy to use at the present time. However, they must be tied to traditional location identification information such as project number, mile point, lane, direction, date, etc.

In Washington State the University of Washington (UW) has, in close collaboration with the DOT, developed an approach which stores most relevant data for Superpave contracts on a web-

based system. The performance, design and construction data can be organized, downloaded and analyzed with the method. This new development was possible because many of the relevant data on materials and construction in WSDOT were available in electronic format (mostly Excel files), and in addition a major effort was made by UW to link these data to performance measurements from the PMS. This system was modified successfully to store, inspect, analyze and report relationships of Superpave data supplied by Maryland as part of the Pathfinder study.

A major advantage of the website approach is that all data are available to all users as soon as data are entered. With proper equipment for electronic data entry, used in the field, it is possible to monitor construction projects quickly with "real time" QC/QA data which could be beneficial.

The Pathfinder study has shown that the collection of relevant data in materials and construction files, required for linking with performance data, is cumbersome and time consuming. In Maryland it took two man-months to collect data for 7 Superpave projects, and even then not all required data could be obtained.

Ideally, all relevant data should be generated in electronic format from the start but there is also a need for a proper and unbiased definition of required data before they can be used in a linking exercise.

The review of information and data, as well as interaction with five state DOTs visited, suggests that with manpower for data entry it is possible for a state DOT to develop a Superpave evaluation database. More than $100 million has been expended to date to develop the Superpave concept, yet no actual performance results exist. It is now essential to compare real Superpave performance to current asphalt practice in state DOTs to validate the value of Superpave. Each state can use pavement management and related data to evaluate Superpave, but it will be faster and more definitive if several states can work together to set up databases with the required data and combine their efforts to make the necessary performance evaluations. A group of 5 to 20 states with coordination among states can produce a large analysis joint database of lasting value.

Preparation of a good work plan for evaluating new concepts using PMS data and such tools as standard Pavement Evaluation Protocols will be useful for state DOTs to encourage them to set-up appropriate evaluation databases and procedures for any new pavement concept they undertake. Similar methods are applicable in many state DOTs.

The approach examined in this project for Superpave is applicable for other new concepts. Specifically, now is an ideal time to set up a plan showing how several state DOTs can record the proper data to evaluate the new 2002 Pavement Design Guide when it is adopted by AASHTO. It is important to define this plan before the new method goes into effect.

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APPENDICES

A. APPENDIX A – LINKING SUPERPAVE MATERIALS DATA TO PMS PERFORMANCE DATA

This section provides a discussion of the linking of the PMS database to a materials database or materials databases, which would permit future evaluation of mixes, designed according to the Superpave mix design methodology. At present, the Superpave mix design procedure is limited to volumetric design considerations. Nevertheless, some agencies include a performance test or tests to aid in the mix design process. Accordingly the materials databases should include provisions for the inclusion of mix performance test data.

A-1 PMS Database Information

Pavement performance information in the PMS database should include the following:

1. Pavement condition data
    a. Rutting – depth
        i. Identification of contributing layer(s)
        ii. Bleeding, if any
    b. Fatigue cracking – extent
    c. Low temperature (thermal or transverse) cracking – extent, e.g. cracks per 1000 ft.
    d. Reflection cracking (e.g. overlays on PCC pavements)
    e. Surface raveling
2. Ride quality – IRI or other measure of surface roughness
3. Skid resistance
4. Traffic
5. Environmental data (if available); temperature, etc.

These data provide the requisite performance information to permit mix data contained in the *Materials* database to be used to evaluate the Superpave mix design methodology and the efficacy of mix performance tests used to assist in the selection of the final mix design.

A-2 Materials Database

The AASHTO Superpave software currently contains the information, which is used as a part of the Superpave volumetric mix design procedure.

For the mix design process this includes the following:

1. Asphalt binder test data – PG classification; (e.g. PG 64-22)
2. Aggregate test data
    a. Coarse aggregate angularity
    b. Fine aggregate angularity
    c. Gradation (see Figure 3.6)
        i. Coarse
        ii. Fine
        iii. Through the restricted area
    d. Other (e.g., sand equivalent)
3. Laboratory mix test data
    a. Superpave volumetric data
        i. Gyratory test data
        ii. Volumetric properties
        iii. Water sensitivity test data (e.g., AASHTO T-283)

b. Other performance data
   i. Marshall stability test data, Hveem and stabilometer "s" values
   ii. Laboratory rut test data; e.g., Asphalt Pavement Analyzer (APA) data
   iii. Stiffness data (E*, phase angle)
   iv. Creep or repeated load test data in shear or axial loading

4. QC/QA data

With the exception of the performance test data the AASHTO software contains the requisite place holders for such data. It will be necessary to include provision for the mix performance data noted in 3(b) above. In addition, provision will be required for asphalt thickness in the database.

With the above information the following section illustrates examples of the type of evaluations that can be made.

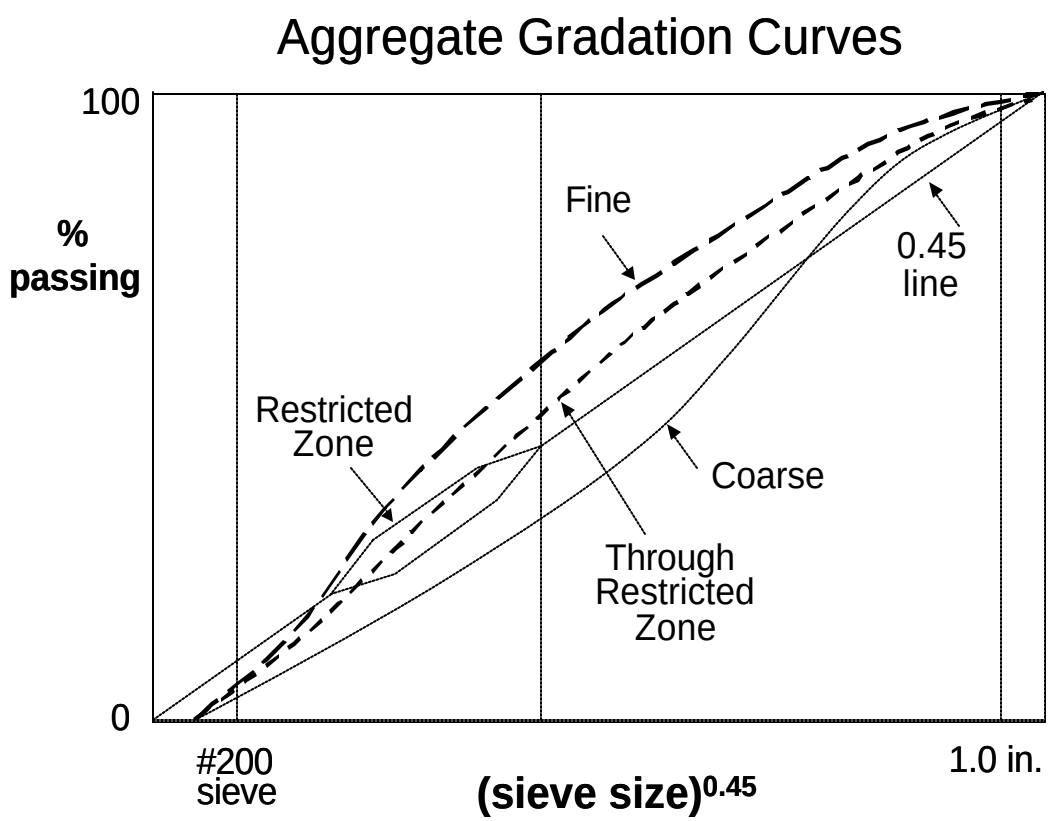


Figure 3.6 Aggregate Gradation Curves for Superpave

A-3 Examples of Data Plots

This section provides examples of the types of data that might be used to assess various aspects of the Superpave binder specifications (AASHTO-MP-1) and the volumetric mix design criteria as well as the usefulness of the performance tests (if any) utilized in the initial design process.

1. Low Temperature Cracking

The results in Figure 3.7 suggest that changed aging requirements may be necessary in the MP-1 binder specifications or that the mix may not have been properly compacted in the field causing excessive binder "hardening."

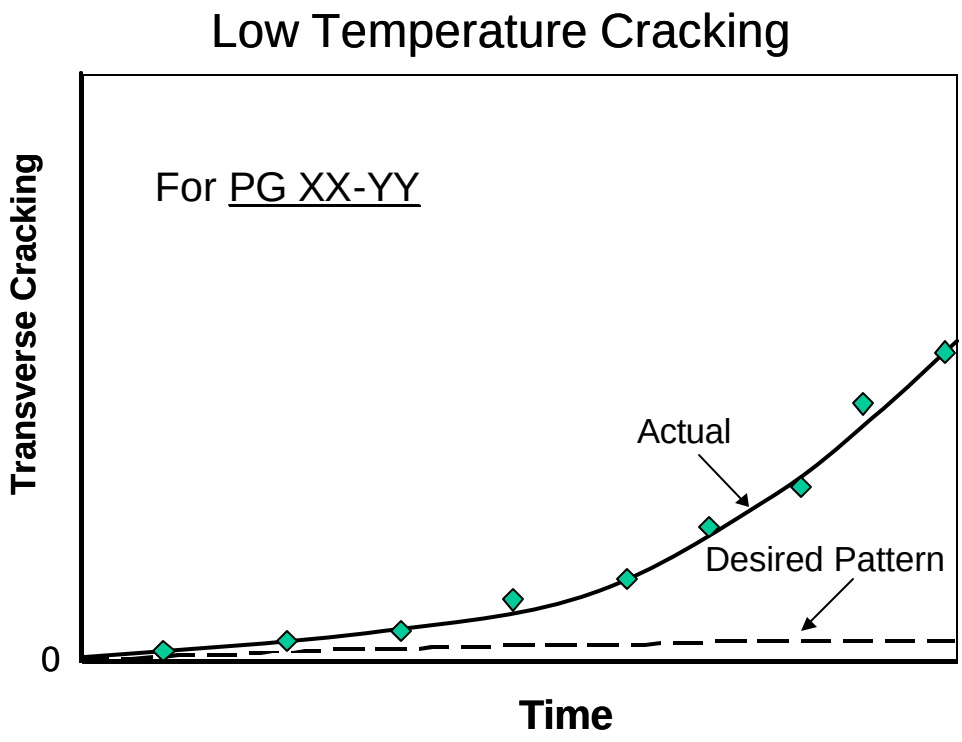


Figure 3.7 Low Temperature Cracking

2. Evaluation of Grade "Bumping"

For "heavy" traffic, grade bumping, i.e., increase by one or two grades is recommended. For example, if a PG64-22 binder were recommended based on the temperature requirements, increase of the binder grade to PG70-22 would be an example of grade bumping to accommodate the increased traffic.

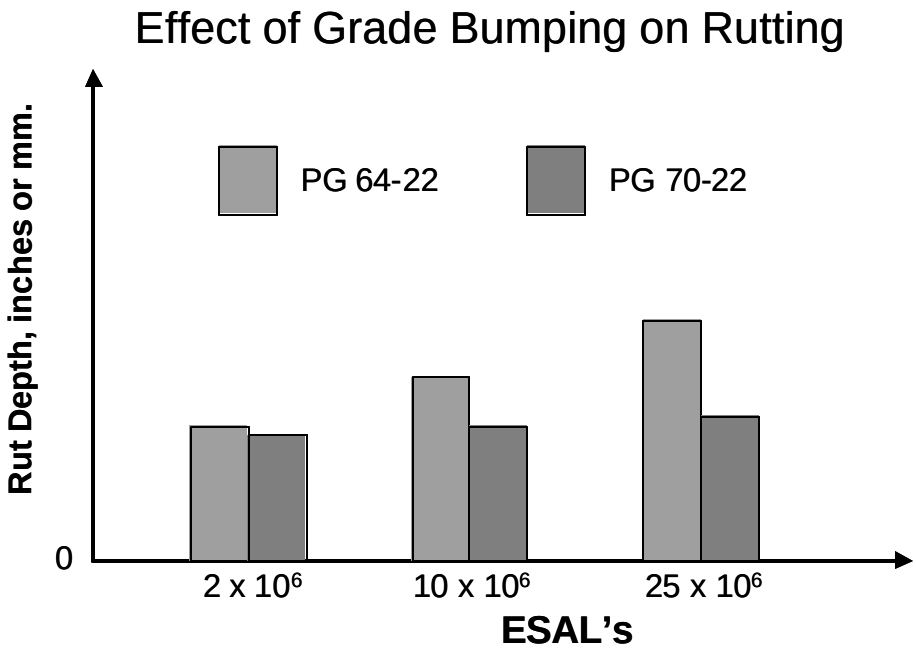


Figure 3.8 Effect of Grade Bumping on Rutting

The results in Figure 3.8 would suggest, for the projects compared, that grade bumping is effective in reducing rutting.

3. Effect of Fine Aggregate Angularity on Mix Performance in Rutting

The parameter Fine Aggregate Angularity (FAA) is used to control the characteristics of the fine aggregate in the mix. The higher the number (FAA), the less the amount of rounded sand, or even elimination of such material in the aggregate usually occurs. Thus one could plot, for a specific level of traffic, rut depth as a function of the FAA parameter.

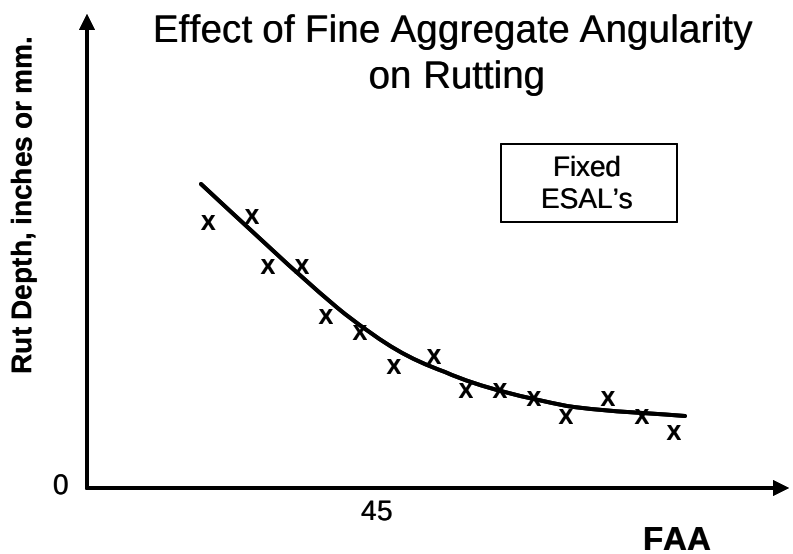


Figure 3.9 Effect of Aggregate Gradation (Coarse vs. Fine) on IRI

The results in Figure 3.9 would suggest for the level of traffic considered (and for mixes of the same type and in the same environment), that the FAA = 45 would be a suitable aggregate parameter.

4. Effect of Aggregate Gradation (Coarse vs. Fine) on IRI

It has been indicated that *coarse* mixes are at times more difficult to compact than *fine* mixes. Results of a study comparing the performance of the two types of mixes might, therefore, lead to differences in roughness, as measured by IRI with time.

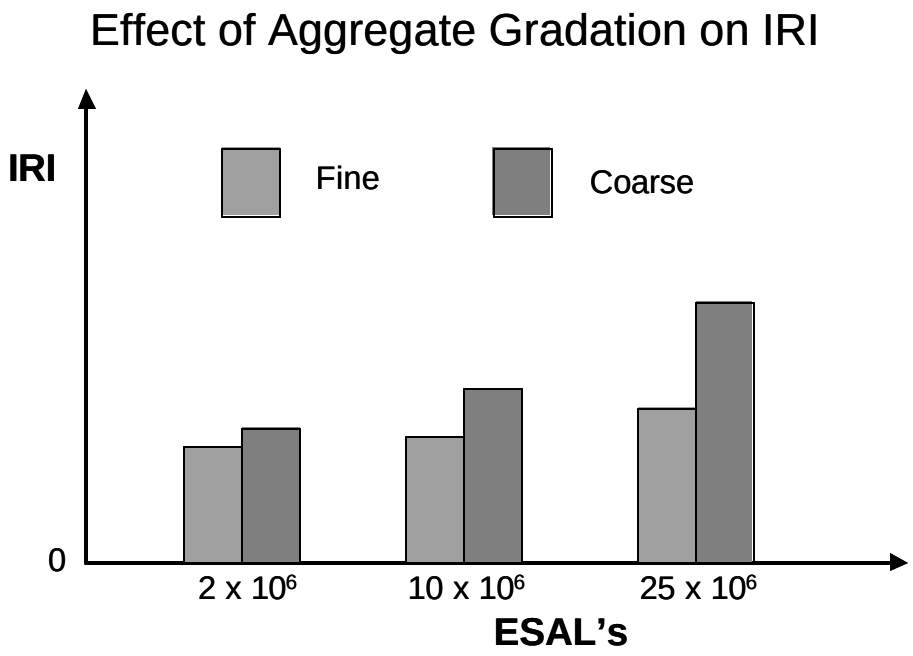


Figure 3.10 Effect of Aggregate Gradation on IRI

Figure 3.10 might suggest that small differences in initial smoothness could lead to larger differences with increased traffic, hereby necessitating more stringent controls on smoothness the heavier the anticipated truck traffic, regardless of mix type used.

The above four examples illustrate how the linking of the PMS and materials databases can be used to effectively enhance the SHRP developed Superpave mix technology – which includes the binder specification, aggregate requirements, volumetric mix design and accelerated performance tests of some type.

APPENDIX B – WEBSITE OF UW AND WSDOT WITH SUPERPAVE INFORMATION

B-1 Introduction

During the First half of 2001 the University of Washington (UW), Department of Civil and Environmental Engineering, in close cooperation with the Pavement Management Section of the Washington State Department of Transportation (WSDOT), developed an online "Hot Mix Database" to monitor data on mix design, field construction and performance, as well as archival of such data in a data warehouse. The website was initially used for data from about 30 Superpave projects carried out between 1997 and the end of 2000. This website can be found at: http://hotmix.ce.washington.edu . Since then a wider group of interested agencies agreed to cooperate in this field. This group consists of NCAT and the DOTs of Washington State, California, Missouri and Texas. More recently Maryland SHA joined this group and provided data on several SMA Superpave projects for the website. The latest additions were the seven Superpave projects selected for the current project, and for these additions several new data entry fields were added. The new website is http://hotmix.ce.washington.edu/hma/
A recent paper about this development is "Online Tools for Hot-Mix Asphalt Monitoring" by George White et al [White 02].

B-2 Capabilities

George White provided the following overview of capabilities:

- Provides a data warehouse for data associated with a hot-mix paving project. The site is capable of data entry, storage, viewing, and analysis.
- You can search through the contracts in the database by specific parameters.
- You can display all of the data associated with a contract from start to finish.
- Graphing functions and summary tables are available on the contract level and on the database level.
- Static GIS map location - this is the spatial map to show the location of each project.
- Data export to Excel is available on a contract level. This allows the user to manipulate the actual data in ways the site could not anticipate.
- User groups/profiles - login and password will allow the user unique access to add/edit contracts and data specific to their group.
- The mapping features are dynamic, allowing for zoom and pan, as well as layer and parameter selection.
- More comprehensive search functions (by specific mix/field parameters) will be available soon.
- Image collection - provide visual images taken during construction, provide infrared images, as in the SPTC Imagebase, and any performance related images.
- Currently data from MD, MI, TX and WA.

Of Particular Interest:

- Most data can be imported into the database with minimal hand data entry if it is already in an electronic form. All of the data currently in the database was brought in from Excel spreadsheets or Access files, at a minimal amount of effort.
- Mapping of other states will NOT be a problem, since the program relies on GIS data (Most of which is available).
- The method in which the data is presented is flexible. This will be determined by each states needs, and current practices.
- Analysis methods will become increasingly effective as more data is entered in. This may include some multi-dimensional data mining.
- Both Internet Explorer and Netscape are supported (newer versions), however Netscape is considerably slower in the presentation of the data.

Some other interesting features are:

1. Data are available to all users as soon as they are entered. When the proper equipment for electronic data entry is used in the field, it is possible to monitor construction projects instantly with "real time" QC/QA data which could be very beneficial.
2. It is possible to do a network analysis based on project data that historically were not available at the network level (e.g. the statewide effects of segregation in certain mixes), or a project analysis using data from projects in other states that in the past were not available at the project level (e.g. use of certain mix additives applied for special conditions only). The usual boundaries between project and network levels are disappearing, but more importantly, there are many more data available to draw statistically correct conclusions.

B-3 Examples

The following Figures 3.11 – 3.16 represent some selected images of the first website (http://hotmix.ce.washington.edu), and Figures 3.17 – 3.31 are examples from the seven Maryland projects on the latest website (http://hotmix.ce.washington.edu/hma/).

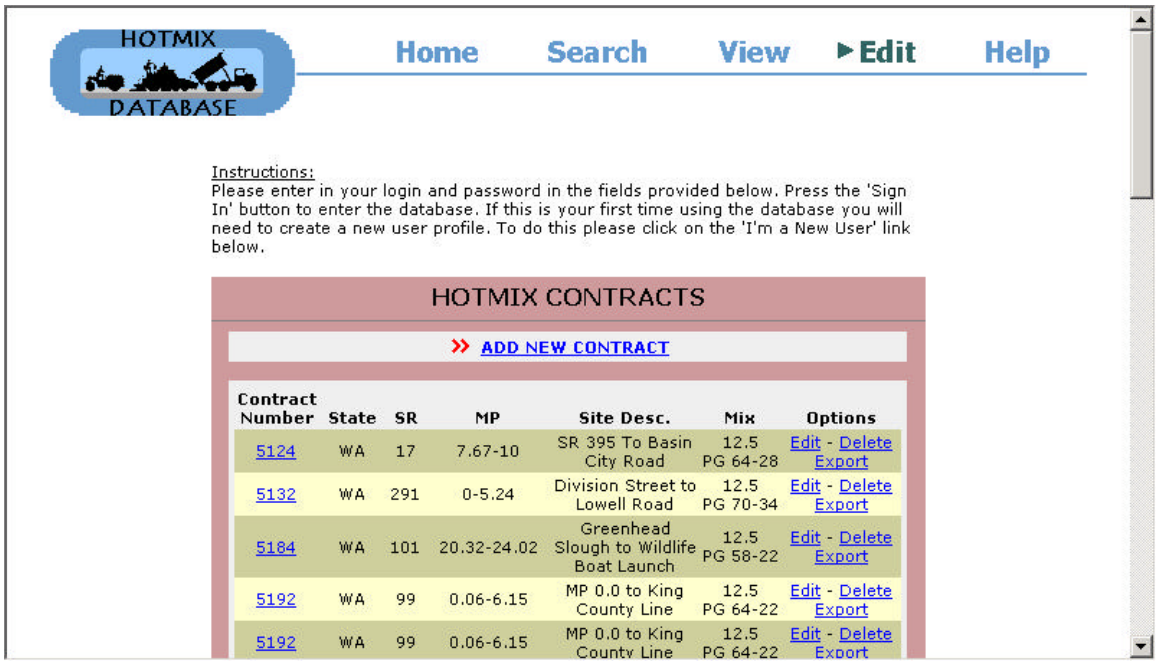


Figure 3.11 List with about 40 SuperPave Projects Constructed in WA since 1997. The Data for each Project can be exported to an Excel File from here

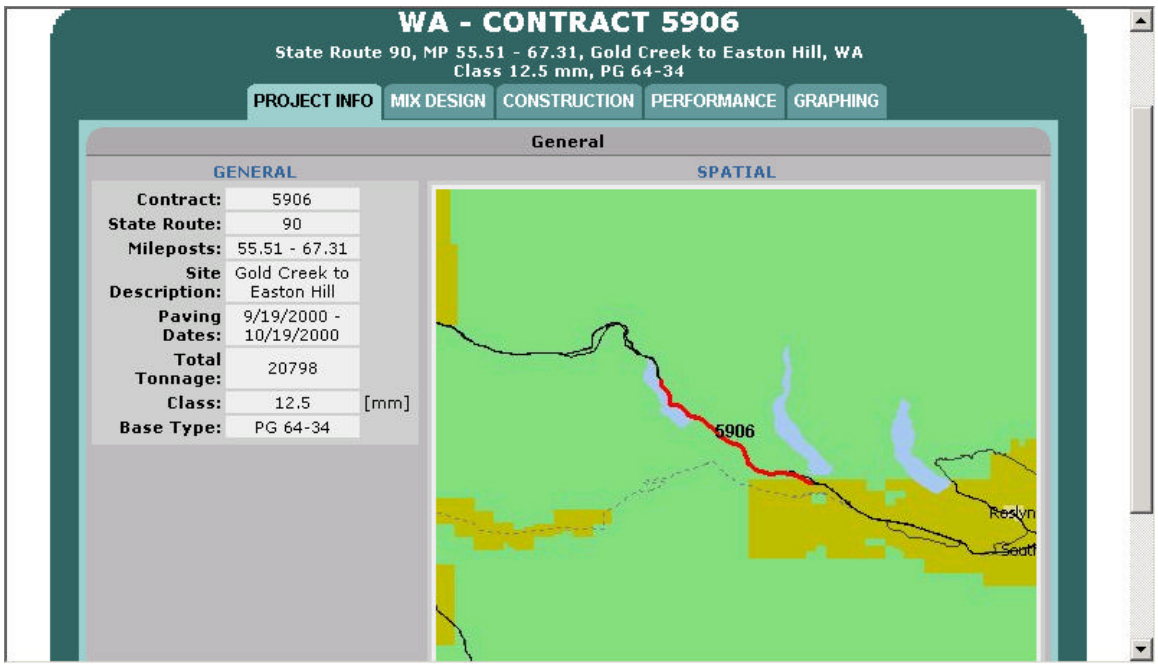


Figure 3.12 General Project Information on Location and Mix Type

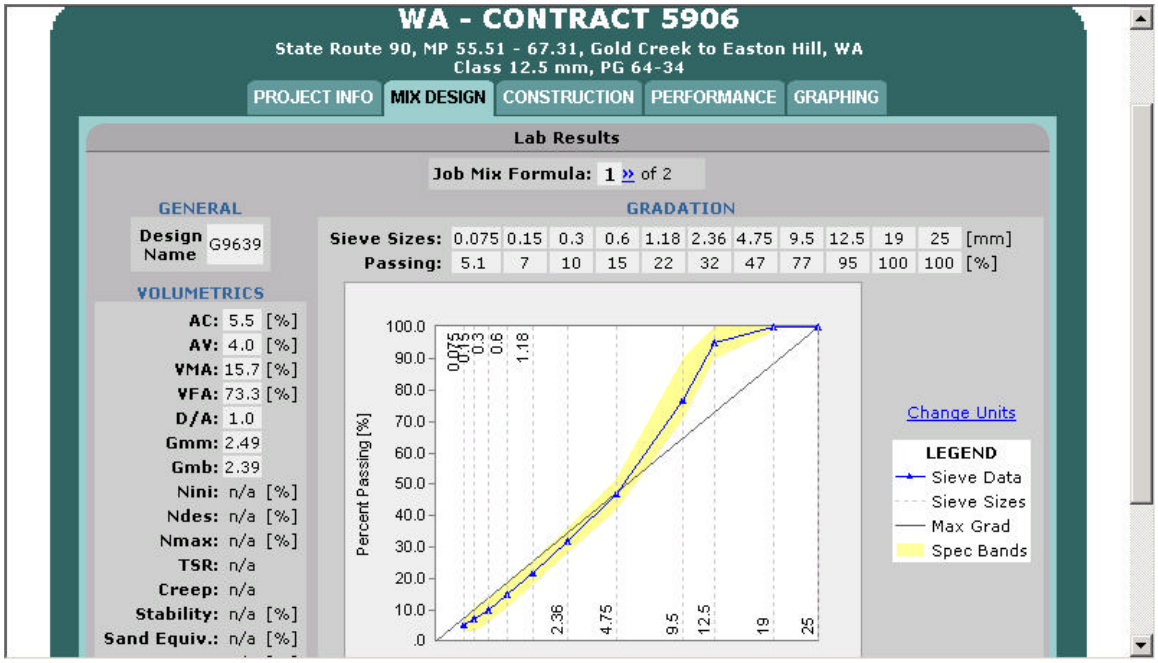


Figure 3.13 Information on Superpave Mix Design (Metric or English Units)

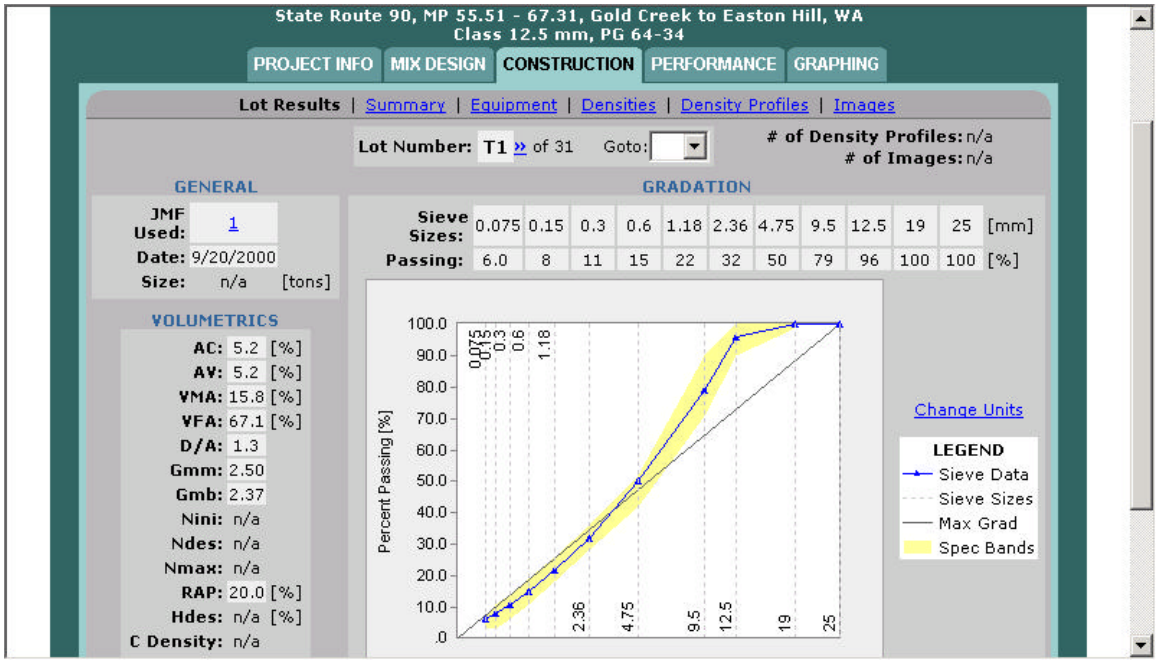
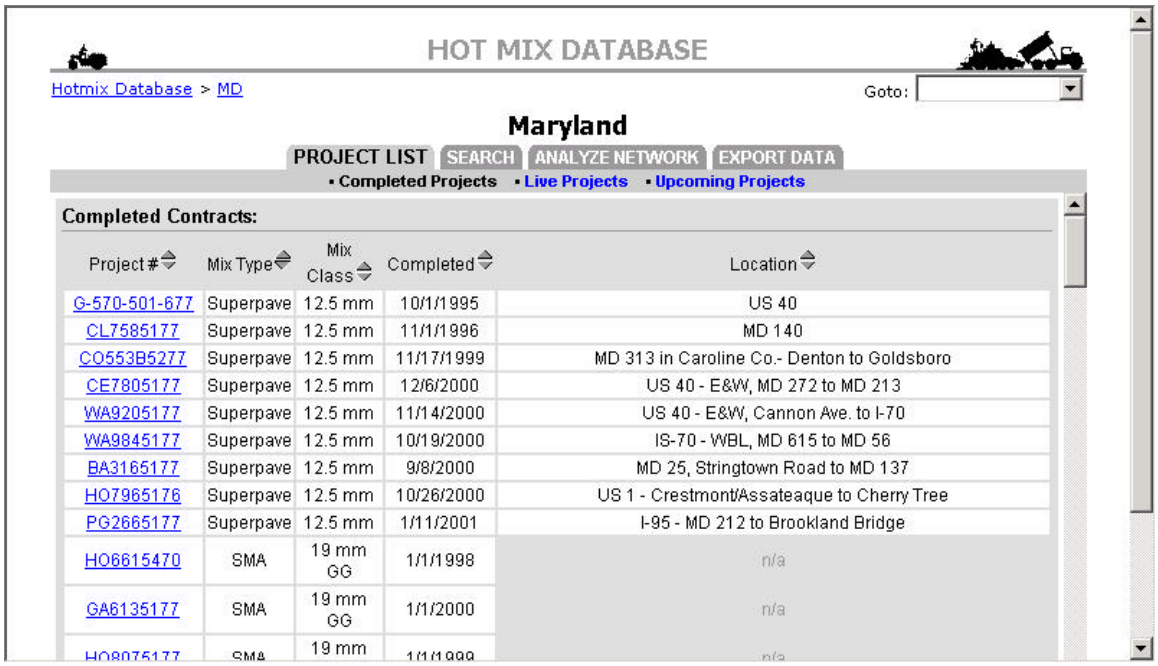


Figure 3.14 In-Place Properties of Superpave Mix

Figure 3.15 Performance Data in Years Prior to Superpave Overlay (the data for 2000 were not available yet)



HOT MIX DATABASE

Hotmix Database > MD

Goto:

Maryland

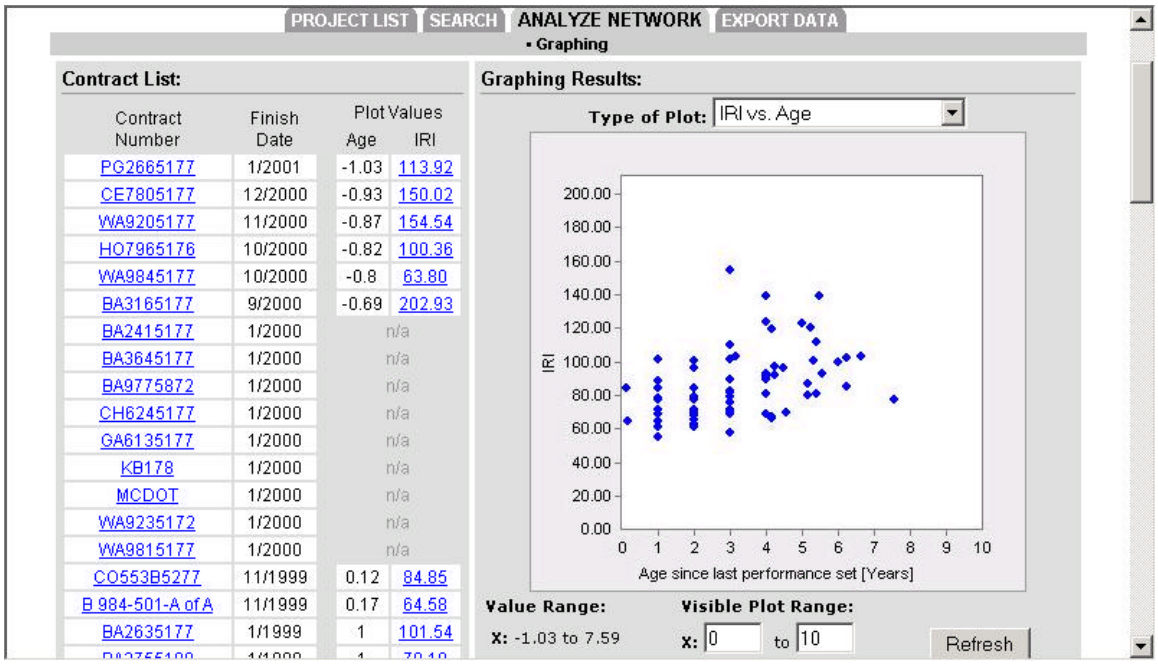
PROJECT LIST | SEARCH | ANALYZE NETWORK | EXPORT DATA

• Completed Projects • Live Projects • Upcoming Projects

Completed Contracts:

Project #	Mix Type	Mix Class	Completed	Location
G-570-501-677	Superpave	12.5 mm	10/1/1995	US 40
CL7585177	Superpave	12.5 mm	11/1/1996	MD 140
CO553B5277	Superpave	12.5 mm	11/17/1999	MD 313 in Caroline Co.- Denton to Goldsboro
CE7805177	Superpave	12.5 mm	12/6/2000	US 40 - E&W, MD 272 to MD 213
WA9205177	Superpave	12.5 mm	11/14/2000	US 40 - E&W, Cannon Ave. to I-70
WA9845177	Superpave	12.5 mm	10/19/2000	IS-70 - WBL, MD 615 to MD 56
BA3165177	Superpave	12.5 mm	9/8/2000	MD 25, Stringtown Road to MD 137
HO7965176	Superpave	12.5 mm	10/26/2000	US 1 - Crestmont/Assateaque to Cherry Tree
PG2665177	Superpave	12.5 mm	1/11/2001	I-95 - MD 212 to Brookland Bridge
HO6615470	SMA	19 mm GG	1/1/1998	n/a
GA6135177	SMA	19 mm GG	1/1/2000	n/a
HO8075177	SMA	19 mm	1/1/1999	n/a

Figure 3.16 Maryland Project List with Superpave and SMA-Superpave Projects. The Seven Projects Starting with Project CO55385277 were the Subject of the Pathfinder Study



PROJECT LIST | SEARCH | ANALYZE NETWORK | EXPORT DATA

• Graphing

Contract List:

Contract Number	Finish Date	Plot Values Age	Plot Values IRI
PG2665177	1/2001	-1.03	113.92
CE7805177	12/2000	-0.93	150.02
WA9205177	11/2000	-0.87	154.54
HO7965176	10/2000	-0.82	100.36
WA9845177	10/2000	-0.8	63.80
BA3165177	9/2000	-0.69	202.93
BA2415177	1/2000	n/a	
BA3645177	1/2000	n/a	
BA9775872	1/2000	n/a	
CH6245177	1/2000	n/a	
GA6135177	1/2000	n/a	
KB178	1/2000	n/a	
MCDOT	1/2000	n/a	
WA9235172	1/2000	n/a	
WA9815177	1/2000	n/a	
CO553B5277	11/1999	0.12	84.85
B 984-501-A of A	11/1999	0.17	64.58
BA2635177	1/1999	1	101.54
BA2755188	1/1999	1	79.19

Graphing Results:

Type of Plot: IRI vs. Age

Value Range: X: -1.03 to 7.59

Visible Plot Range: X: 0 to 10

Refresh

Figure 3.17 Network Analysis for all Listed MD Superpave Projects (including SMA Projects): Average IRI per Project versus Age. Similar Plots are Available for Rutting and Friction. Cracking Data were not yet Available in Electronic Format

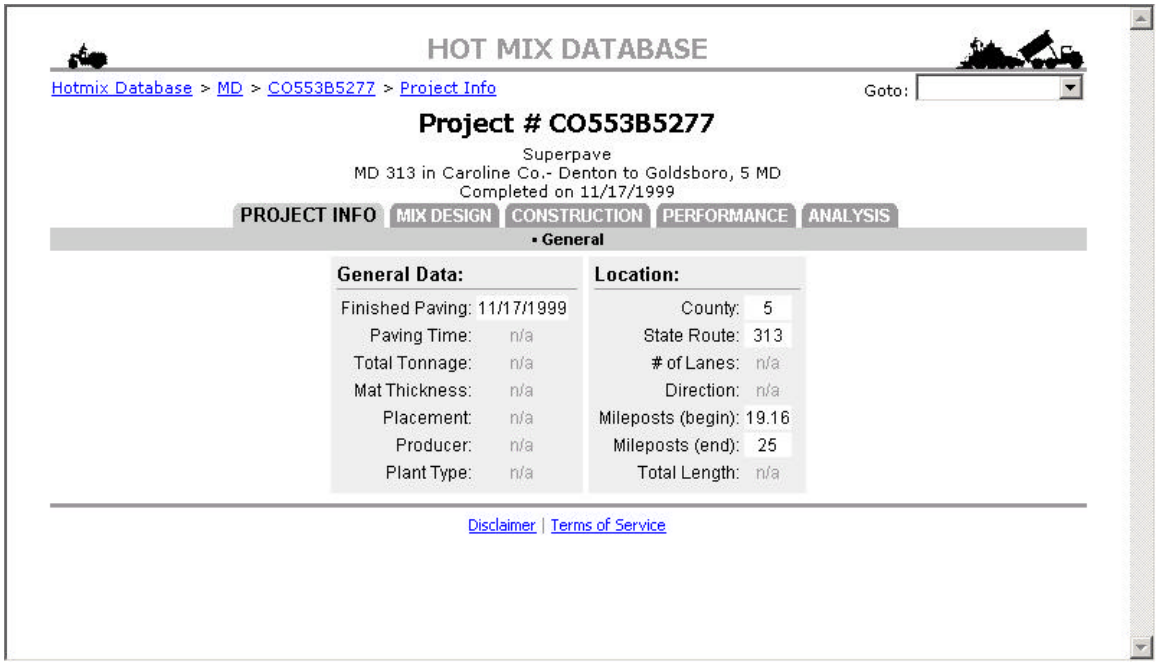


Figure 3.18 Summary of General Project Information with Desired Fields. Not all Data were yet Available in Electronic Format

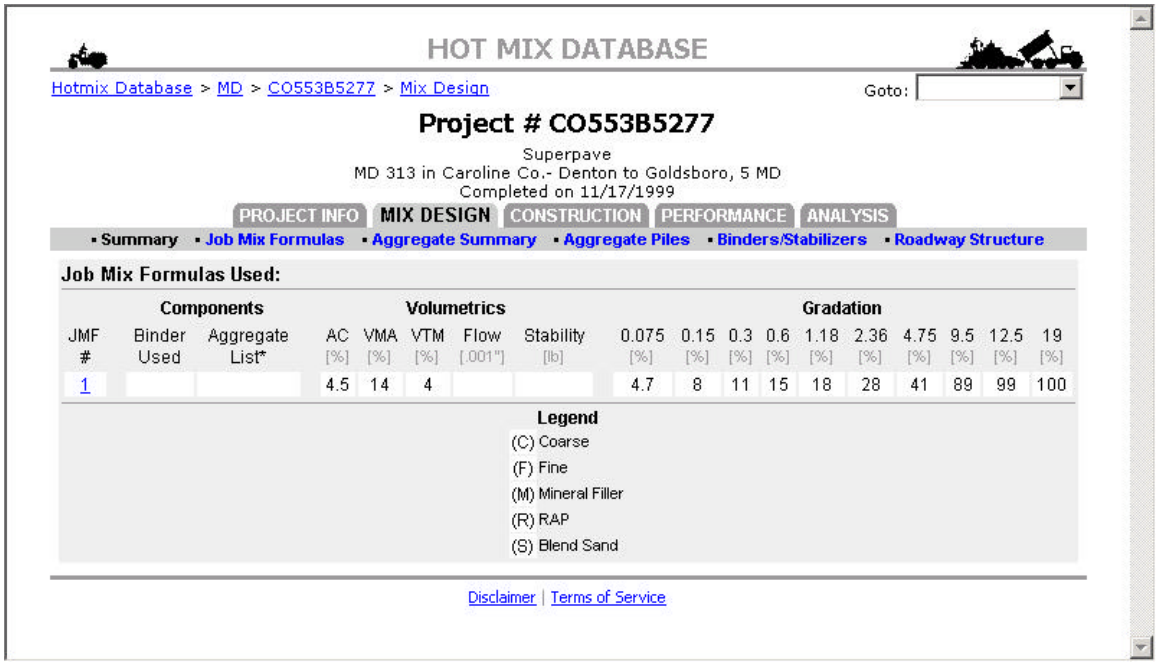


Figure 3.19 Summary of Mix Design Data for One Job Mix Formula. Information for some Data Fields is not yet Available Electronically

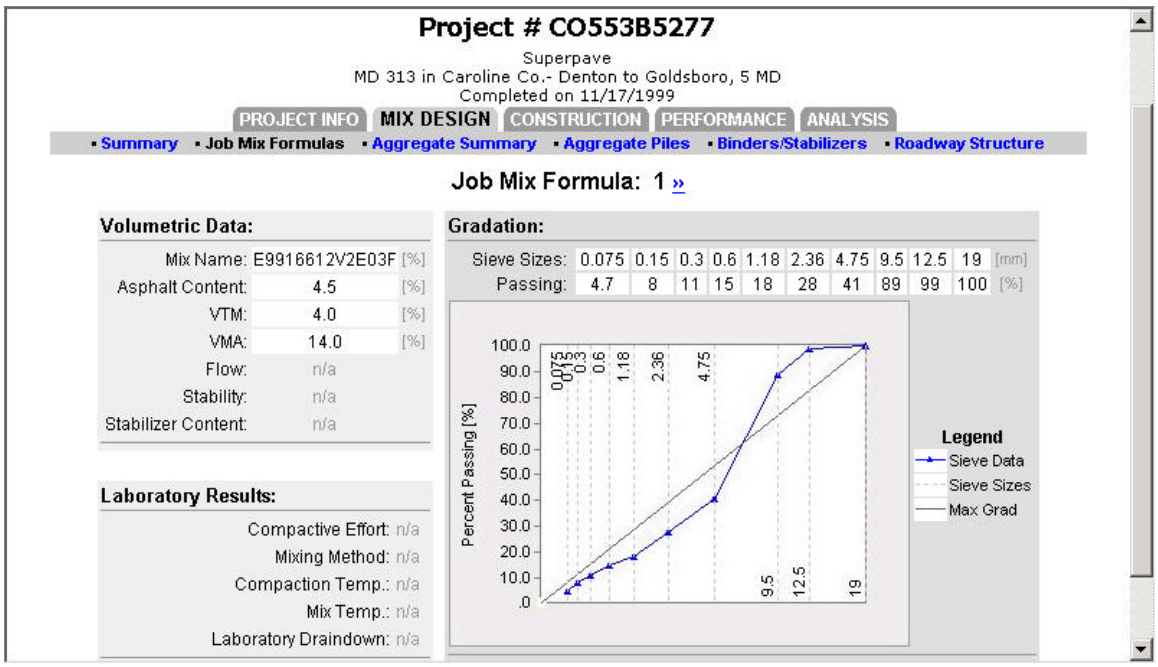
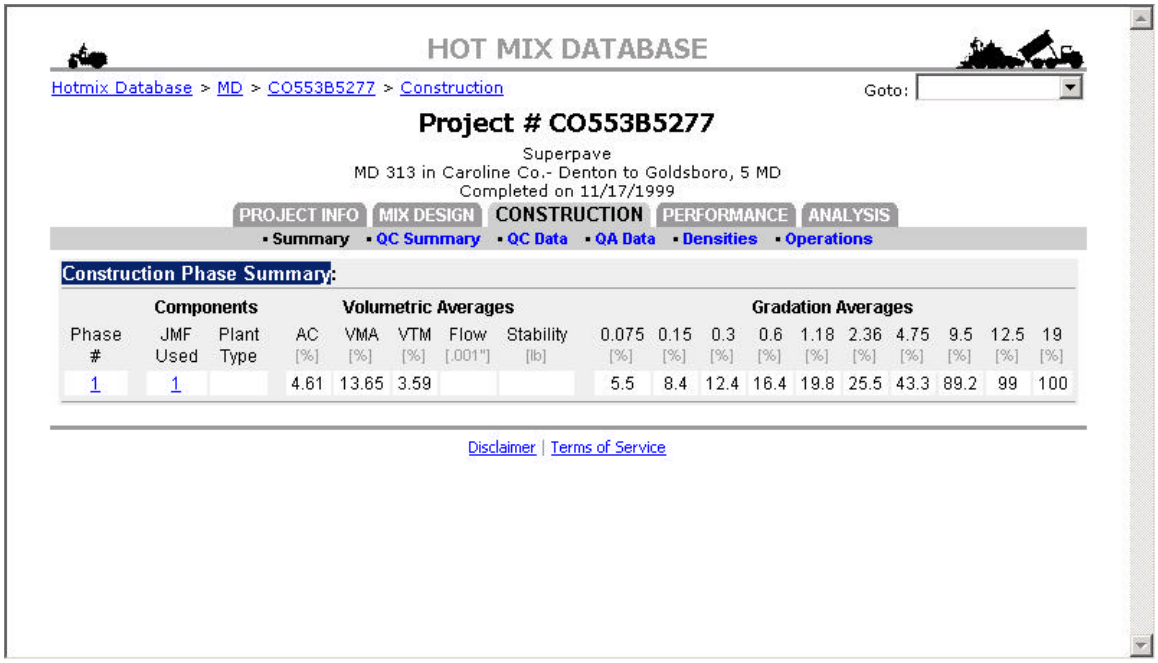


Figure 3.20 Data for one of the Job Mix Formula including a Sieve Curve. Some of the Volumetric Data and Laboratory Data are still Missing



Components			Volumetric Averages					Gradation Averages									
Phase #	JMF Used	Plant Type	AC [%]	VMA [%]	VTM [%]	Flow [.001"]	Stability [lb]	0.075 [%]	0.15 [%]	0.3 [%]	0.6 [%]	1.18 [%]	2.36 [%]	4.75 [%]	9.5 [%]	12.5 [%]	19 [%]
1	1		4.61	13.65	3.59			5.5	8.4	12.4	16.4	19.8	25.5	43.3	89.2	99	100

Figure 3.21 Summary of Construction Data for one Job Mix Formula with Room for Additional Information

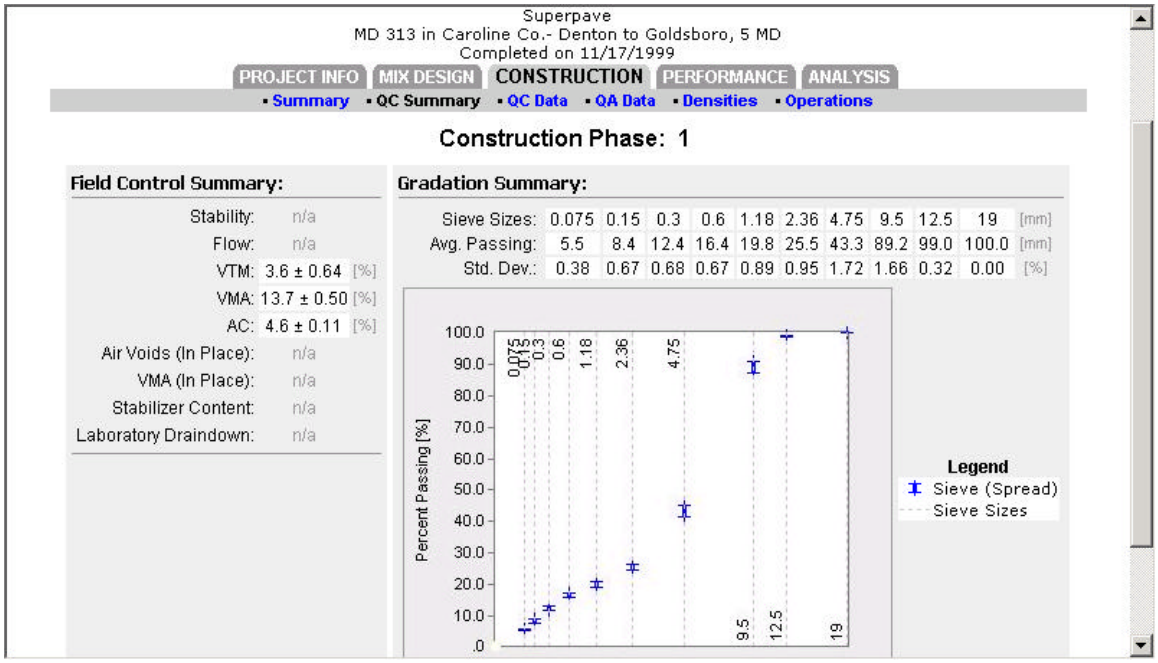


Figure 3.22 Summary of Quality Control Information from the Field including the Amounts Passing the Standard Sieve Sizes and the Standard Deviations

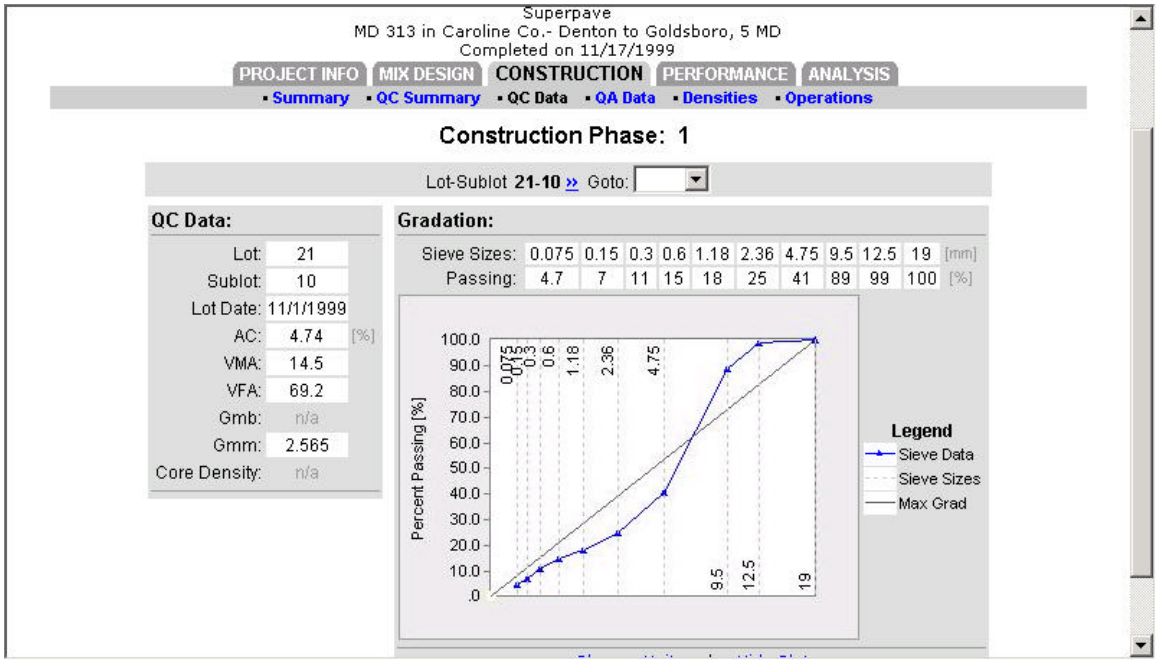


Figure 3.23 Quality Control Data for one Sub Lot of the Project including a Sieve Curve. Some Data were not yet Available Electronically

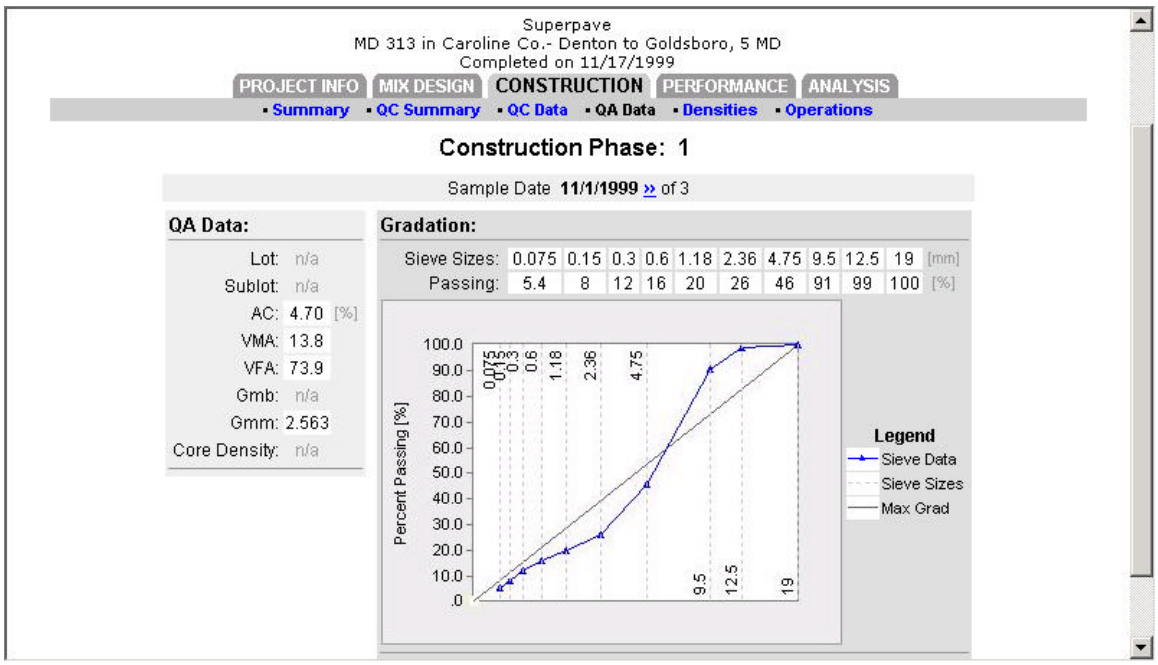
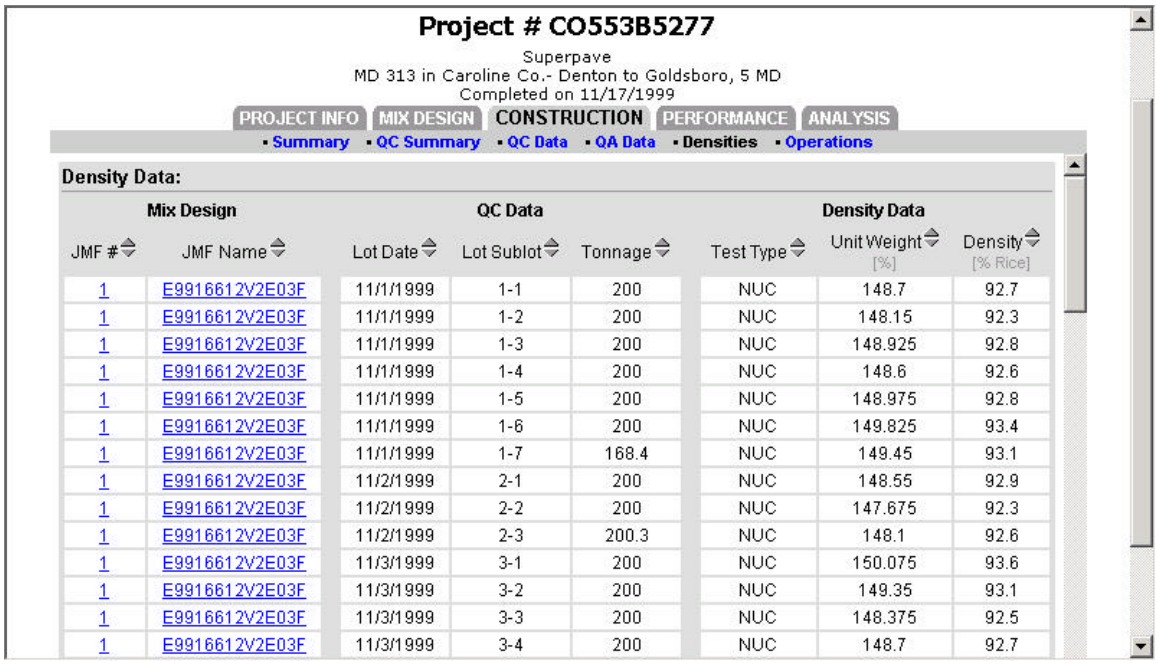


Figure 3.24 Quality Assurance Data taken at the Date Indicated. Several Data are not yet Available Electronically



Mix Design		QC Data			Density Data		
JMF #	JMF Name	Lot Date	Lot Sublot	Tonnage	Test Type	Unit Weight [%]	Density [% Rice]
1	E9916612V2E03F	11/1/1999	1-1	200	NUC	148.7	92.7
1	E9916612V2E03F	11/1/1999	1-2	200	NUC	148.15	92.3
1	E9916612V2E03F	11/1/1999	1-3	200	NUC	148.925	92.8
1	E9916612V2E03F	11/1/1999	1-4	200	NUC	148.6	92.6
1	E9916612V2E03F	11/1/1999	1-5	200	NUC	148.975	92.8
1	E9916612V2E03F	11/1/1999	1-6	200	NUC	149.825	93.4
1	E9916612V2E03F	11/1/1999	1-7	168.4	NUC	149.45	93.1
1	E9916612V2E03F	11/2/1999	2-1	200	NUC	148.55	92.9
1	E9916612V2E03F	11/2/1999	2-2	200	NUC	147.675	92.3
1	E9916612V2E03F	11/2/1999	2-3	200.3	NUC	148.1	92.6
1	E9916612V2E03F	11/3/1999	3-1	200	NUC	150.075	93.6
1	E9916612V2E03F	11/3/1999	3-2	200	NUC	149.35	93.1
1	E9916612V2E03F	11/3/1999	3-3	200	NUC	148.375	92.5
1	E9916612V2E03F	11/3/1999	3-4	200	NUC	148.7	92.7

Figure 3.25 Overview of Field Density Measurements for each Lot or Sub Lot of the Project together with Additional Information

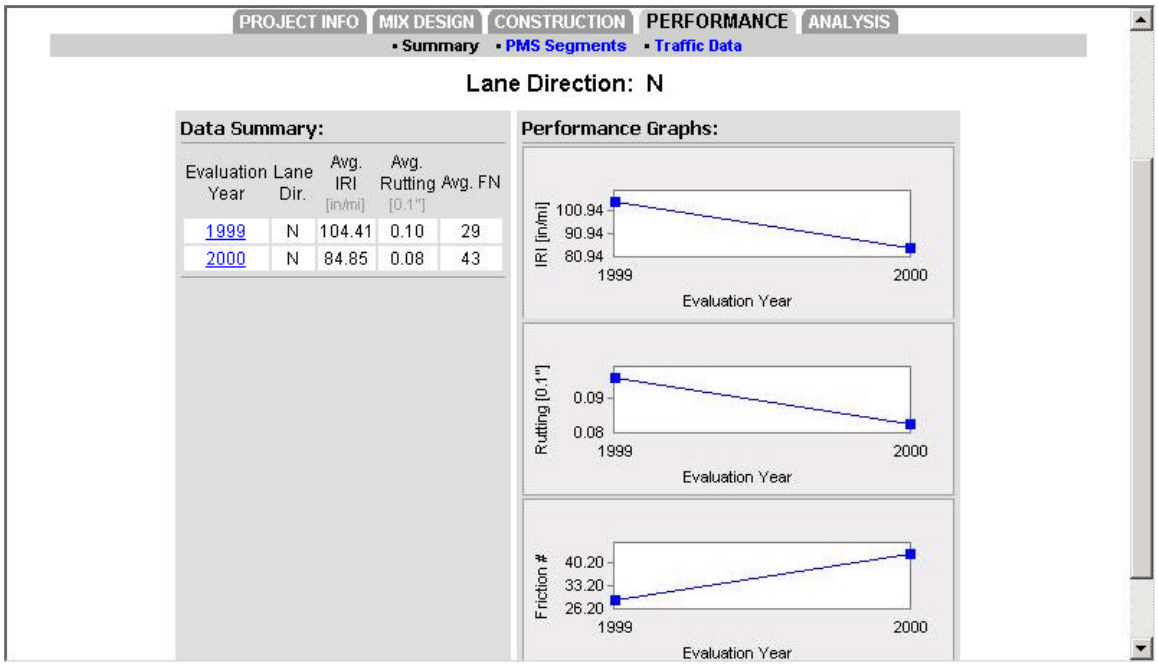


Figure 3.26 Average of Annual Performance Measurements for Ride, Rutting and Friction for one Project. In this case Performance Data were only available for Two Years

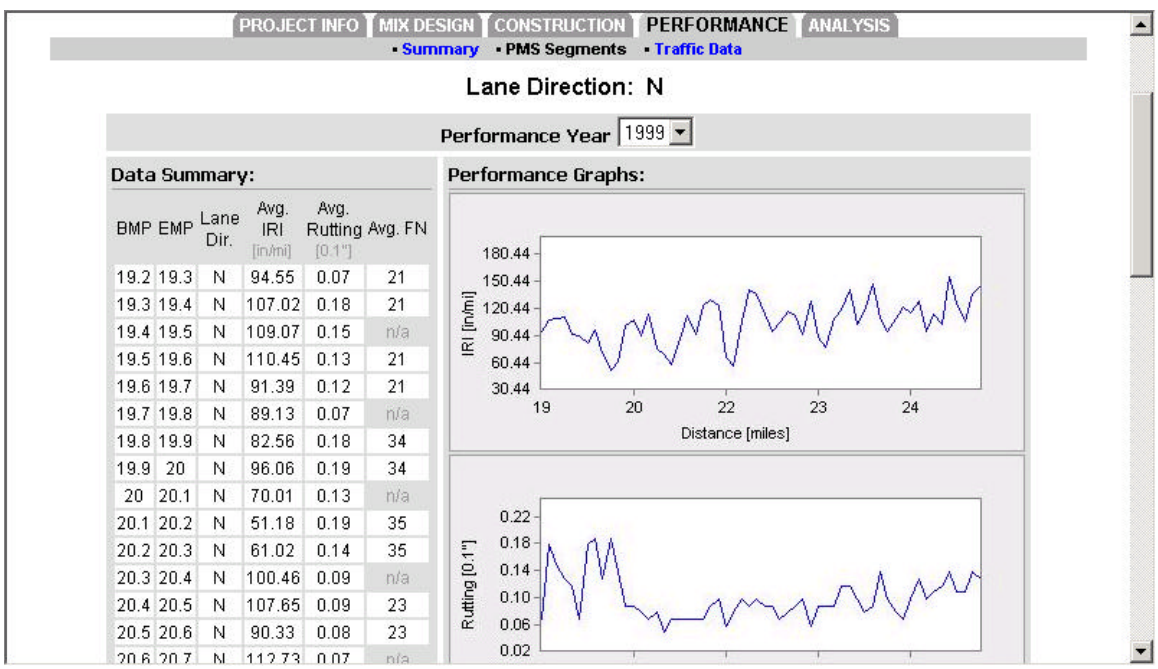


Figure 3.27 Project Performance Data (IRI, Rutting, Friction) per Segment, in this Case every 0.1 Mile, for one Particular Year

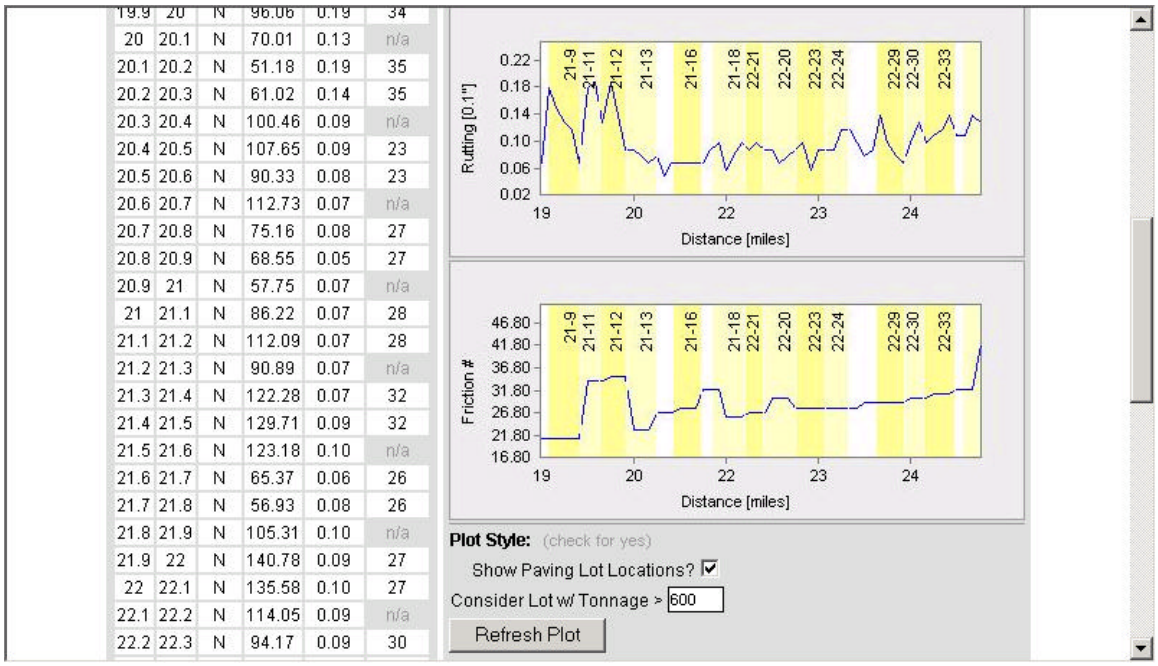


Figure 3.28 Same Plots as in Figure F-16 but with Superimposition of Paving Lot Location Numbers in this way the performance of each Individual Lot can be Monitored Easily. It is Possible to Check Lots with a Minimum Size only (in this case lots with more than 600 tons of asphalt mix have been considered only)

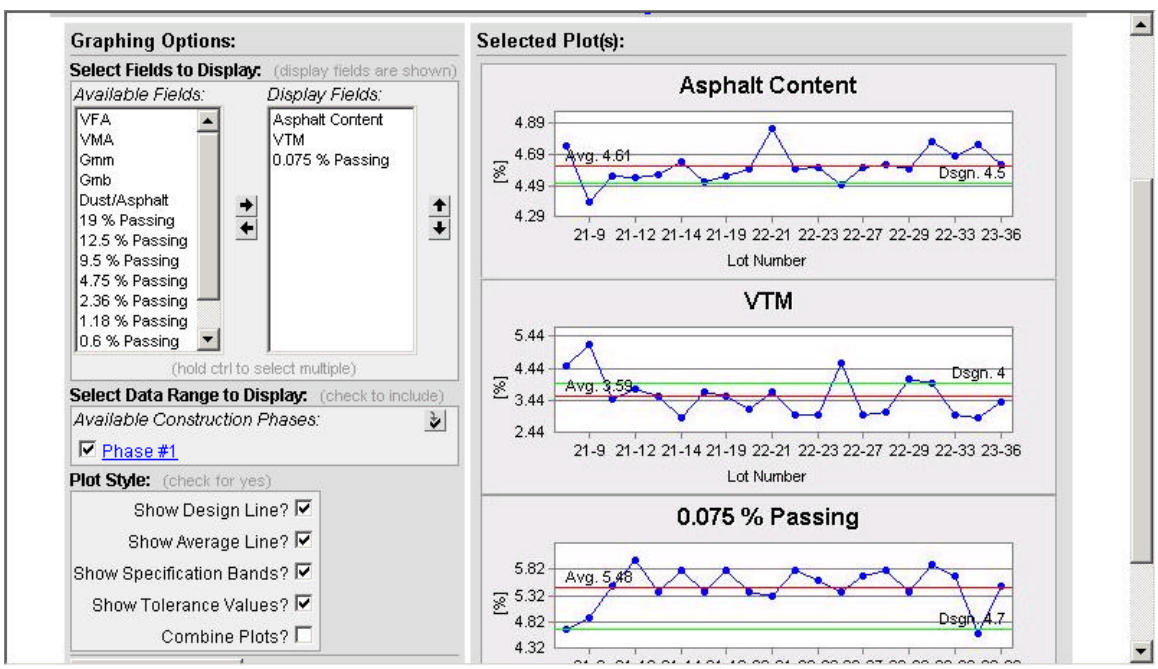


Figure 3.29 Graphing Options to Show Various Asphalt Mix Parameters (in this case Asphalt content, Voids in Total Mix, and Fines Passing 0.075 Sieve) for each Lot

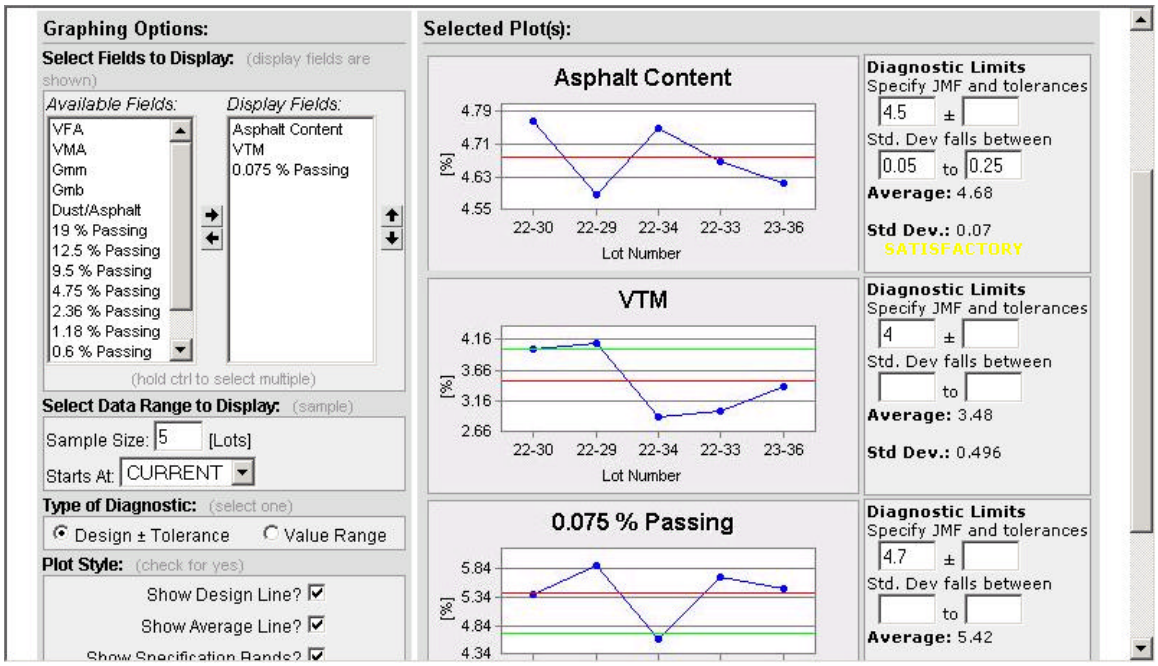
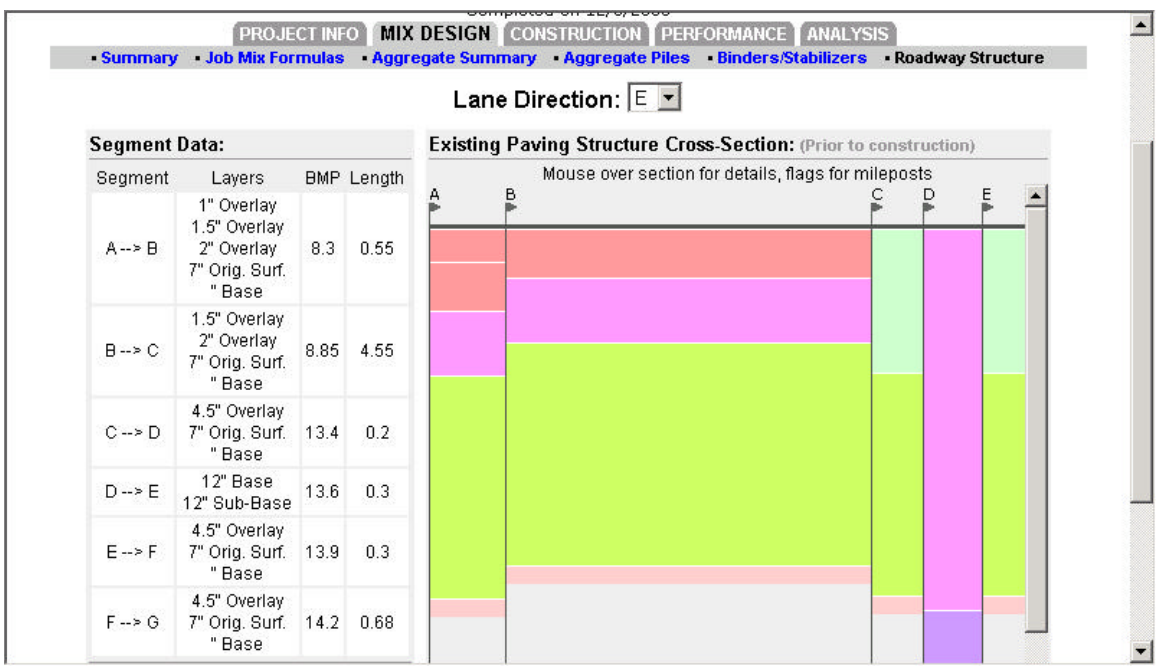


Figure 3.30 Diagnostic Statistical Analysis Possibilities for Various Samples (in this case the sample size is 5 lots) with Design Lines, Average Lines and Specification Bands for Certain Selected Parameters (in this case the same as selected in Figure F-19)



Segment	Layers	BMP	Length
A --> B	1" Overlay 1.5" Overlay 2" Overlay 7" Orig. Surf. " Base	8.3	0.55
B --> C	1.5" Overlay 2" Overlay 7" Orig. Surf. " Base	8.85	4.55
C --> D	4.5" Overlay 7" Orig. Surf. " Base	13.4	0.2
D --> E	12" Base 12" Sub-Base	13.6	0.3
E --> F	4.5" Overlay 7" Orig. Surf. " Base	13.9	0.3
F --> G	4.5" Overlay 7" Orig. Surf. " Base	14.2	0.68

Figure 3.31 **Pavement Cross Sections of the Pavement Prior to Treatment for a Selected Project**

4 PERFORMANCE AND PAVEMENT MODELING

4.1 INTRODUCTION

The main objective of this chapter is to provide an understanding of the applications of performance and design models, and, in the second part, to discuss the use of such models and to give examples from work carried out by DOT's in Pennsylvania, Washington State, and Wisconsin.

Performance models are mainly found at Network Level activities, their main purpose is to predict performance and deterioration of pavements as a function of time, and thus to predict pavement life. Various types of distress, such as roughness, rutting, cracking, etc, or indexes based on combinations of such distresses, can be used as input for these models.

Design models are mainly used at the Project Level for the design of new pavements or overlays for existing pavements. The main parameters influencing the design are properties of pavement materials, subgrade and existing pavement, traffic loadings, climate, and anticipated design life. In most cases an economic analysis is carried out, including an optimization on the basis of the expected life-cycle costs of the pavement structure.

4.2 CLARIFICATION OF PERFORMANCE AND DETERIORATION PREDICTION

The serviceability-performance concept, developed at the AASHO Road Test, has been a valuable and important part of pavement technology since the 1960's. Development of good models for predicting performance, in terms of PSI or RCI versus age or accumulated axle load applications, has been a major challenge for pavement engineers.

Despite the fact that performance has had a precise definition since the Carey-Irick development of the serviceability-performance concept [Carey 60], the term performance is currently used in a more general way by people in the pavement field. A major reason may be that it has a general meaning in everyday life. Consequently, it has become common practice among practitioners and researchers to use alternate terms such as deterioration or damage. For purposes of this course, the term performance will be used to generally describe the life-cycle behavior of a pavement in terms of deterioration of any or all of the condition parameters tracked in a PMS.

Pavement performance analysis is a primary form of engineering application of PMS data. It involves the analysis of a variety of pavement parameters, many of which are found in a PMS database, to determine the effects on pavement deterioration. Engineering analysis is performed to quantify these effects and develop mathematical models that predict pavement performance over time. Engineering attributes such as material properties, environmental conditions, traffic volumes and loads, design type, construction quality, and maintenance levels all contribute to pavement deterioration and thus must be considered in performance analysis.

4.3 PARAMETERS OR MEASURES TO BE PREDICTED, AND THE REQUIREMENTS

In order to estimate needs and timing for the sections in a pavement network, it is necessary to predict the rate of change of those measures for which criteria have been established. As well, it may be desirable to predict the rate of

change of some of the components of a measure, such as the cracking component of surface distress, to estimate maintenance requirements.

Figure 4.1 is an illustration of how deterioration prediction would be applied to an existing pavement section to estimate the rate of future deterioration, and the "needs year". As well, it illustrates the application of the deterioration model to rehabilitation alternatives applied in the needs year. The basic requirements for development of any prediction models have been described by [Darter 80] and they include the following:

1. An adequate database
2. Inclusion of all significant variables affecting deterioration
3. Careful selection of the functional form of the model to represent the physical, real-world situation
4. Criteria to assess the precision of the model

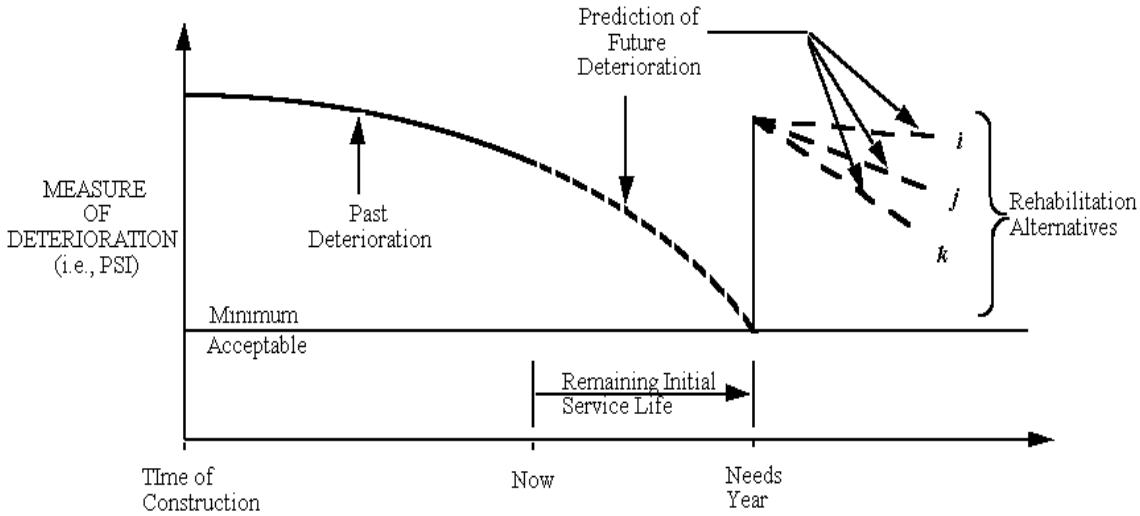


Figure 4.1 Illustration of how Deterioration Model is used to Predict Future Deterioration of an Existing Pavement, and Rehabilitation Alternatives Constructed in the "Needs Year".

4.4 BASIC TYPES OF PREDICTION MODELS AND EXAMPLES

A classification of prediction models has been suggested by [Mahoney 90], based on earlier work by [Lytton 87], as summarized in Table 4.1. It considers the network and project levels of pavement management and two basic types or classes of models: deterministic and probabilistic. These are further broken down into primary response, structural, functional and damage for the first type, and survivor curves and transition probabilities for the second type.

A convenient way of aggregating the breakdown of Table 4.1 into four basic types, for operational purposes, would be as follows:

1. Purely mechanistic, based on some primary response (behavior) parameter such as stress, strain or deflection.
2. Mechanistic - empirical, where a response parameter is related to measured structural or functional deterioration, such as distress or roughness, through regression equations.
3. Regression, where the dependent variable of observed or measured structural or functional deterioration is related to one or more independent variables like subgrade strength, axle load applications, pavement layer thicknesses and properties, environmental factors, and their interactions.

Subjective, where experience is "captured" in a formalized or structured way, using transition probability models, for example, to develop deterioration prediction models.

Table 4.1 Classification of Prediction Models [Mahoney 90]

	Types of Models						
	Deterministic				Probabilistic		
Levels of Pavement Management	Primary Response • Deflection • Stress • Strain • Etc.	Structural • Distress • Pavement • Condition	Functional • PSI • Safety • Etc.	Damage • Load Equiv.	Survivor Curves	Transition Process Models	
						Markov	Semi-Markov
National Network				•	•	•	•
State Network		•	•	•	•	•	•
District Network		•	•	•	•	•	•
Project	•	•	•	•			

4.4.1 Purely Mechanistic and Mechanistic – Empirical

The first type, purely mechanistic, has not yet been developed because pavement engineers do not use primary or fundamental response parameters as ends in themselves. Rather, they are only useful if they can be related to pavement distress, or to pavement properties used in other models such as for overlay design. Consequently, the mechanistic-empirical type of deterioration modeling approach has been developed. A good example is provided in [Queiroz 83] where linear elasticity was used as the basic constitutive relationship for pavement materials in a study of 63 flexible pavement test sections. Calculated responses included surface deflection, horizontal tensile stress,

strain and strain energy at the bottom of the asphalt layer, and vertical compressive stress and strain at the top of the subgrade. Various attempts were made to relate these responses to observed roughness and cracking through regression analysis. For example, the following predictive equation for roughness was developed (see Figure 4.2 for a plot of the equation):

$$\text{Log(QI)} = 1.297 + 9.22 \cdot 10^{-3}\ \text{AGE} + 9.08 \cdot 10^{-2}\ \text{ST} - 7.03 \cdot 10^{-2}\ \text{RH} + 5.57 \cdot 10^{-4}\ \text{SEN1 Log}N \qquad \text{Eq. 4.1}$$

where

QI	=	roughness (quarter car index, in counts/km)
Age	=	pavement age in years
ST	=	surface type dummy variable (0: as constructed, 1: overlayed)
RH	=	state of rehabilitation indicator (0: as constructed, 1: overlayed)
SEN1	=	strain energy at bottom of asphalt layer (10^{-4} kgf cm)
N	=	cumulative equivalent single axle loads (ESAL)
R^2	=	0.52 and standard error for the residuals = 0.11

Another predictive equation from [Queiroz 83] involved cracking, as follows (see also Figure 4.3):

$$\text{CR} = -8.70 + 0.258\ \text{HST} \cdot \text{Log N} + 1.006 \cdot 10^{-7}\ \text{HST1} \cdot \text{N} \qquad \text{Eq. 4.2}$$

where

CR	=	percent of pavement area cracked
HST	=	horizontal tensile stress at the bottom of the asphalt layer (kgf/sq cm)

The squared correlation coefficient (R^2) for equation 4.2 is 0.54 and the standard error for the residuals is 15.40.

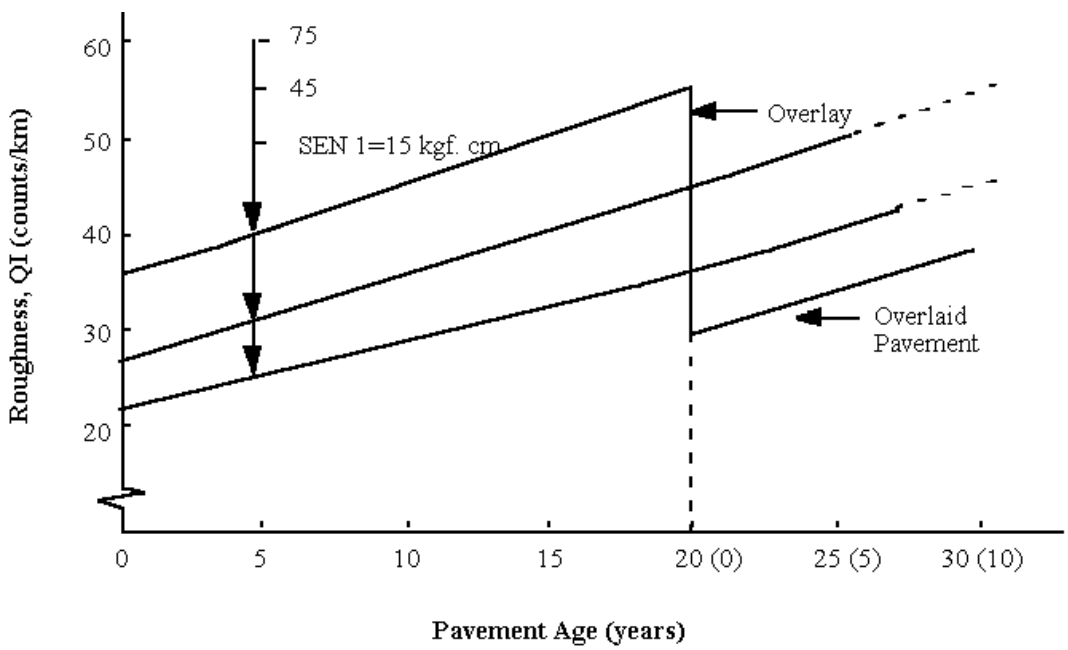


Figure 4.2 Roughness Prediction from Mechanistic – Empirical Model [Queiroz 83]

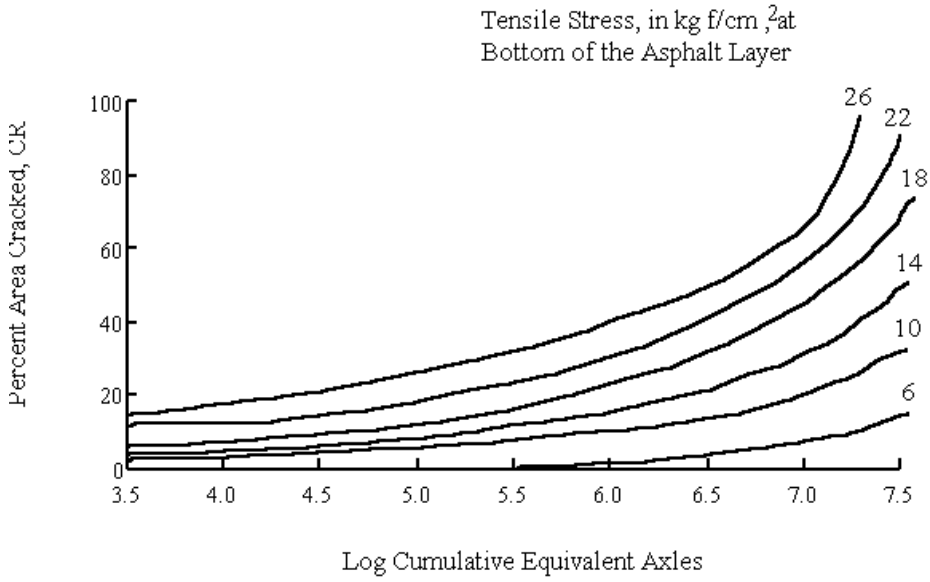


Figure 4.3 Cracking Prediction Regression Model [Queiroz 83]

4.4.2 Regression Modeling

The third type, direct regression, is particularly applicable where a long-term database has been acquired. For example, up to 25 years of data on roughness, surface distress, traffic, deflection and other factors were used in Alberta [Karan 83] to develop deterioration models, such as the following for conventional granular base pavements (see Figure 4.4 for a plot of the equation):

$$RCI = -5.998 + 6.870 \cdot LOG_e(RCI_B) - 0.162 \cdot LOG_e(AGE^2 + 1) + 0.185 \cdot AGE - 0.084 \cdot AGE \cdot LOG_e(RCI_B) - 0.093 \cdot \Delta AGE \qquad \text{Eq. 4.3}$$

where

RCI = Riding Comfort Index (scale of 0 to 10) at any AGE
RCI_B = previous RCI
AGE = age in years
ΔAGE = change in age from previous to current RCI measurement (four years for Equation 4.3, but it can be 1, 2, 3, 4 - etc. years)

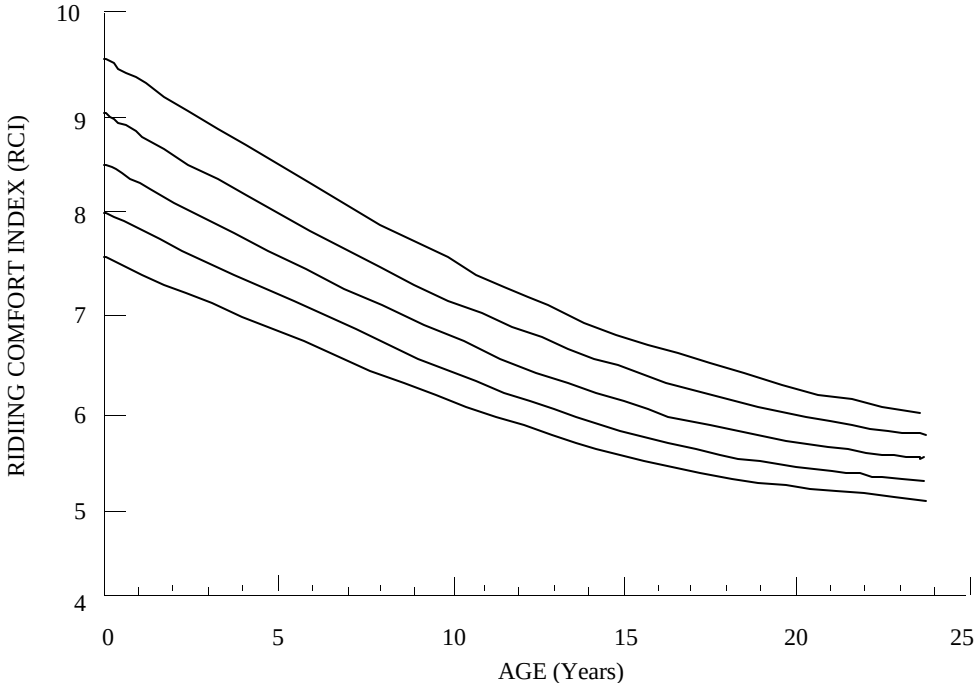


Figure 4.4 Performance from Equation 4.3

The squared correlation coefficient (R^2) for Equation 4.3 is 0.84 and the standard error of estimate is 0.38. While a number of variables were considered, such as traffic in terms of ESAL's, climate zone, subgrade soil type and others, only pavement age, Δ AGE and RCI_B were found to be significant variables in Equation 4.3, which is a recursive regression type of model. A possible reason is that the pavements were primarily designed in the first place for environmental deterioration, with structural sections significantly thicker than required by traffic alone.

Similarly, the state of Washington [Jackson 90] has developed a set of regression equations, based on a long-term pavement performance database, of the form:

$$PCR = C - mA^P \qquad \text{Eq. 4.4}$$

where

PCR = pavement condition rating, scale of 0 to 100
C = 100
m = slope coefficient
A = age of the pavement, years
P = constant which controls the shape of the curve

Table 4.2 provides an example listing of the standard or default performance curves, for Western Washington, using Equation 4.4 for different pavement designs or types.

Table 4.2 Standard Performance Equations of Equation 4.4 Type, in Washington State's Pavement Management System

Location	Type of Construction / Pavement Surfacing	Number of Analysis Units	Performance Equation	Age to PCR = 40
	New or Reconstructed/ Bituminous Surface Treatment	2	$PCR = 100 - 0.086\ (AGE)^{2.50}$	13.7

	New or Reconstructed/ Asphalt Concrete	26	PCR = 100 - 0.22 $(AGE)^{2.00}$	16.5
	New or Reconstructed/ Portland Cement Concrete	19	PCR = 100 - 0.85 $(AGE)^{1.25}$	30.1
	Resurfacing/BST over AC	5	PCR = 100 - 8.50 $(AGE)^{1.25}$	4.8
	Resurfacing/BST over BST	6	PCR = 100 - 3.42 $(AGE)^{1.50}$	6.8
Western Washington	Resurfacing/AC Overlay (under 1.2 inches)	75	PCR = 100 - 0.58 $(AGE)^{2.00}$	10.2
	Resurfacing/AC Overlay (1/2 inch to 2.4 inches)	126	PCR = 100 - 0.76 $(AGE)^{1.75}$	12.1
	Resurfacing/AC overlay (over 2.4 inches)	19	PCR = 100 - 0.54 $(AGE)^{1.75}$	14.8

Visual Modeler

A pooled fund study, carried out by TRDI, and sponsored by the FHWA and the Pennsylvania Department of Transportation, produced the Visual Modeler [TRDI 97] software system that allows pavement management engineers to produce three types of regression based performance models with five model forms each, in addition to extensive plotting and data review capabilities. The three methods are

1. Group or family based modeling,
2. Ad-Hoc modeling, and
3. Section-based modeling.

The data review capabilities allow the user to inspect the data in detail and correct any problems. He can then develop models using any of the three methods.

The production of pavement performance models requires accurate and reliable data. Capabilities in the Visual Modeler software allow users to inspect the data before models are produced. The first data inspection tool allows viewing historical condition data for a given section to see when rehabilitation activities were applied. The section data review screen is shown in Figure 4.5 below.

Users can also view the complete performance history of a section regardless of the last rehab year by selecting the section and clicking the section behavior command from the Action menu on the performance master file window. The screen shows the performance history of a section. The vertical line on the graph indicates a rehabilitation action.

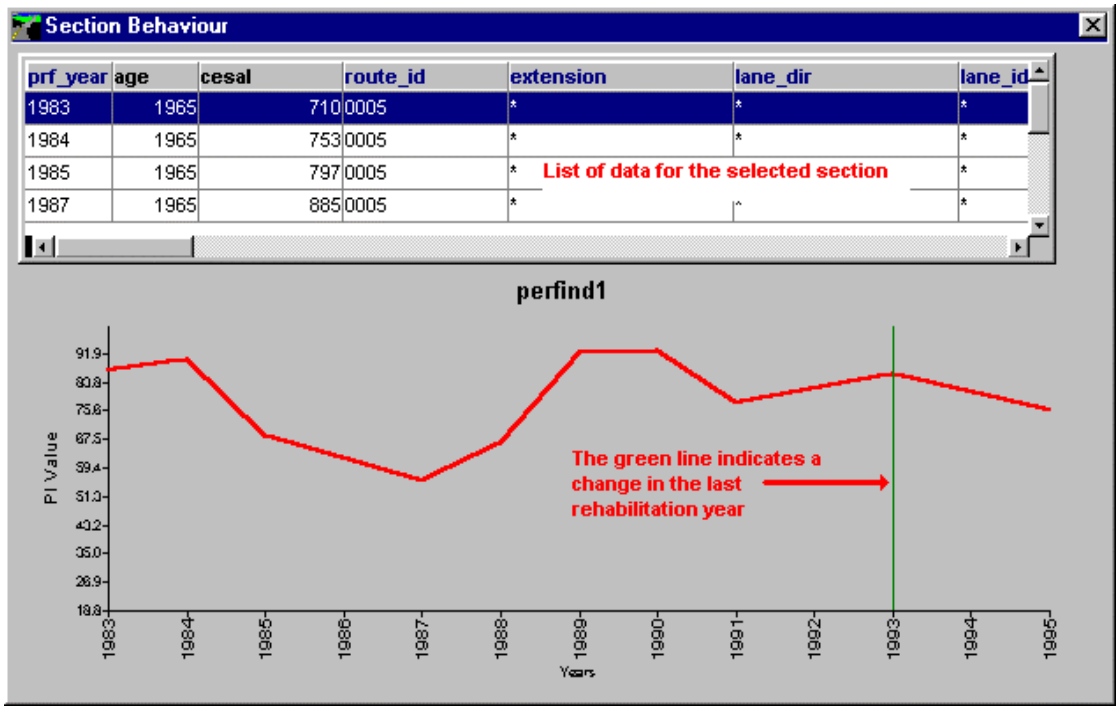


Figure 4.5 Section Data Review Window

After reviewing section data, users can begin to plot and produce models using any of the three available model-building methods. Group or family based modeling uses significant variables to partition the network into homogeneous performance groups or families. Then performance models are developed from regression equations on historical data within each pavement group. The groups typically are defined by such variables as Traffic level, functional class, subgrade type, and pavement structural type. These general models are then used to predict performance for each pavement in the group. Pennsylvania DOT has been using this process for creating models and examples of the grouping and model building are discussed in more detail in the examples in the second half of this Chapter.

The second type of modeling is ad hoc modeling. Ad hoc modeling uses simple queries of the database to produce models from user-selected sets of data. While similar to group modeling this method allows the engineer to define criteria for data set creation and then investigate the performance of specific data. Rather than using variables to create a set of groups for model building the ad hoc modeling process allows the user to create one group based on any data criteria. This type of modeling is useful in plotting performance histories for sections that meet a specific query. For example, users could retrieve data from the database that allows investigating specific problems or questions. An example might be, "How are the thin overlays in District 1, built in 1994, performing so far?" The ad hoc modeling module allows retrieving and plotting available data for thin overlays and produces regression equations that represent the pavement performance for those sections. Figure 4.6 shows the ad hoc model building window with a simple query. The query in the figure returns data in functional class numbers 1, 2, 3, and 4 (these are interstate and NHS highways in this database) and Road Structure Categories 1 and 2 that include thin overlays. Then the analyst can plot these data and review the pavement performance.

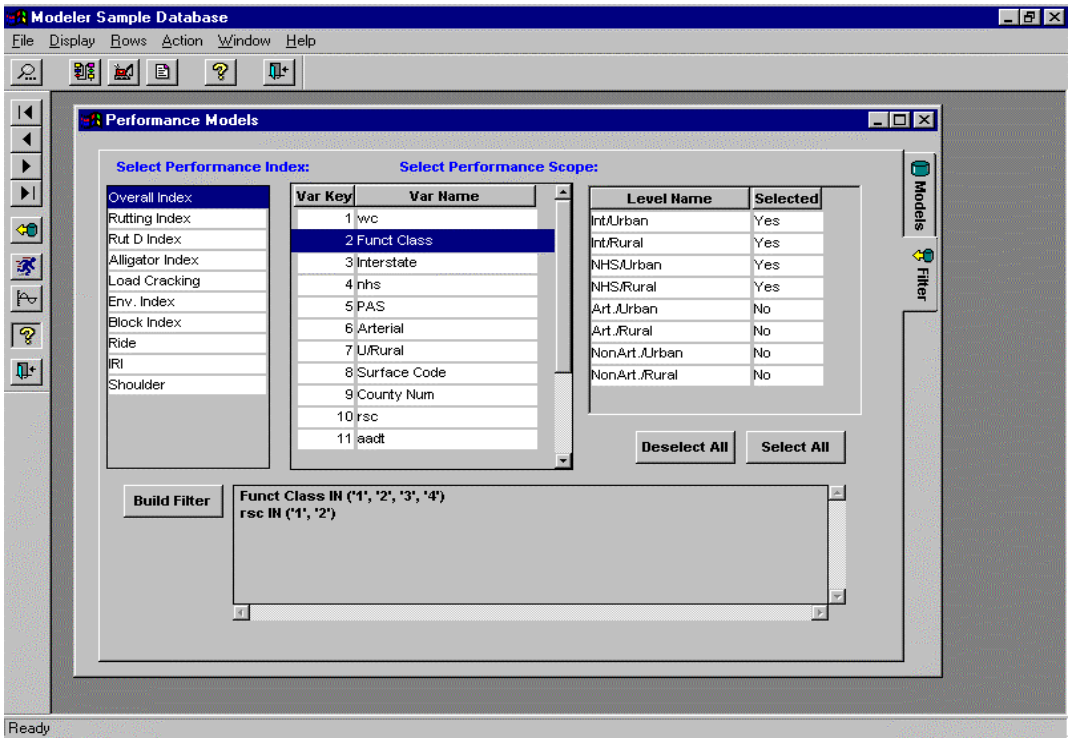


Figure 4.6 Ad Hoc Model Filter

Once users have created a data set for modeling regardless of the type of data set the model building and data review process is the same. Figure 4.7 shows a section based model plotted on the screen. The analyst can choose from four different model equations and plot the results of regression calculations on the screen. The Visual Modeler regression engine is the same core calculation components used in creating models for the Pennsylvania DOT. The model building process is the same regardless of the source of the data. Specific Examples of model building and regression calculations are given later in this Chapter. In addition, the analyst can also elect to view the actual data on the graph in a tabular format.

Users can carry out section based modeling by selecting the data (with the mouse) and viewing the raw data in a tabular format to help investigate performance of specific sections. Figure 4.8 shows the review data window.

Visual Modeler is a powerful performance data evaluation, review and modeling tool that can be used by any agency to create models and review their data.

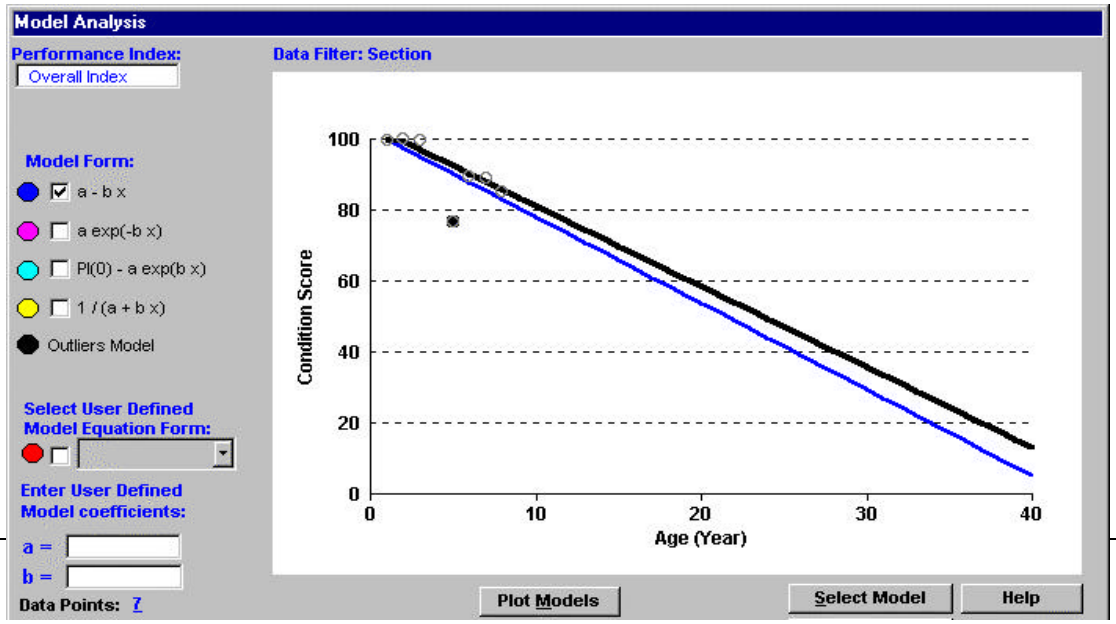


Figure 4.7 Section Based Model in Analysis Window

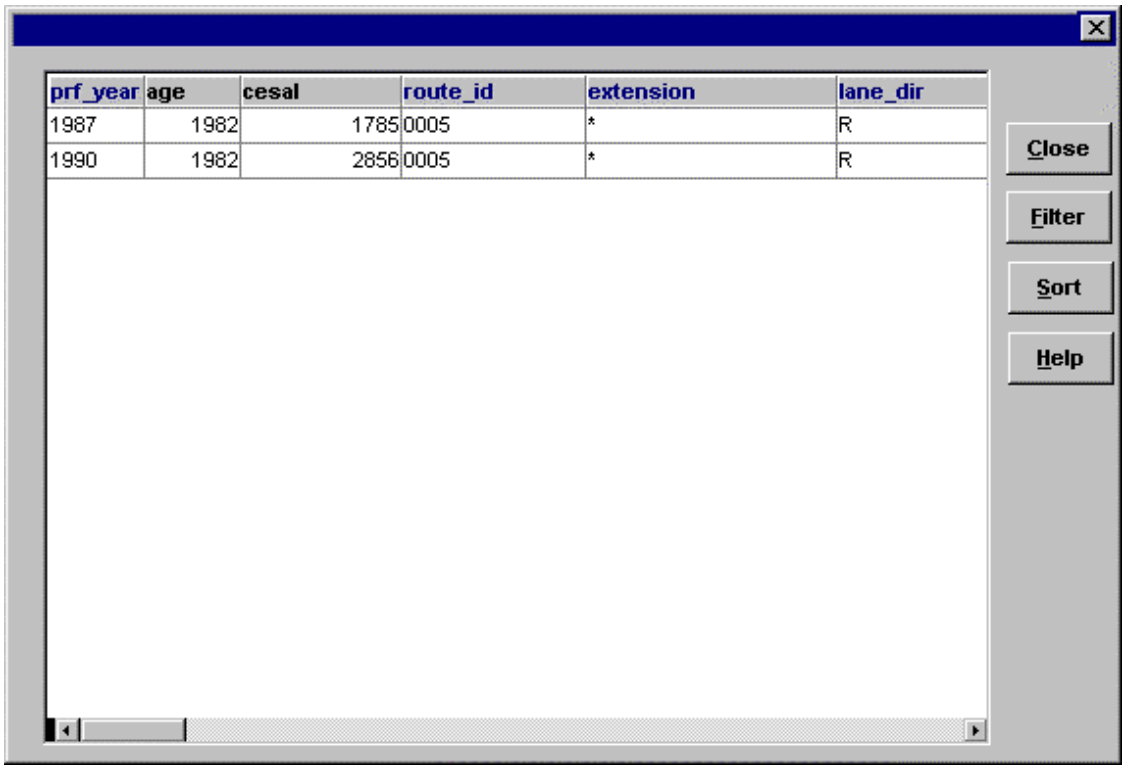


Figure 4.8 Review Section Data Window

4.4.3 Subjective Modeling

Capturing the subjective experience of local engineers and experts, the fourth basic approach, is an excellent way to create initial performance models for the development of a pavement management system. The one-step transition probability technique based on the Markov process is well suited for this purpose [Smith 74, Finn 74, Karan 76]. In the Markov process, the future state of the model element, e.g., a pavement is estimated solely from the current state of the element.

The state of the element is defined by condition measures, e.g., for pavements, condition measures include roughness or serviceability, pavement condition index or percent surface cracking, skid number, etc. For modeling purposes, the state is defined with respect to a range of the condition measures. Table 4.3 demonstrates that nine condition states would result from using two condition measures at three levels each. The numbers within the table represent the various condition states, for example condition state five corresponds to the condition state with medium cracking and medium roughness levels.

Table 4.3 Example Condition States for a Markov Process Model

Pavement Roughness	Surface Distress, percent area cracked		
	0-3	3-7	>7
0-40	1	4	7
41-90	2	5	8
>90	3	6	9

A transition probability matrix defines the probability that a pavement in an initial condition state will be in some future condition state. Table 4.4 is an example transition probability matrix for the nine condition states defined in Table 4.3. As shown in Table 4.4, the majority of pavements will stay in the same condition state from one year to the next, e.g., according to Table 4.4, a pavement in condition state 4 has a 92% probability of remaining in condition state 4 after one year.

One of the requirements of the transition probability matrix is the sum of each row must equal 1.0. Formal interview methods, such as the Delphi method, can be used for developing the transition probability matrices. The experts are asked to determine to the best of their ability the probability a pavement in one of the condition states will go to each of the future condition states in one time period. Analysis of a historical database to estimate the actual transitions that have occurred may also be used to estimate the probability values.

Table 4.4 Example Transition Probability Matrix for a Markov Process Model

Initial Condition State	Future Condition State								
	1	2	3	4	5	6	7	8	9
1	0.90	0.04	0.02	0.03	0.01	0	0	0	0
2	0.01	0.90	0.03	0	0.05	0.01	0	0	0
3	0	0.01	0.92	0	0.01	0.03	0	0.01	0.02
4	0	0	0	0.92	0.05	0.02	0	0.01	0
5	0	0	0	0.01	0.94	0.03	0.01	0.01	0
6	0	0	0	0	0.01	0.94	0	0.01	0.04
7	0	0	0	0	0.02	0	0.95	0.02	0.01
8	0	0	0	0	0	0	0.01	0.96	0.03
9	0	0	0	0	0	0.01	0	0.01	0.98

Because the Markov process estimates the future condition state solely from the current condition state, other factors that affect pavement behavior are handled by defining a transition matrix for each combination of factors that affect pavement performance, such as:

1. Pavement type
2. Pavement thickness
3. Traffic volumes or loads
4. Subgrade type or strength
5. Environmental and regional effects

For example, one application of the Markov process uses two levels for surface thickness, three levels of traffic, and two levels of subgrade strength for a total of 2 x 3 x 2 or 12 combinations [Turay 91]. Thus, 12 transition probability matrices are required for this application, which is limited to one pavement type and environment. By multiplication of the transition probabilities together for several time steps, performance curves can be developed as shown in Figure 4.9 [Turay 91].

There are several advantages to the Markov process model, including direct local calibration by capturing, in a formalized way, the experience of local engineers and the ability to develop performance curves without any historical database. In addition, after the pavement management system has been in place for several years it is possible to calibrate the models with field data [Wang 92]. Disadvantages of the approach include the need for developing a transition probability matrix for each combination of factors that affect the pavement performance. In addition, pavement history is difficult to include in the Markov model since the estimate of the future state of the pavement is based only on the current state. The Arizona Department of Transportation uses rate of crack change as one of the condition state factors, thereby allowing pavements with a rapid deterioration history to have a greater probability of a transitioning to a lesser condition state than pavements that do not have a history of rapid crack development.

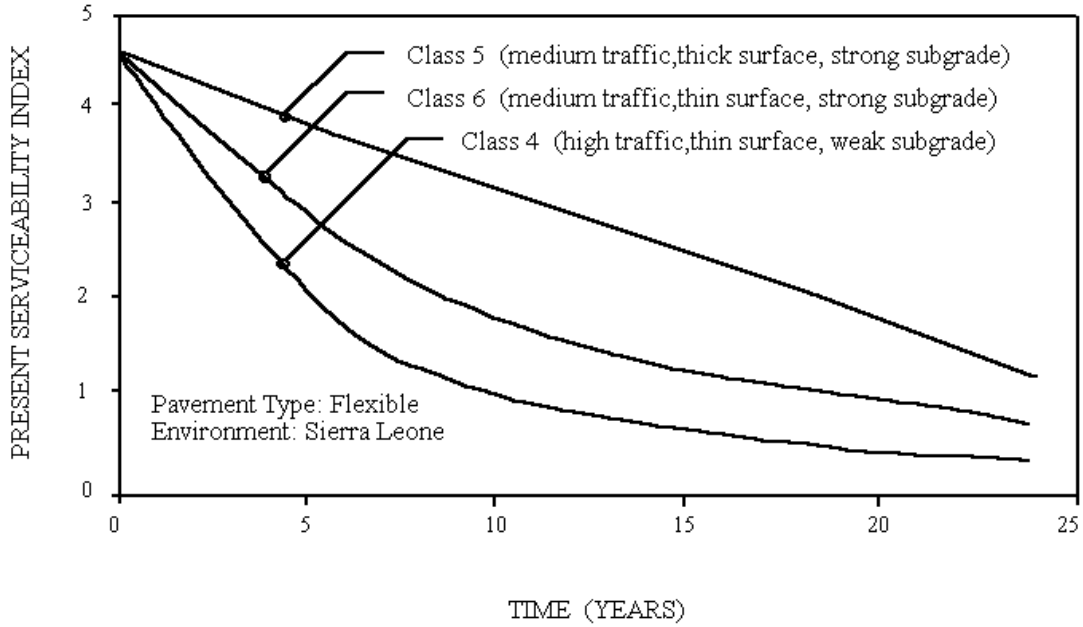


Figure 4.9 Example Performance Curves from Transition Probability Matrices [Turay 91]

4.5 PAVEMENT DESIGN FRAMEWORK AND COMPONENTS

In a pavement management system, the pavement design phase is an engineering application that requires several activities. These may be broadly classified as:

1. Information needs relating to inputs, objectives, constraints, etc.
2. The generation of alternative design strategies
3. The structural analysis, economic evaluation, and optimization of these strategies.

Figure 4.10 is a diagram of the major components of a project level pavement management system. The general nature of this diagram applies to both flexible and rigid pavements. Another version of the design phase [Hudson 73] is shown on Figure 4.11. Although the organization of this diagram is somewhat different from Figure 4.10 both contain the same basic activities and represent the same basic philosophy. Figure 4.11 is an updated version of Figure 4.1. The reader should compare these two figures to note the subtle refinements that were made to the project level pavement management framework as it evolved.

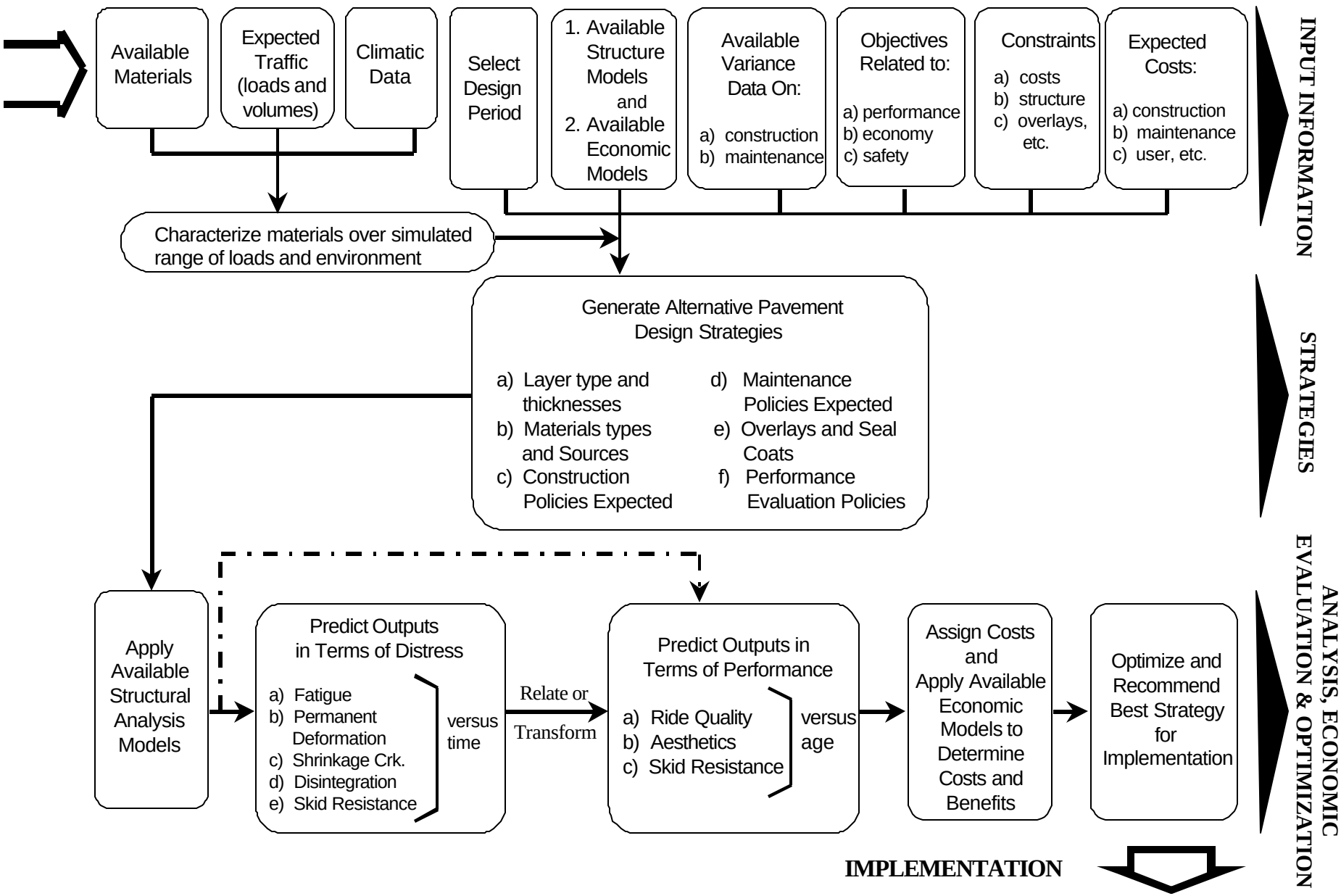


Figure 4.10 Major Pavement Design Components

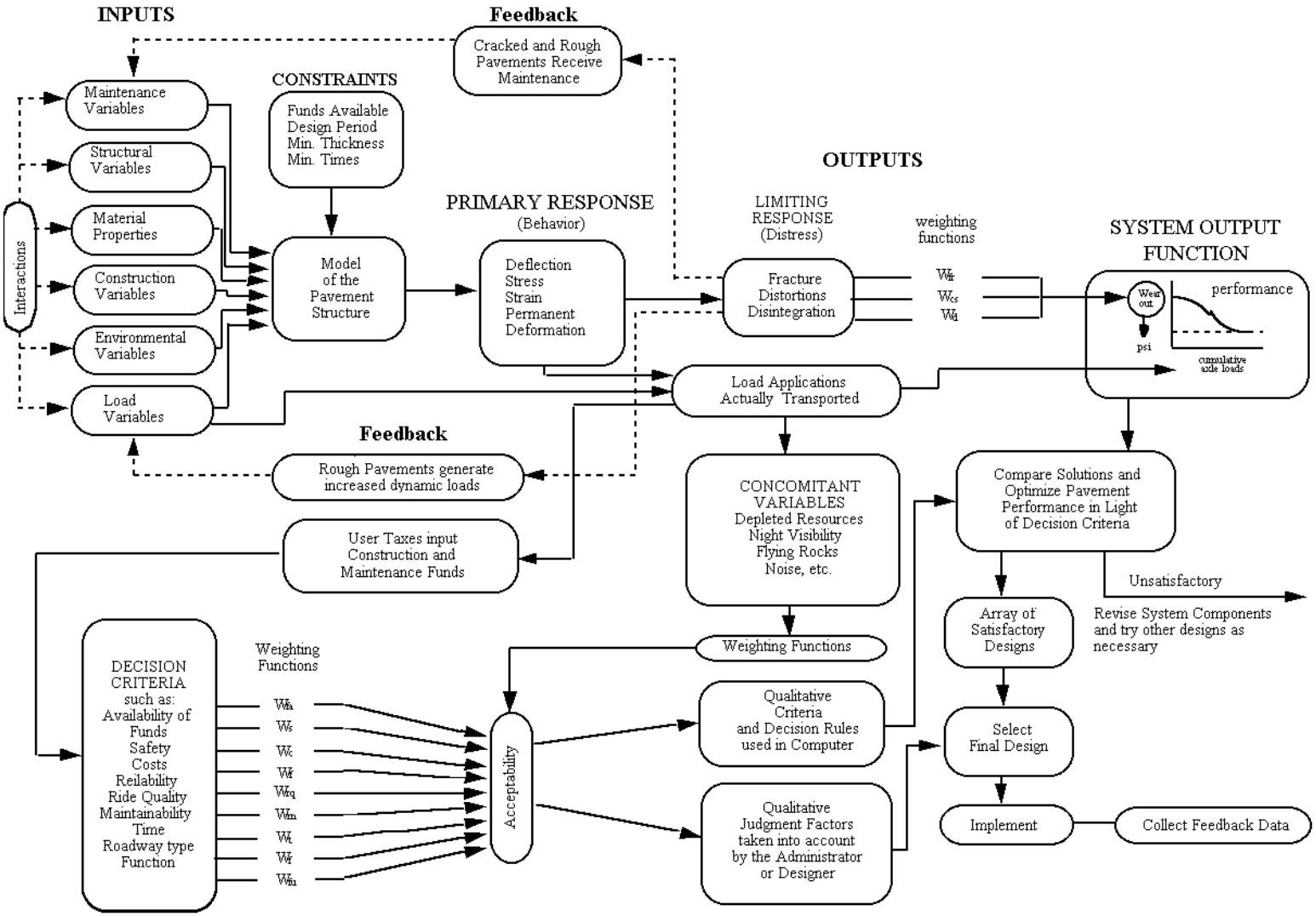


Figure 4.11 Block Diagram of Pavement Design System [Hudson 73]

4.5.1 Information Needs

The top row of Figure 4.10 represents the information and analysis methods that should be available or acquired before generating alternative design strategies. Data on available materials, expected traffic, and climatic factors are often the first information items acquired. Design methods that include materials characterization use these data for establishing a range of load and environment for testing purposes. It might also use the data in both generating and analyzing design strategies.

Depending on the design method, the structural model(s) can range from simple to complex. For example, methods based on empirically based limiting strength values are simple in concept and use; alternatively, a linear visco-elastic analysis is relatively complex both in terms of material characterization and mathematical analysis. This also demonstrates an interaction between the material characterization and the structural analysis processes. Prior to testing for material characteristics, one must know the type of analysis model that will be used in the design process.

The selection of a design period (analysis period, planning horizon, etc.) is only implicitly included in some methods. These involve the situation where design charts are used to provide a particular pavement structure to satisfy the particular input requirements. The charts would then inherently be based on a certain service life (e.g. 20 years) being achieved by the pavement structure. Other design methods might explicitly select a design period, such as 25 years, over which the alternatives are compared. Without a consistent analysis period, detailed economic analysis and comparison of alternatives cannot be properly performed.

The economic analysis models also range in complexity from one design method to another. A straightforward estimate of the initial capital costs of construction may be used in one method, whereas another method might use a net present value model that incorporates present and future costs and benefits over the design period.

Available construction and maintenance variance data are listed along the top row of Figure 4.10, but most design methods use such information only in a subject manner. The 1986 AASHTO Guide for the Design of Pavement Structures incorporated a measure of the observed variance in pavement performance into the design equation. This represents a significant improvement in the state of the art in pavement design. However, it also presents a significant need to quantify the true construction and maintenance variability.

The objectives established for design should be primarily related to performance, safety, and economy. The objectives would define certain minimums and maximums for these outputs. For example, minimum levels of serviceability and friction, and a maximum level of structural capacity, may be defined. Many existing design methods, however, incorporate objectives only on an implicit basis.

Constraints on a design method, per se, or on the designs produced by that method, are usually stated more explicitly. For example, there should be a limit on expenditures, a minimum time to the first overlay, a minimum thickness of pavement, etc.

Expected costs are a vital part of the information needed for design. Among the major cost categories, both present and future, are materials, construction, maintenance, and user costs.

4.5.2 Generating Alternative Pavement Design Strategies

The generation of alternative pavement design strategies is shown along the middle row of Figure 4.10. The term *strategies* emphasizes that a design alternative should consist not only of pavement layer types and thicknesses, including future rehabilitation actions such as overlays and seal coats, but also material types and sources. Additionally, it is desirable to include the expected construction, maintenance, and performance evaluation policies.

The need for a design alternative to specify material types and layer thicknesses is apparent. However, without including the construction and maintenance policies, there may be appreciable error in the predicted performance outputs for that pavement structure.

Overlays, seal coats, or other rehabilitation alternative also become part of any design strategy if the serviceability of the pavement drops to the minimum acceptable or terminal level before the end of the design period. An exception would be a maintenance policy that kept the pavement at or about the minimum acceptable level of serviceability to the end of the design period. Alternatively, if financial constraints prevailed, maintenance at this level might continue only until funds were available for rehabilitation.

In the formulation of rehabilitation alternatives, there are two major, interrelated aspects that should be considered:

1. Structural aspects, with respect to dealing with excessive distress, lack of adequate serviceability, lack of adequate safety, etc.
2. Policy aspects, with respect to traffic handling procedures and time of day and season of the rehabilitation

The method of handling traffic can be a very important consideration in certain situations, primarily in terms of the user delay costs.

The specification of a policy for performance evaluation of a pavement throughout the design period, as part of a design strategy, might be considered unusual in conventional pavement design methodology. An example can illustrate the concept involved. Suppose a particular strategy included an overlay of a certain thickness and materials at 10 years. Some error is, of course, associated with any such prediction. Consequently, it is desirable to have some means for periodically monitoring the performance and structural capacity of the pavement as it is in service so that the design strategy may be updated.

4.5.3 Analysis, Economic Evaluation, and Optimization

The bottom row on Figure 4.10 lists the main component activities that would ideally be involved in the analysis, economic evaluation and optimization of the various alternative design strategies. Most design methods do not include all of these activities; however, design methodology in general appears to be moving toward this idealized form.

The first major step in the analysis of any pavement design alternative is the application of structural models. If they are sufficiently comprehensive, they are used first to predict the outputs of that alternative in physical terms, for example, the distress that is expected to occur over the design period. The major distress modes are fatigue cracking, permanent deformation (primarily rutting of flexible pavements and faulting at joints of rigid pavements, and distortions associated with foundation and environmental influences), shrinkage cracking, disintegration, and loss of skid resistance. Fatigue and usually permanent deformation are associated with repeated loads, whereas shrinkage cracking is associated with temperature change. Disintegration could be associated with both traffic and environment, and the loss of skid resistance is usually associated mainly with traffic effects.

Current technology cannot adequately predict the type and degree of all these distress modes as a function of time and traffic. Consequently, several structural models used today attempt to make a direct prediction of serviceability (or ride quality) versus age. This approach is represented by the dashed line of Figure 4.10, noted as a "combined subjective/analytical link." These terms are used to indicate that some methods might make only a subjective estimate of serviceability versus age, or perhaps only of service life, based on experience. Alternatively, they might involve some combination of such subjective estimates with calculated estimates of service life. The other two outputs, aesthetics versus age and skid resistance versus age, are difficult to predict with current technology. In fact, no acceptable measure of aesthetics or appearance has been developed, probably because most agencies feel that this is a relatively minor component of performance.

Research on pavement performance has failed to produce a definitive relationship of distress outputs and pavement performance. The need for these relationships was defined as a first priority research need in 1970 [HRB 71] and remains an elusive relationship.

The economic evaluation of a pavement design alternative, as shown on Figure 4.10, should involve the assignment of costs and benefits to the predicted outputs, in terms of the user, plus the costs of providing that alternative to the agency in terms of materials, construction, maintenance, etc. These are then incorporated into an economic model to determine the total costs and benefits of the strategy. It is desirable to include benefits in the economic evaluation,

but this is a relatively undeveloped aspect of pavement design. One reason is that it is difficult to separate pavement benefits from overall highway benefits. Another is that the differences in user costs between pavement design alternatives, which can be considered as benefits, are difficult to determine. Research by the World Bank and subsequently by the Federal Highway Administration has successfully related vehicle operating costs to pavement condition [Zaniewski 82 and Chesher 85]. Thus, the information is available for including user cost analysis in the economic analysis of a pavement strategy.

When all the alternative design strategies have been analyzed and evaluated, optimization should be used to select the best strategy. Frequently, this is accomplished by selecting the alternative with the least discounted total cost. Recommending this strategy for implementation then completes the design.

4.6 LIFE-CYCLE COST ANALYSIS

4.6.1 Introduction

The life-cycle cost of a pavement is the discounted value of investments that occur over the life of the pavement. Pavement improvements are meant to correct structural or functional deficiency. Current procedures allow the designer to correct inadequacies in several ways. Unfortunately, these procedures do not always include an estimate of the most cost-effective alternative. Life cycle cost analysis is used to address this problem.

In classical life cycle cost analysis, a finite number of alternatives are formulated, such that each corrects the deficiency in the pavement structure. Then estimates of the investments required to maintain the pavement over a common analysis period are made. The total investment of each alternative over the analysis period is computed and discounted appropriately to arrive at the expected life-cycle cost. Using these estimates, the decision maker is able to choose the most cost effective life-cycle alternative that meets the objectives and goals of the project.

These estimates of life-cycle cost require that the analyst estimate many factors that, by their nature, are inherently uncertain. To estimate a life cycle cost for a pavement investment, the analyst must predict the future performance of a pavement, the costs of materials, appropriate discount rates and other factors, which greatly affect the output of any life-cycle cost model. In many cases, however, decisions are made using life-cycle costs that have neglected the fact that these estimates are not known values but have an inherent variability that should be represented by a distribution of values. This section discusses the life cycle cost analysis procedure that accounts for this variability in the inputs and provides a clearer picture of its effect on the life-cycle costs of pavement treatment alternatives.

Since it is important to estimate variability involved in the choice of an alternative, the question remains as to the method of measuring or quantifying the variability across the alternatives. Many factors used for life cycle cost analysis are variable, and it is possible to use historical data and judgment to estimate the distributions of these factors over the life cycle of a pavement. It may not be known exactly how long a particular maintenance or rehabilitation treatment will last, but it is possible to estimate the expected distribution of the life through historical data or local experience. Therefore, the analyst should estimate not only the expected values for variables, but also the distributions of these variables for input into the analysis. Then, using these distributions of the input variables, the analyst can estimate the distribution of the output variable, which is the present value of the investments needed over the life of the pavement (life-cycle cost).

The distribution of this cost is used to estimate the risk of each alternative. The risk of the design alternative is directly proportional to the standard deviation of the distribution of total life-cycle cost. In short, the higher the standard deviation of this distribution, the more uncertain you are about its actual cost, hence more risk.

4.6.2 Life Cycle Cost Principles

Decisions are not made in a vacuum but, rather, with some objective or goal in mind. A transportation agency's primary goal is to provide a network of serviceable and safe pavements at a minimum cost to the agency. This is the goal or objective on which the economic analyses discussed in this section are based.

The application of economic principles occurs at two levels. First there are the management decisions required to determine project feasibility and timing and considering the many possible projects that may be undertaken at any one time. Second is the requirement to choose an alternative that provides the maximum economic benefits for the project once it has been selected. This is the typical distinction between network and project levels of pavement management. Project feasibility is determined at the network level while comparing within-project alternatives as part of the project level pavement management process.

Economic analysis takes place at both levels of pavement management. The major difference in economic evaluations between these two levels of pavement management relates to the amount and types of information required. However the basic principles that apply for economic analyses at both the project and network levels of pavement management remain the same. [Haas 94]

These can be summarized as follows:

1. The management level at which the evaluation is to be made must be clearly defined
2. The economic analysis provides the basis (information) for a management decision but does not by itself represent a decision.
3. Criteria, rules, or guides for such decisions need to be separately formulated before the results of the economic decision are applied.
4. The economic analysis itself has <u>no</u> relationship to the method or source of funding for a project.
5. An economic analysis should consider all possible alternatives, within the constraints of time and other planning and design resources.
6. All alternatives should be compared over the same time period.
7. The economic analysis should include all of the benefits and costs of the project alternatives if possible.

When these basic economic principles are followed, the effective use of economic analysis methods will help manage the difficulties associated with project selection (network level) and alternative selection (project level). Better management will occur by improving both the timing and cost effectiveness of asset acquisitions, and the quality of the assets purchased.

4.6.3 LCCA Requirements

Weston [Weston 90] lists six necessary steps to follow when performing life cycle cost analysis for a project. These steps identify the minimum information that should be identified for a complete analysis. These are:

1. The benefits and costs of the project must be identified.
2. Estimates of expected cash flows (benefits and costs), including the terminal value of the asset at a specified terminal date must be developed.
3. The appropriate discount rate must be determined.
4. The expected cash flows are discounted to a present value basis to obtain an estimate of the projects current value.
5. The risk of the projected cash flows; and/or information (sensitivity analyses) about the probability distributions of the cash flows must be determined.
6. The present values of the expected cash flow and sensitivity analyses are compared.

Identifying Project Benefits and Costs

In evaluating a project, only cash flows (benefits and costs) resulting directly from the project is used. They represent the change in the total cash flow that occurs as a direct result of accepting or rejecting the project. When identifying project costs for inputs into the life cycle cost analysis the following principles are followed [MDOT 94]:

1. Only incremental cash flows are used. These are cash flows that result directly from the project being analyzed. Sunk costs are not used because they have already been incurred and cannot be recovered.

2. The costs used for analysis should reflect the opportunity cost of any resource used, measured by the return to those resources in their most productive application elsewhere. This is typically measured by the market price that represents what society is willing to forego in order to obtain a given benefit. [OMB 92]

3. Both tangible and intangible benefits and costs should be recognized and accounted for whenever possible. [OMB 92]

4. Benefits and costs may be measured in real or nominal terms. A real benefit or cost has been adjusted to eliminate the effect of expected inflation. A nominal benefit or cost is equal to the corresponding real benefit or cost plus a premium for expected inflation. An important note about inflation is that nominal benefits/costs include a factor corresponding to expected or future inflation, not inflation that has been experienced in the past or present. The question of how to take inflation into account during an economic analysis is of great concern to many. Basically, the answer is that inflation is **not used** in the evaluation, except where substantial evidence exists that real prices will change. Real price changes can be defined as the change in the price of an item that does not follow the general trend of inflation of all goods in a society. Some would argue that inflation should be included in the analysis because ignoring inflation leads to underestimating out-of-pocket costs and therefore budgets will be incorrect. This argument indicates a misunderstanding of the objective of an economic analysis, which is to provide management with a tool for the selection of specific options from a set of alternatives. Once the option is selected, a separate budget analysis is required to determine cash flow requirements. The budget analysis generally includes inflation and other factors. [Haas 94] Therefore it is recommended that only current market prices be used for economic analysis.

Estimation of Expected Cash Flows

The estimation of expected cash flows requires determining two elements: the magnitude of the cash flow and the point in time when that cash flow will occur. Both these elements are not point values and are subject to variation. To estimate these values the analyst should estimate the following parameters for each element of each cash flow in the analysis:

- The mean (expected value) of the cash flow or when the cash flow will occur.
- The distribution of the cash flow (for both its location in time and its magnitude). The distribution may be estimated in a number of ways. In MDT's LCCA software package the distributions of cash flow timing and magnitude can be input by assuming the distributions are either triangular or normal. The mean and standard deviation is sufficient to define a normal distribution. For triangular distributions the analyst must specify the expected value, and the range of possible values. The specification of these distributions will be discussed in more detail later.

Reduction of Cash Flows to Present Values

The life cycle cost model used to prepare the estimates is based upon a cash flow diagram and the time value of money. Each cash flow is appropriately discounted to today's dollars so the present value can be determined.

Risk of Projected Cash Flows

Risk can be described as the amount of uncertainty present in a decision made under unknown or uncertain conditions. For pavement investments, many factors cannot be predicted with certainty; therefore, there is a risk involved with every decision made. Since there is any number of design alternatives available that will provide an adequate pavement structure for the analysis period, the final decision is dependent upon the total life-cycle cost of each acceptable design. These costs are based upon uncertain variables and, therefore, contain some amount of risk. This risk must be considered during the decision making process for a truly informed and competent decision to be made.

Risk is defined as the probability that an unfavorable event will occur. Estimates of benefits and costs are typically uncertain because of imprecision in both underlying data and modeling assumptions. Useful information in an analysis includes the key sources of uncertainty; expected value estimates of outcomes; the sensitivity of the results to the important sources of uncertainty; and the probability distributions of the project benefits and costs. [OMB 92]. As stated above the analyst should determine the distribution of the input variables to the life cycle cost model. These distributions can then be used to perform three types of analyses that, when used together, can present a picture of the risk of a selected alternative. These analyses are simulation, sensitivity, and scenario analyses.

A simulation analysis is used to construct the distribution of the total present value of the life-cycle cost for an alternative. Basically the life-cycle of the project is simulated using a Monte Carlo technique by randomly selecting values from the distributions of all the cash flows expected to occur during the project life-cycle and then producing a present value estimate of the life-cycle cost. Therefore each simulation run represents a possible outcome of the life cycle for the alternative. These simulations are run many times and the present values tabulated to produce an estimate of the expected distribution of possible life-cycle costs for the alternative.

The sensitivity analysis basically portrays the sensitivity of the present value of the total life cycle cost to changes in each of the input variables. The life cycle cost for all alternatives is calculated using a range of values for each input variable based upon a user-selected range of that variable. The results are plotted to show the relative effects of these variables on the life-cycle cost.

Scenario analysis uses the input distributions of the cash flows to give worst, best and expected case life-cycle costs for each alternative. These estimates show the possible range of life-cycle costs for each alternative.

Using the output of the three analyses the decision maker can compare each alternative and select the best based upon the agency's goals and other constraints.

4.7 DESIGN OBJECTIVES AND CONTRAINTS

Pavement management helps the engineer focus on the basic function and constraints of the design process. This avoids routine application of the design procedure while ignoring other available alternatives. One of the first activities that should be accomplished under the systematic approach is to carefully define the objectives and constraints of the problem. Objectives and constraints apply to both the pavement and the design process or the designer's activities as explained in the next three sections. Some apply to both [Haas 94].

4.7.1 Pavement Objectives

The principal objectives to be fulfilled by a pavement, both during construction and in service, are economic and social in nature. They include the following:

1. Maximum or reasonable economy, in terms of both agency and user costs
2. Maximum or adequate safety
3. Maximum or reasonable pavement serviceability over the design period

4. Maximum or adequate load-carrying capacity, for both the magnitude and number of repetitions of the loads
5. Minimum or limited physical deterioration due to environmental and traffic influences
6. Minimum or limited noise and air pollution during construction
7. Minimum or limited disruption of adjoining land use
8. Maximum or good aesthetics

These objectives pose several major contradictions, which occur with any complex system that attempts to fulfill social and economic needs. Consequently, each pavement constructed represents some compromise or trade-off between competing objectives. The relative influence of the objectives in a particular design situation varies with such factors as rural or urban area and traffic volumes.

The first three objectives listed are very important. Economy is a prime requisite, because most public agencies feel that they have insufficient funds available for all the desired pavement investments. Safety, as achieved through sufficient skid resistance, proper lane demarcation and light reflectivity during night driving, etc. is another major requirement the pavement must fulfill. Serviceability should also be sufficiently high over the design period to ensure the desired level of quality to the traveling public in terms of speeds, vehicle operating costs, user delays, and comfort.

The fourth objective, maximizing load carrying capacity, is applicable to both the pavement and to the design process. This objective competes directly with the objective of maximizing economy.

The fifth objective, minimizing physical deterioration, is related to serviceability in that deterioration is a major factor in causing serviceability loss. However, it is identified as a separate objective because it also competes with the economic objective in terms of maintenance and rehabilitation costs.

The last three objectives are usually of lesser importance. Aesthetics or appearance can be a significant objective for certain special applications (e.g. pedestrian walkways) or in those cases where patching and crack filling are extensive and appear ugly to motorists or adjacent property owners.

4.7.2 Pavement Design Objectives

In one sense, the objectives of the pavement design process is to design a pavement structure that, when constructed, will accomplish the functional objectives defined above. However, the specific objectives of the design process can be stated in specific technical, economic and social terms:

1. Develop a design strategy of maximum (or reasonable) economy, safety, and serviceability.
2. Consider all possible design strategy alternatives.
3. Recognize the variability in the design factors.
4. Maximize the accuracy of predictions of serviceability, safety, and physical deterioration for the alternatives considered.
5. Maximize the accuracy of estimating cost and benefits.
6. Minimize the costs of design, including labor, testing and computer time.
7. Maximize information transfer and exchange between construction and maintenance personnel.
8. Maximize use of local materials and labor in the design strategies considered.

Some of the foregoing objectives correspond with the objectives for the pavement itself, whereas others are peculiar to the design process. Most of the objectives compete with other objectives and require tradeoffs.

The first objective simply reflects the need for the design to recognize the pavement objectives of economy, serviceability, and safety. In order to accomplish this, it is necessary, as stated in the second objective, to consider many possible design strategy alternatives. Because these can amount to several hundred combinations for any given situation, the use of computer analysis is necessary for accomplishing this objective.

The third objective–recognition of the stochastic nature of pavement design, construction, and maintenance factors–is accepted as important criteria for a pavement design method. Early contributions to the inclusion of variability in the pavement design process were presented by researchers at the University of Texas in the early 1970's [Darter 71, Darter 73a, Darter 73b]. Variability concepts have been incorporated into the 1986 version of the AASHTO Guide for the Design of Pavement Structures.

The fourth objective of maximizing the accuracy of the design predictions is related to the third objective and also to the quality of the input data. Decreased uncertainty in design predictions is important to planning activities in that investment programming can be accomplished more reliably.

The accuracy of the cost and benefit prediction, the fifth objective, is directly related to the accuracy of the design predictions as well as the ability to estimate the construction, maintenance and user costs.

Minimizing the costs of the design process in terms of materials testing, personnel time, preparation of drawings, etc., the sixth objective, is important but obviously competes with the other objectives. If the cost of the design process is under funded, the result can be an incomplete, inadequate or excessively costly design strategy.

The seventh objective of information exchange between those responsible for construction and maintenance has been neglected to a large extent in design practice. There have been instances in North America of maintenance crews unfamiliar with continuously reinforced concrete pavements trying to fill with asphalt the numerous hairline cracks that are a design feature of this pavement type. The resulting mess might be more readily ascribed to the designers' failure to communicate with the maintenance people than to the maintenance people themselves. It is extremely important that such communication occur, because it is construction and maintenance policies that "carry" the design through its design period.

The last objective, involving the use of local materials and labor, is perhaps a self-evident desirability for most situations. This objective may complement the first objective of maximum economy, but in some cases local materials may require stabilization or treatment.

4.7.3 Design Constraints

The major design constraints are economic and physical or technical in nature and include the following:

1. Availability of time and funds for conducting the design and the construction.
2. Minimum level of serviceability allowed for the pavement before rehabilitation.
3. Availability of material.
4. Minimum and/or maximum layer thicknesses.
5. Minimum time between overlays or seal coats.
6. Capabilities of construction and maintenance personnel and equipment.
7. Testing capabilities.
8. Capabilities of the structural and economic models available.
9. Quality and extent of the design information available.

The first constraint might involve not only a limit on the funds that can be applied to a particular project, but also the opportunity costs of those funds. All the other constraints, which are essentially physical or technological, are relatively self-explanatory. Each has economic implications and is therefore related to the first constraint.

4.8 EXAMPLES OF HISTORICAL PERFORMANCE AND PAVEMENT MODELING

This section will discuss the use of performance and design models for engineering analyses, and examples will be given about the way such models create possibilities to quantify performance over time in measurable terms. These examples come from work carried out by DOT's in Pennsylvania and Washington State. The subjects are:

- PennDOT: Pavement performance data and modeling;
- WSDOT: Pavement performance modeling;

The examples of engineering analyses will be preceded by a brief description of the PMS organization and procedures in each of these DOT organizations.

4.8.1 PENNDOT – Pavement Performance Data and Modeling

PennDOT's PMS Overview

The Pennsylvania Department of Transportation's (PennDOT) pavement management system (STAMPP) was implemented in 1986 as a part of a comprehensive Roadway Management System (RMS). The following is a list of the primary management systems that are a part of the RMS.

- Systematic Technique to Analysis and Manage Pennsylvania's Pavements (STAMPP)
- Bridge Management System (BMS)
- Project Management System (PRMS)
- Maintenance Operations Resource Information System (MORIS)
- Accident Records System (ARS)
- Financial Management Information System (FMIS)
- Construction Management System (CMS)
- Construction and Materials Management System (CAMS)
- Geographic Information System (GIS)

The RMS serves a highway network, which has 4,300 miles on the National Highway System and 41,000 miles of other highways. This system provides the primary input for developing an annual construction and maintenance program of $1.75 billion.

STAMPP

The original Pennsylvania pavement management system known as STAMPP conducted an annual pavement condition survey on all of the Interstate roadways and 50% of the non-interstate systems. The data are input into PennDOT's mainframe database. The core of STAMPP is to provide a comprehensive compilation of pavement distress data collected annually by the eleven engineering districts. Distress is reported separately for each segment of road, with typical segments being a nominal half-mile length. These segments are well defined and marked on the roadway, making segment data comparable from year to year. All lanes of the roadway are evaluated with multiple passes required in each direction for four or more lane highways. Surveys are conducted between May and September each year. Survey methods are divided into three primary groups: bituminous, jointed concrete, and continuously reinforced concrete. A majority of PennDOT's highways are either jointed concrete or bituminous. The conditions and distresses surveyed are listed in Table 4.5.

Table 4.5 Conditions and Distresses Surveyed

Bituminous Pavements	Joint Concrete Pavements	Continuously Reinforced Concrete Pavements
Excess Asphalt	Joint Seal Failure	Longitudinal Joint Spalling
Raveling & Weathering	Longitudinal Joint Spalling	Transverse cracking
Block Cracking	Transverse Cracking	Punchouts
Transverse Cracking	Transverse Joint Spalling	Bridge Approaches
Longitudinal Cracking	Faulting	Damaged Terminal Joints
Alligator Cracking	Broken Slab	Rutting
Edge Deterioration	Bituminous Patching	
Bituminous Patching	Surface Defects	
Potholes	Rutting	
Widening Drop off		
Profile Distortion		
Rutting		

When surveyed each distress is rated with regard to severity and extent. Severity reflects the degree of each distress and extent quantifies the amount of distress that occurs within the segment. Distresses can be assigned one of the three different severity levels and three different extent levels for most of the distresses surveyed.

For definition of distresses please contact PennDOT for the "Pavement Condition Survey Field Manual," Publication 36 [PennDOT 92].

Combined Pavement Indexes

Individual distress indexes are weighted and combined to produce combined pavement indexes by other equations. PennDOT uses the following indexes: structural index (SI), surface distress index (SDI), safety index, overall pavement index (OPI) and a present serviceability rating (PSR). For project analysis, which usually includes several segments, the individual and combined pavement indexes are obtained by averaging the values for all segments within a particular project.

Project Analysis

Projects limits are established by pavements of similar performance usually based on the length of the last resurfacing or reconstruction project. The type of treatment for a project is based on a decision tree of established maximum or minimum values for different indexes. The five treatment groups defined in STAMPP are as follows:

- Routine Maintenance: Strategies include such items as crack sealing, joint cleaning and resealing, shoulder maintenance, and minor patching.
- Preventive Maintenance: Strategies include minimum overlays with leveling, mill and overlay, patching, diamond grinding, and minor joint repair.
- Concrete Pavement Restoration (CPR): Strategies include patching and joint repair, joint repair, slab stabilization, diamond grinding, and shoulder rehabilitation.
- CPR & Overlay: Strategies are similar to CPR with the addition of a 3.5" overlay or greater.
- Reconstruction: Strategies involve complete reconstruction, either by removing the existing pavement or by incorporating rubblization, break and seat, or crack and seat.

Project costs are based on the required treatment group. The approximate dollars needed to address the pavement conditions are based on average costs for the treatment and number of lanes involved.

Outputs of RMS and STAMPP

The RMS with data taken from STAMPP and other management systems produces numerous reports and programs.

Every year each PennDOT's county maintenance organization puts together an annual work plan based on information contained in the RMS. Information such as pavement distress, guide-rail conditions, and traffic are used to establish the work plan. Allocation of maintenance funds are partially based on pavement conditions, roughness data, traffic counts, mileage, bridge deck area, etc. Automated permit routing for overloads and wide loads are also developed using the system.

Graphical maps are also provided using GIS. Maps showing pavement condition and needed projects can be developed for county, district, or state maps. An annual State-of-the-Interstate Report is produced using pavement condition data, roughness data, and detailed traffic-count data from RMS to analyze the current and projected rehabilitation needs of the Interstate system.

The metropolitan planning organizations use pavement management data to develop their multiyear transportation improvement program (TIP). In turn these plans are combined with PennDOT's TIP to produce a statewide TIP. PennDOT develops Highway Performance Monitoring System reports that are submitted to the Federal Highway Administration using RMS and STAMPP data.

Pavement Performance Modeling Program (Visual Modeler)

PennDOT has been using Visual Modeler as a PMS subsystem for Pavement Performance Modeling in developing reliable pavement performance models from the available data. Visual Modeler imports data files from the PennDOT mainframe. The system provides visual displays of each model and provides methods for adjustment where necessary. The models built by this program can be applied to network level resource optimization for pavement maintenance and rehabilitation and project level life cycle cost analysis. The Visual Modeler subsystem is also integrated in PennDOT's new prototype pavement management software, Visual/PMS™ that has not yet undergone a full implementation process. Details of the performance model program and examples of model development follow [Hudson 97].

PennDOT's Change to Automated Condition Survey Methods

The PennDOT Bureau of Maintenance and Operations performs a yearly pavement management survey of approximately 26,000 miles. The survey typically includes 100% of the National Highway System and 50% of other pavements under the PennDOT system. Through 1996, the distress data collection portion of the survey relies on manual windshield methods to collect several distress types on the two major surface types used by PennDOT: bituminous surface and jointed concrete pavements. In addition to the manual distress surveys, PennDOT collects International Roughness Index (IRI) data using several South Dakota style profilers.

The manual distress survey used to be performed by approximately 200 college students trained in the data collection method. Students were sent to data collection regions in Pennsylvania where they collected data during the summer months. In addition to the college students, PennDOT staff followed behind the distress rater teams performing independent quality assurance audits to ensure data collected by the teams are within the quality guidelines established by PennDOT.

PennDOT decided to change to contracting for automated collection of pavement condition data because of the following disadvantages of their current in-house process.

- Yearly, time consuming manual training programs sponsored by PennDOT
- Exposure of the manual raters to traffic
- Variation in distress data results between rater teams
- Travel and per diem expenses for the raters
- Use of an independent profiler and, therefore, additional time and effort for roughness data collection

Beginning in the 1980's several states and private companies began research and development of automated equipment with the objective of collecting surface distress and other data at highway speeds. Data analysis and reporting were made either off-line or in real-time depending upon the data reduction approach. PennDOT closely monitored the developments and in 1995, undertook a study to evaluate and establish a means to select an automated distress data collection service provider to replace the manual distress rating procedure/PennDOT profiler combination [TRDF 97] [TRDF 97a] [TRDF 97b].

The objectives of this study were to:

- Review current PennDOT distress measurements, redefine distress measures as appropriate and prioritize pavement distress measurements.
- Design an experiment to evaluate the equipment offered by interested service providers.
- Assist PennDOT in developing a set of data collection specifications used in a request for proposal.
- Analyze the data to assist PennDOT in selecting a "best-value" data collection service provider.
- Review the service provider submitted data to see how well distress data under the development for the FHWA was reported by the service providers.

The research project team first concentrated on development of a new distress rating procedure for collecting pavement distress data. The new distress data collection procedure considered both PennDOT's pavement management needs and current reported capabilities of network level, distress data collection equipment.

Next, an experiment design was developed to determine the appropriate testing procedure to ensure that service provider supplied data could be successfully compared to PennDOT manual distress data using a manual procedure based on the new automated distress rating procedure. Finally, data collected by the raters and service providers were compared to each other to determine how well each vendor and manual rater compared to the group.

Information from the service provider data analysis was used to award a pavement management data collection contract for 1997. The procedure used to award the contract was not a simple function of a service provider's ability to collect distress and report pavement distress. Rather, PennDOT used a weighting scheme that included: distress collection and performance, data take-off/analysis method, proposed work plan, and price to evaluate which vendor would provide the "best-value" for pavement management surface distress and roughness data collection.

Pavement Performance Analysis

Pavement performance modeling has been shown to be absolutely essential for good pavement management practice at all levels: project level to national network level. Pavement performance models in PennDOT are based upon the pavement historical data collected in the STAMPP program over the last 15 years. Many factors such as pavement surface type, M&R treatment, traffic, subgrade type, construction materials, maintenance level, environment, climate etc. have effects on pavement performance. The STAMPP programs previously used by PennDOT do not make use of predictive models for estimating future condition. PennDOT is currently in the process of upgrading their pavement management software by implementing the TRDI Visual/PMS™ [TRDF 97] [TRDF 97a] TRDF 97b] system for developing and using performance models in network and project level analysis for multi year planning and life cycle costing. These new models will greatly improve the state's ability to predict pavement life correctly.

Pavement performance prediction is the most technologically difficult portion of pavement management because 1) the uncertainties of pavement behavior under changeable traffic load, environment etc., 2) the difficulty of quantifying many factors affecting pavements, 3) the error associated with using discrete testing points to represent the total pavement area when estimating pavement condition, and 4) the nature of the subjective condition survey. To develop the best models from the available data and update these models as more data becomes available is one of the most important tasks for engineers and researchers in pavement management. The development of pavement performance models involves extensive effort to create a data file (or a database) by joining and calculating data from original data files. Currently, many researchers use a single modeling equation to produce pavement performance models for all types of pavements. One reason is that no specific software has been available to allow relatively easy manipulation of a historical database along with development of various performance model types. A single model form may produce reasonable results, but may not get the best results due to pavement performance

variability observed in the field. The PennDOT system however creates individual equations for each performance class and Performance Index used.

Modeling Process

The Visual/PMS™ modeling process uses group-based modeling. The engineer selects any number of variables known to affect pavement performance and divides the pavement network into homogeneous performance groups. Inside of each group models are developed for each condition index used in the system. Table 4.6 shows an example list of variables that may be considered for creating the homogeneous groups of pavements.

For example suppose the engineer chose AADT level and a functional class variable to define the pavement performance classes. In PennDOT there have been eight functional classes defined and three levels of traffic. This produces 8 x 3 =24 groups for creating models. This definition creates a matrix of groups as shown in Table 4.7.

Table 4.6 Example Performance Group Variables used in the PennDot Modeling Process

• Traffic Level (AADT)
• Subgrade Strength
• Truck Percentage
• Pavement Type
• ESAL Loading Level
• Functional Classes
• Pavement Surface Thickness
• Surface or Base Material Types

Table 4.7 Illustration of an Example Group Definition

	Low Traffic AADT<750	**Medium Traffic** 750<AADT<3250	**High Traffic** AADT>3250
Interstate Urban	1	9	17
Interstate Rural	2	10	18
NHS Urban	3	11	19
NHS Rural	4	12	20
Arterial Urban	5	13	21
Arterial Rural	6	14	22
Non Arterial Urban	7	15	23
Non Arterial Rural	8	16	24

In addition models are automatically created for different pavement structural types. These classes of pavement structural types are called Road Structure Categories (RSC) and are usually defined by a combination of three values:

1. Pavement surface material type (asphalt concrete or a specific type of portland cement concrete)
2. Pavement surface layer thickness
3. A rehabilitation code that indicates whether the pavement is an overlay of an existing pavement structure or a newly constructed pavement

For example, a typical RSC used in PennDOT is: "Thick ID2 over PCC". This category indicates that the pavement surface is thick, its material type is ID2 (this is a particular asphalt mix as defined by PennDOT design specifications), and it is an overlay of a plain jointed portland cement concrete pavement. Another example of a RSC is: "Thick ID2 New Construction" which indicates a thick ID2 surfacing layer on a newly constructed pavement.

Using these two Road Structure Categories as examples and the grouping variables listed above we now have a total of 3 (AADT Levels) x 8 (Functional Class Levels) x 2 (RSC) = 48 homogeneous pavement performance groups. Figure 4.12 illustrates the grouping concept. The modeling process then allows the engineer to build one model for each condition index within each of the 48 groups. Thus it is important to carefully select grouping variables and levels to limit the total number of models, which must be developed and maintained. It must always be considered that these are network level models, developed from network level data for use in network level analysis. In selecting the variable to use for grouping availability and reliability of network PMS data used in the modeling process must be considered. For example, although materiel properties may greatly affect pavement performance inclusion of a variable such as "deviation of asphalt cement content from original mix design limits" at three levels (e.g., 1. below limits, 2. within limits, and 3. above limits) is impractical from several perspectives. First, such data

is generally not available at the network level. Second, inclusion of such detailed grouping variables across all performance influencing factors would result in literally hundreds of thousands of groups needing separate models. And finally, even if the user set up such detailed groupings, there would be very few groups that would contain an adequate number of sections to produce statistically valid models. Therefore, selection of grouping variables must e done to select variables that most significantly influence performance, are readily available at the network level, and keep the total number of performance groups to a reasonable level.

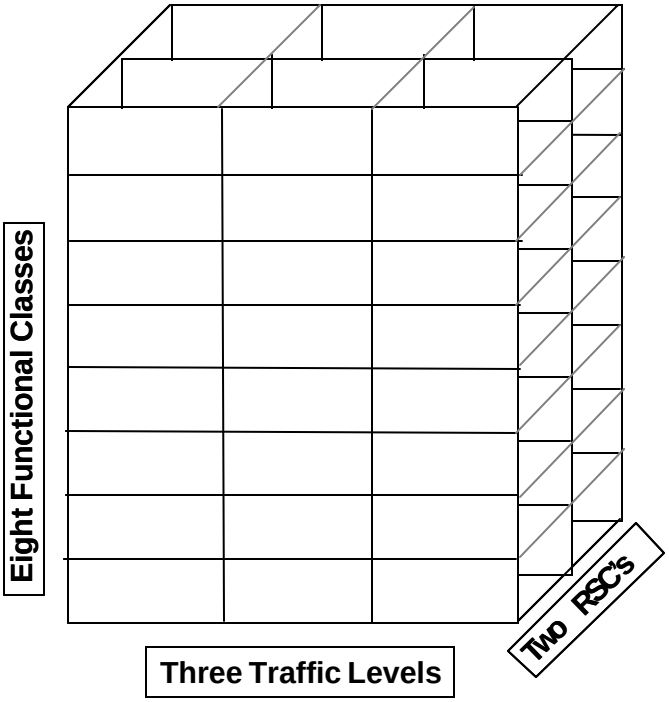


Figure 4.12 Illustration of Performance Class Structure with Traffic, Functional Class and Road Structure Categories (RSC)

Use of Condition Data

In the PennDOT modeling procedure the distress and profile condition data is first converted from raw ratings and measurements into distress scores. Each condition measure has an associated distress score on a zero (bad) to 100 (excellent) scale. Then the distress scores are combined to produce the performance indexes that are used for modeling. Distress scores in the Visual/PMS™ system are the inputs to the performance index calculations. Each distress included in the system will have an associated distress score. The distress score is a number (or index) on a scale from 0 - 100 that represents the condition of the pavement with respect to that distress. For example a pavement can have an alligator distress score that is calculated from the observed severity and extent ratings for alligator cracking on that pavement. It also has a ride distress score that is based upon the IRI for the pavement. During the calculation of the performance indexes these distress scores are combined using a multiplicative index approach we will discuss later.

There are two methods for calculating a distress score:

1. Distress converters - these are single dimension distresses that are directly converted into a distress score. IRI (roughness) is a condition measure that is translated using the distress converters to a 0-100 scale.
2. Maximum allowable extent - This method allows you to convert multi-dimensional distresses (i.e., distresses with both severity and extent measures) to distress scores. This method uses a linear conversion

between distress deduct value and extent measurements within each severity level and a multiplicative combination of deducts across severity levels.

Distress Scores Calculated by Conversion

Distress scores that are calculated by conversion are used for those distresses that are measured in one dimension, either by a severity level (IRI could be a measure of severity in this context), or by an extent level without regard to severity, for example a count of the number of potholes. These distresses can be directly converted to a distress score using the distress converters. Consider the calculation of the PennDOT Ride Index that is used to summarize roughness in the annual Pennsylvania State of the Interstate report. The equation performs the distress converter function. Figure 4.13 shows the relationship between IRI and the PennDOT Ride Index. This is also the converter function used for creating distress scores for roughness (IRI).

In the Visual/PMS™ system this conversion is done by approximating this relationship using four line segments. A possible linear approximation of the Pennsylvania ride index equation is shown in Figure 4.14. These line segments are user definable and can be approximated by choosing the endpoints of each line segment by minimizing the area between the line segments and the equation. This was done to produce the graph shown in Figure 4.14. The amount of error between the approximation and the equation is affected by the range of possible IRI values. In the PennDOT database there are three characters for the IRI number, therefore the maximum possible number is 999. However, an IRI of 350 in/mi. may be the maximum likely value to occur and using this as the upper limit would penalize the segments with IRI values greater than 350 in/mi. more than the original equation. This forces most of the error in the linear approximation to the >350 in/mi. range and allows the lower ends of the scale to be more accurate. Therefore, minimizing the amount of error between the equation and the converted IRI values between 0 and 350 may be the best use of the distress converter.

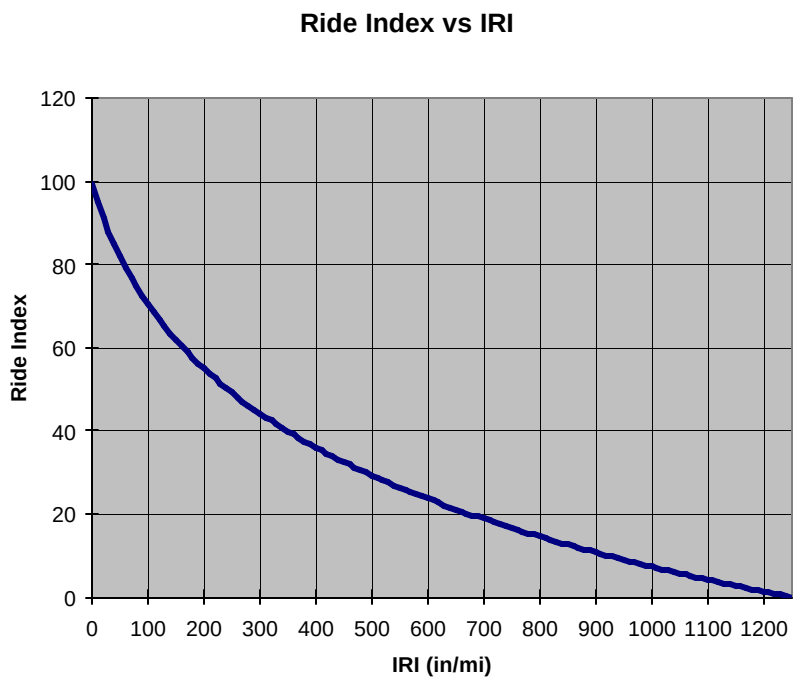


Figure 4.13 Pennsylvania Ride Index vs. IRI

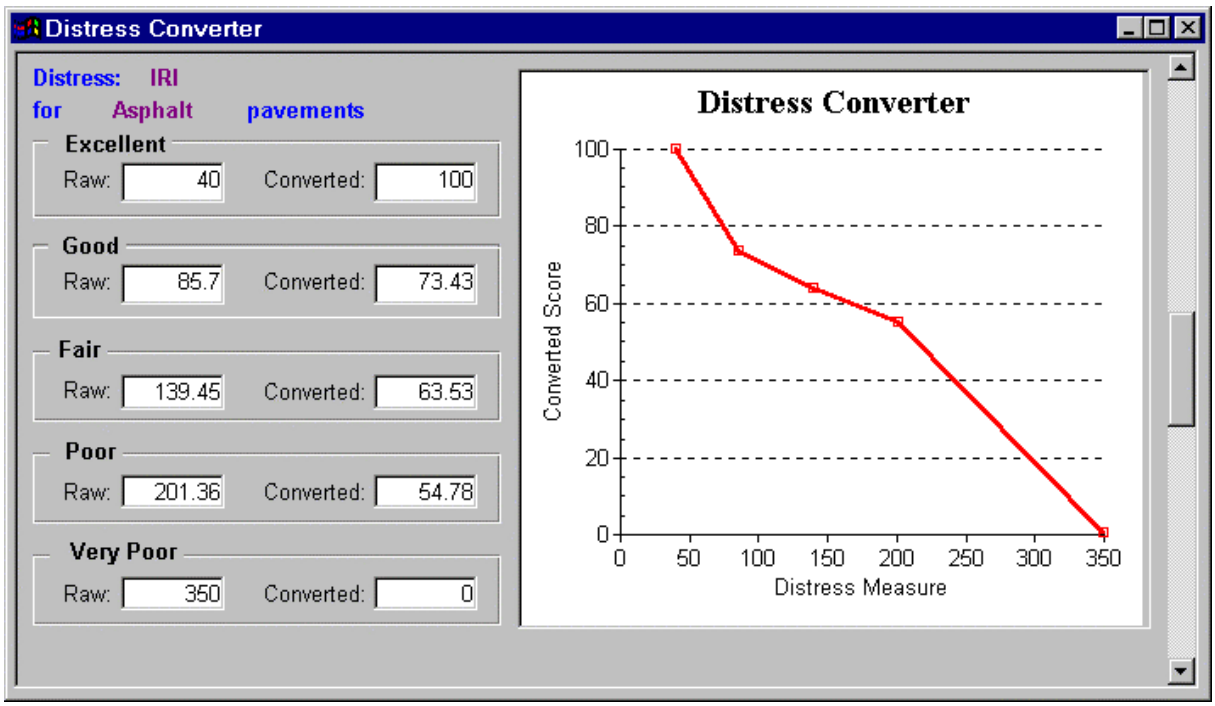


Figure 4.14 Converter for IRI to Produce the Ride Distress Score

Calculation of Distress Scores using the Maximum Allowable Extent (MAE) Algorithm

Maximum allowable extent (MAE) is used to convert two dimensional distresses into distress scores. These distresses are typically those with both severity and extent measurements. The graph shown in Figure 4.15 is a typical maximum allowable extent setup. On the "Y" axis is the 0-100 score assigned for each severity level of the distress. On the "X" axis is the extent measurement for the distress. The three lines are the deduct functions for each of the severity levels. Each of these lines is determined by two coordinates, each specified by the user. The low range value and the high range value. The low range value is that point where deduct points begin to accumulate for each distress. In most cases this is zero on the extent scale as shown for both the moderate and high severity levels in Figure 4.4. But in some cases you may wish to specify that the distress score only begins to deteriorate after the extent value reaches a particular level. Usually if this is specified it is for low severity distresses. In Figure 4.15 note that the low range value is 20% extent. The high range indicates the lowest possible deduct score for a given severity level. In Figure 4.15 this is shown as a score of 40 for an 80 % extent for low severity distress.

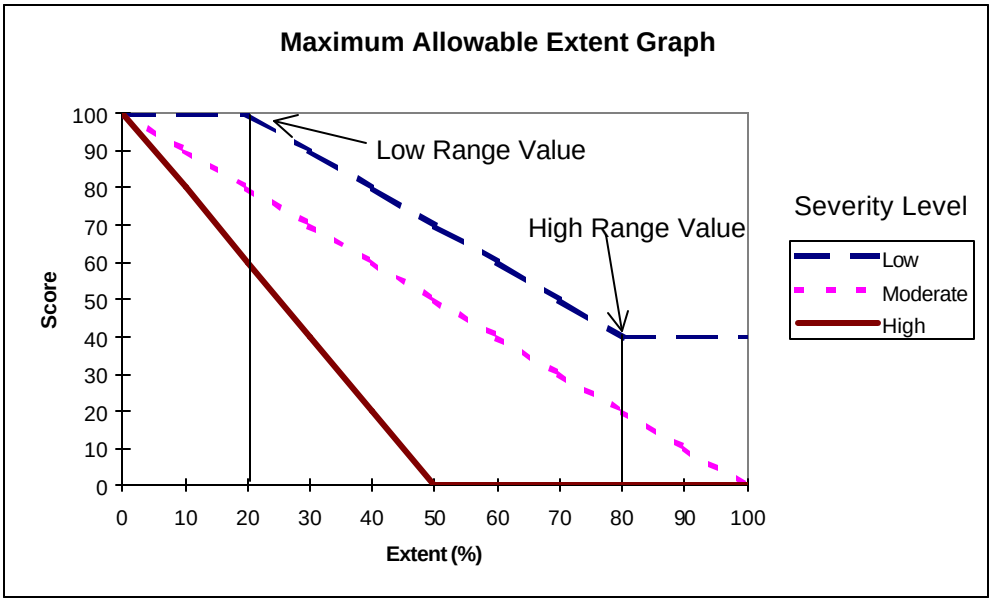


Figure 4.15 Maximum Allowable Extent Setup

To calculate the distress score the extent values for each severity level of distress that is present is used to produce an individual severity level distress score based upon the maximum allowable extent setup shown in Figure 4.15. Then these severity level specific scores are combined using an iterative process that results in a composite score, which is:

1. Equal to the individual severity score when there is only one severity of the distress surveyed, and
2. A dampened additive combination of the individual severity scores where the result is limited to the highest single severity minimum score where there is actual distress surveyed for that severity.

Calculation of Performance Indexes

Performance Indices (PI) is the only pavement condition indicators used in performance analyses and network analyses. PIs are indices that combine distress scores through the use of user defined weights.

A pavement performance index in Visual/PMS™ is a calculated index, on a scale from 0 to 100 (100 being perfect) that represents the pavement's condition at the time that distress measurements were made on the pavement section. The performance index can combine many different types of condition scores for the pavement. Many performance indices combine types of distress or condition measures that relate to the pavement's condition with respect to a given type of deterioration. For instance a fatigue index might include just fatigue cracking while a load index might include: fatigue cracking, rutting and base failures. Also some performance indexes combine all distress types into a single measure commonly called an Overall Pavement Index (OPI). In Visual/PMS™ the pavement performance indexes are defined by specifying the following:

1. The name of the index
2. The types of distress scores to be included in the index
3. The weight (emphasis) each measure has on the index (zero means not included)

To define an index, each measure included in the index is assigned an importance or weight; the larger the number, the more weight that distress will have with the index. The typical recommended range for PI weights are values between 0 and 1. For example, an Overall Index might include fatigue cracking and rutting, which are signs of a weak or under designed pavement structure, and would likely be assigned large weight(s) in the overall index, say

1.0 each. Other distress, *while important for other reasons*, may not have the same weight as load associated distress in the overall index and therefore their weight in the overall index might be lower. For example, bleeding may be given an overall index weight of 0.25. The following equation is used to combine distress scores into performance indexes.

$$PI_k = 100\prod_{i=1}^{n}\left[1-\left(1-\frac{DS_i}{100}\right)w_{i,k}\right]$$

for K = 1 to 10

where:

PI_k is the kth performance index
$w_{i,k}$ is the relative weight of the distress or condition
DS_i is the *distress scores* for the condition measure *i*.
i^{th} distress out of a total number of "n" distresses

Visual/PMS™ supports up to 10 PI calculations that can be established to evaluate different criteria or combinations of distress. For example, a load PI could be developed that is influenced equally by fatigue cracking and rutting; w_i = 0.50 for each. Similarly, if failures were to be included as a minor influence on the fatigue PI, the equation could be developed with the following weights; fatigue = 0.45, rutting = 0.45 and failures = 0.10. The Performance Indexes should be developed with the intent of evaluating the influence of different distress types on pavement section performance, mixing distress into a PI not commonly seen together on a pavement section can result in unreliable models and strange results in network analysis.

PIs are used in performance modeling as the independent modeling variables. For each performance group of sections, defined as all sections within the same grouping variables, i.e. traffic, subgrade, facility grouping classification, etc., a PI deterioration model is calculated as PI_i = f(years) using a regression equation for the ith PI. A probabilistic deterioration model in the form a Transition Probability Matrix or TPM is also formulated which defines a section's probability to stay at the same condition level or move to the next worse condition level. In Visual/PMS™ there are five condition levels called condition states. Condition levels calculated for the Markov TPM models use the condition state definitions established by the user.

Road Structure Categories

The user can define several different road structure categories (RSC) based upon the type of work performed on the pavement network. The RSC should be defined in such a way as to describe a level of effort to construct/reconstruct or rehabilitate a pavement rather than describe a strategy in detail. The level of effort can often be associated with the expected structural improvement. Remember that the RSC is used to group like pavement management sections together for the purposes of defining pavement performance groups.

In the PennDOT implementation, there are 22 RSC's defined for modeling purposes. The categories are all used to subdivide past construction history data (typical sections) into performance modeling groups.

Specifying Performance Classes

Performance classes are an integral part of the Visual/PMS™ performance analysis module. Performance classes control pavement data segregation into homogeneous groups for performance modeling. This segregation allows the user to take into account factors such as varying levels of traffic, different pavement subgrades, and other characteristics that the user expects to affect the pavement performance.

Caution is recommended when choosing to include a performance class. First, by subdividing the performance classes criteria into many pieces, the number of data points available for modeling will decrease, thus increasing chances for inadequate data to develop good models. Second, as the number of performance classes is increased the number of models used in the system and maintained by the user increases. In the PennDOT implementation the maximum number of models that could be created by selecting all available performance class variables, and

defining ten different performance indexes is 155,520. Obviously, careful consideration and planning is necessary before defining the final model set grouping.

Model Building and Review

The modeling module in Visual/PMS™ allows the user to plot the raw data along with the models built by the regression modeling module, analyze data with outliers, and select the best model. In some groups, for some performance indexes, the models built from the available data may not be realistic. This module provides a practical tool for the user to determine whether the models can be used or adjusted, in addition to a test of statistical significance. In some cases, models cannot be obtained due to lack of data. From a network M&R planning point of view, models for some groups may be desired. In these cases, the models can be made subjectively based on the available models with similar modeling criteria to and supplemented with engineering judgment.

It is important that a performance model should meet "a priori" condition. If a performance model cannot meet a priori condition, it should be deleted from the model set. The "a priori" conditions for a performance model outlined by researchers can be used to evaluate the usability of a model. The most important "a priori" conditions for the models built by this program are:

1. The initial value of a model should be less than or equal to 100 (remember that the excellent condition for all performance indexes is 100).
2. The slope of a model should be negative. For those with positive slopes, "No model" indications are presented.
3. The final value should be greater than or equal to zero.

Deterministic Models in the Performance Modeling Module

Table 4.8 shows the model forms and transformations used for deterministic models. A total of five forms are included. The user can calculate and select from each of the regression equations to choose the best deterministic model.

Table 4.8 Forms of Deterministic Models

No.	Model Form	Transformation
1	$y = a + bx$	None
2	$y = 100 - a(e)^{bx}$	$y' = \ln(100 - y)$
3	$y = a(e)^{-bx}$	$y' = \ln(y)$
4	$y = 1 / (a + bx)$	$y' = 1 / y$

where: y = Performance Index being predicted
x = Age (years)
a, b = Regression coefficients

The steps for building deterministic models are:

1. Prepare a set of data of Y (condition index) and X (pavement age) from the performance master file;
2. Transform Y (for Model Forms 2 through 4);
3. Run regression analysis without removing any data points;
4. Identify outliers (all $(Y_i - \ddot{y})/ SD \geq 3$, i = 1, 2, ..., SD = standard deviation);
5. Remove all outliers from the data set and run regression analysis again, if desired.

Probabilistic (Markov) Performance Models

Probabilistic models are presented in a matrix form. Each cell of the matrix represents a transition probability or the probability that a pavement section will transition from one condition state to another in a given amount of time, usually one year. A typical Markov TPM (transition probability matrix) is shown below in Table 4.9.

Table 4.9 Example TPM

	Condition Level After 1 Year				
Existing Condition Level	Very Good	Good	Fair	Poor	Very Poor
Very Good	0.9	0.10			
Good		0.75	0.25		
Fair			0.75	0.25	
Poor				0.70	0.30
Very Poor					1.0

Each cell in this matrix contains the probability value that a pavement in the condition state level in the first column of the matrix will transition to the condition state in the top row of the matrix in one year. For example, in this matrix the probability that a pavement in very good condition this year will remain in very good condition in the next year is 0.9. In addition the probability that the same pavement will be in good condition the following year is 0.1. There are some rules for determining the probability values used in the Markov transition probability matrix. Note that the shaded cells in the matrix are automatically given a zero value by the software. The assumptions that require this are:

1. A given pavement section can only transition from one state to the next lower state in the period of one year. Therefore the probability of a pavement in very good condition this year being in very poor condition the next year is always zero. Thus the cells at the top right of the transition matrix are zero and the sum of the probabilities in any row must be equal to one. This requirement states that pavement deterioration is continuous (i.e., no states are skipped) with respect to the condition states.

2. A pavement's condition cannot improve from one year to the next. Therefore, a pavement in poor condition this year has a zero percent probability that it will be in good condition the next year, so the cells in the bottom left corner of the matrix are also zero. This requirement means that pavement deterioration is strictly decreasing with increasing age.

A defined property of a transition probability matrix is that the expected (or average) amount of time a pavement will remain in a given condition state is given by the reciprocal of its probability that it will transition from that state. Using the example matrix we can calculate the expected amount of time in years that a pavement in fair condition will remain in fair condition. This is given by the reciprocal of its probability to transition from the fair condition state to the poor condition state. In our example matrix this is:

Expected time in the fair condition state = 1/(Transition probability)
= 1/0.25 = 4 years.

Because of this property of transition probability matrices we can show that there is no difference between a deterministic model and a probabilistic model, except for the format in which each is presented. Consider the deterministic model shown in Figure 4.16 below. It is a linear pavement deterioration model. Suppose, for this example, we had previously defined the condition state boundaries as shown by the dotted lines on the figure, then the condition states would be defined as follows:

Very Good (100-80), Good (80-60), Fair (60-40), Poor (40-20), and Very Poor (20-0)

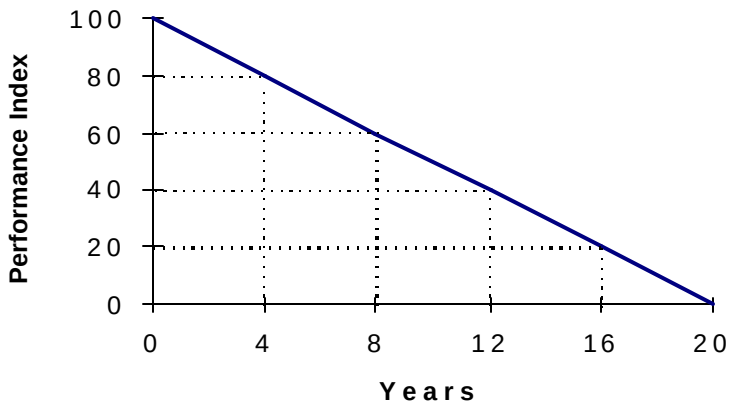


Figure 4.16 Example Deterministic Model with Condition State Boundaries

Using the deterministic model we can then determine the amount of time the pavement will remain within each condition state. For this simple example, the time within each condition state is four years (as shown by the dotted lines dropped down from each condition state boundary onto the X-axis in the graph). Therefore, using the property of transition probability matrices inversely we can solve for the probability that the pavement will transition from any of the condition states:

Transition probability = 1/(# of years within each state) = ¼ = 0.25

Then using rule #1 for transition probability matrices given above we can determine the probability of the pavement remaining in any given condition state:

Probability of Remaining in a given transition state = 1- Transition probability = 1 - 0.25 = 0.75

Then using rules 1 and 2 for transition probability matrices we can fill out the transition probability matrix for this deterministic model as shown in Table 4.10.

Table 4.10 Calculated TPM

	Condition Level after 1 Year				
Existing Condition Level	Very Good	Good	Fair	Poor	Very Poor
Very Good	0.75	0.25			
Good		0.75	0.25		
Fair			0.75	0.25	
Poor				0.75	0.25
Very Poor					1.0

This discussion shows that transition probability matrices are interchangeable with deterministic models within the Visual/PMS™ modeling module framework. Therefore, the probabilistic performance models are calculated directly from the regression based deterministic models using the method discussed here.

Building and Analyzing Pavement Performance Models

An important feature of the system is the ability of the user to create many models for all the pavements in the network. Unfortunately, in some cases these models will not be reasonable or may not reflect actual situations encountered in the field. For these cases the analysis package has been designed.

The software automatically creates many forms of performance models. These models are calculated using simple linear regression as an appropriate transform for non-linear models. Sometimes the models are found to have a positive slope (i.e., increasing condition with age) and since this is counter intuitive, these models are disallowed. In other cases a model will be calculated, but have no statistical significance. The user is cautioned not to just "accept" the default models that are produced. The software has been made so that each model can be reviewed, edited, and accepted before it is used.
The following sections discuss how to review models. This section shows the tools available to PennDOT for model review and provides some guidelines for using those tools effectively.

Analyzing Models

The analyze models screen shown in Figure 4.17 has several parts providing for user interaction and data manipulation. At the top left are text boxes that show what model is currently under analysis based upon Performance Classes, Performance Index and Road Structure Category. At the top center is the number of data points that are available for the analysis. Along the left side are the forms of the equations that can be examined and compared to the historic data for this performance model. At the bottom left is space for the user to enter their own equation by selecting the model form and the coefficients for the equation. The center of the screen shows the plot of condition vs. age for the selected group, category and performance index and plots of the currently selected equation forms. Along the bottom are several buttons that control commands associated with this window. These include:

- **Plot Graph with Outliers** to see the influence of outliers on one selected equation form at a time. When this command is executed, the outlier data point(s) will be color coded and two plots of the equations will be drawn.
- **Plot Selected Graphs** to review only the equation forms they wish to study further.
- **Select Model** opens the model selection window – see below for more details.
- **Close** returns the user to the Model Development Screen.
- **Help** activates the on-line help engine for information about the subject.
- **Print Screen** sends a picture of the window to the current default printer.

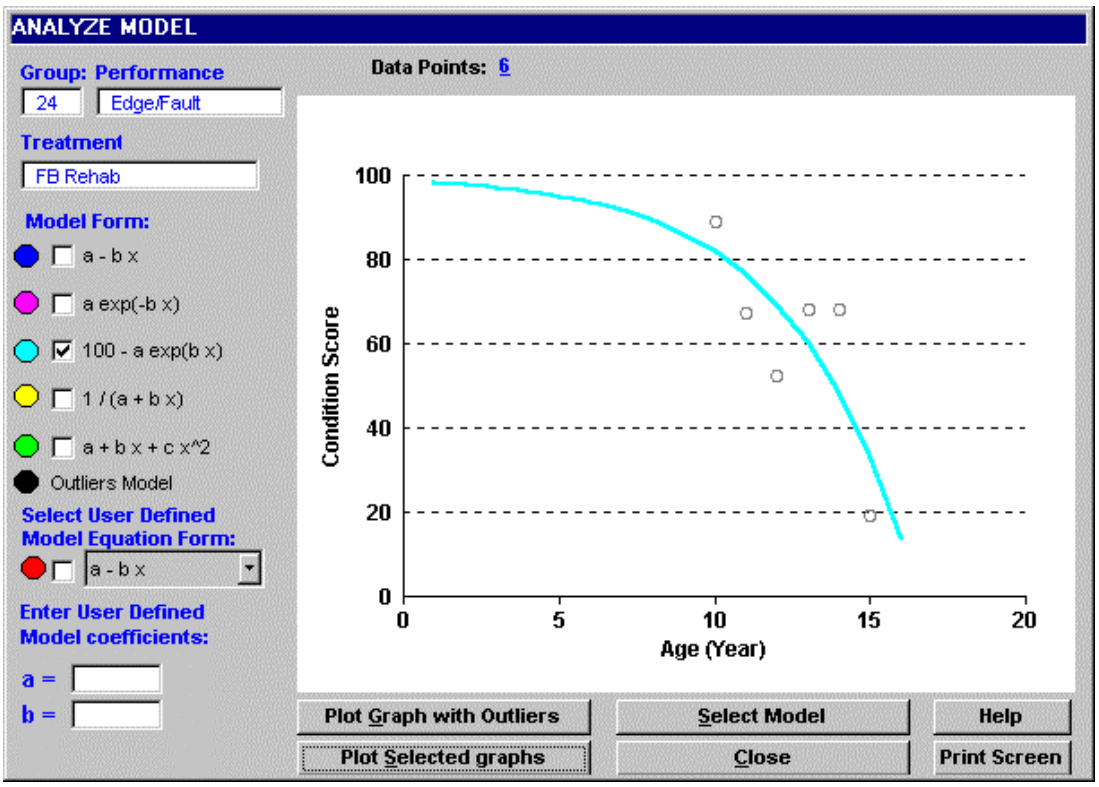


Figure 4.17 Analyze Model

To analyze a model the following procedure is followed:

1. The user clicks the "Analyze Model" command from the Action Menu and the window shown in Figure 4.17 appears.
2. Initially this window shows the graphs of all available models.
3. The user selects the model form(s) desired and clicks the "Plot Selected Graphs" button to see them.
4. Typically, the user will then note the mo del form, which they will use in further analysis.
5. The user will then plot this model by itself by deselecting the other forms in the "model form" box and clicking on the "plot graph with outliers" button. This will plot the selected graph and calculate two models:
   - The original model without outliers removed from the regression, and
   - Another model with any data points identified as outliers removed. An outlier is a data point that is more than three standard deviations away from the regression line.
6. Next the user chooses between any of the models by clicking on the "Select Model" button. In the resulting dialog box, shown in Figure 4.18, the user selects the desired model. It will be copied into the model development windows and the model selection window will close.

MODEL SELECTION

Model Form	Outliers Removed?	Model Equation	Record Number	Outliers Number	St. Error	R^2	Model Status
a + b x	No	No model	3	0	0	0	
a + b x	Yes	No model	0	0	0	0	
a exp(b x)	No	No model	3	0	0	0	
a exp(b x)	Yes	No model	0	0	0	0	
100 - a exp(b x)	No	No model	3	0	0	0	
100 - a exp(b x)	Yes	No model	0	0	0	0	
1/(a + b x)	No	No model	3	0	0	0	
1/(a + b x)	Yes	No model	0	0	0	0	
c + bx + ax^2	No	No model	3	0	0	0	
c + bx + ax^2	Yes	No model	0	0	0	0	
User Defined	No	No Model	0		0	0	0

Ok Cancel

Figure 4.18 Model Selection Window

Evaluating Models with Visual/PMS™

Models calculated by Visual/PMS™ may or may not be the best fit to the data considering the variety of available models. In any case it will be necessary to examine the models produced for both statistical soundness and reasonable results. To ensure that the models created are the best possible, PennDOT uses the following guidelines for analyzing pavement performance models produced by Visual/PMS™.

First, they examine the output from the performance model calculations as shown in Figure 4.18. Each model can fall into one of three categories, where each category is analyzed in a particular fashion.

- The first category is that the program produced no default model. This means that the least squares regression calculated a positive slope for the age variable. This is an unreasonable result as it says that the pavements in that group will improve with age. Therefore, some type of judgment is made by the engineer in assigning a model to this group. It is always best to allow the data to dictate which model is best for predicting performance. Therefore, if no default model is found, all the models available within this group are immediately examined. If a satisfactory model is found, then the pavement group is no longer part of the first category and should be analyzed as part of the next category. If no acceptable models are found, then the analyst assigns a user-defined model to that pavement group.

- The next category that a particular model may fall into is a model that has a negative slope but is not statistically significant. In this case the calculated slope of the model is not significantly different from zero. This implies that the model is not very robust. Therefore, the analyst reviews the calculated model to see if the model is in fact reasonable. The deterioration rate is compared to local engineering experience to see if the model will suffice. If not, then a user-defined model is supplied if there is enough experience with that group of pavements to justify changing the model.

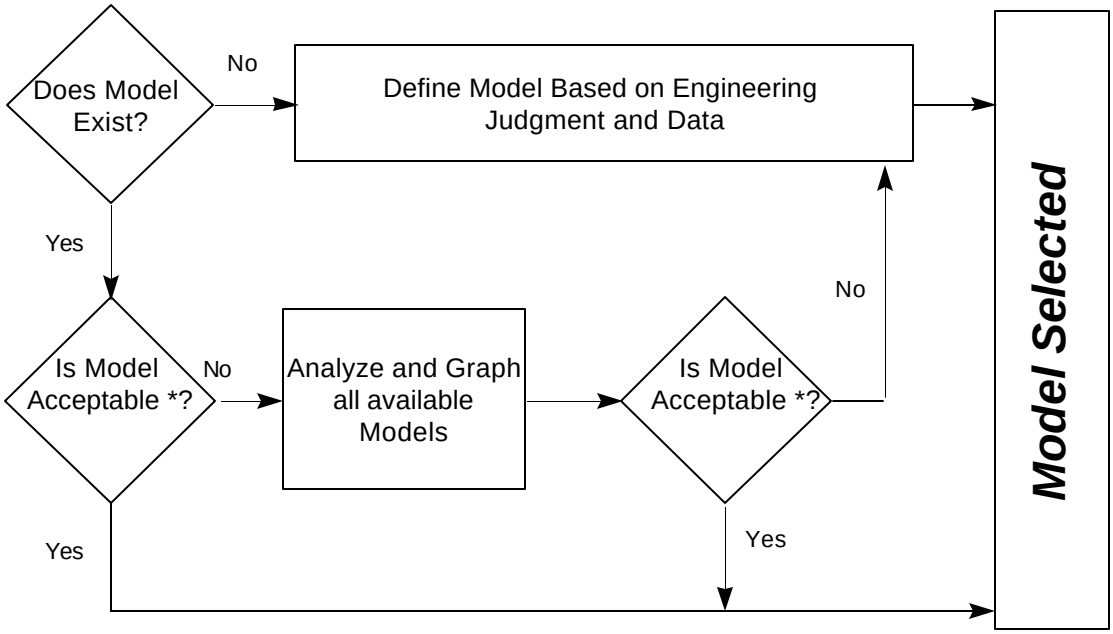
- The third group of models that can exist are models that are statistically significant. This means that the regression line calculated from the data was found to have a negative slope that is significantly different from zero. Generally, these models are accepted as is. It is possible, however, to find other nonlinear models that fit the data better and are still significant. These other models are usually explored. When the best model has been found, it is necessary to determine if the model is acceptable.

Shown below in Table 4.11 is a list of criteria that are used to determine if a significant model is acceptable. These criteria are listed as an approximation only, and the relative weight given to each of these criteria is determined by the engineer. It is important to realize that a model, whether significant or not, should always be examined closely before it is discarded for any reason. Models that at first seem unreasonable may actually be showing a trend in pavement performance that has gone unrecognized.

A simple flow chart for analyzing performance models is shown in Figure 4.19. Using the method shown should produce reasonable models. Generally the engineer should use their best judgment to determine if a model is acceptable.

Table 4.11 Criteria for Determining Model Acceptability

Criteria	Measurement of the Criteria for Acceptance
Reasonable Deterioration Model	This should be based on engineering judgment. A model should not be discarded if its results seem reasonable. The best alternative is always to use models created from historical data. Unfortunately, this can sometimes conflict with local experience.
Statistically Sound Model	**USUALLY SHOWN BY SIGNIFICANCE OF THE SLOPE PARAMETER CALCULATED FROM THE REGRESSION. IF THE MODEL IS STATISTICALLY SIGNIFICANT, IT SHOULD BE CONSIDERED CAREFULLY BEFORE IT IS DISCARDED.**
Reasonable Amount of Variation Explained	This is measured with the R^2 statistic that is produced with each model. It is possible to have statistically significant models that have extremely low R^2 values. In some cases, especially when the model is calculated from a large number of data points, this does not mean the model is not acceptable



* See Guidelines to Model Acceptability Table.

Figure 4.19 Determining Acceptable Models

Performance Modeling Example

As an example we will build a performance model for a particular type of pavement using the PennDOT historical condition database. The model will be built for an Overall Pavement Condition Index for the Road Structure Category and group information listed in Table 4.12. For this example we will investigate the performance of FB rehabilitation pavements for High Traffic NHS Urban Roadways. There are eighteen available data points in this performance class for investigation and model building. The initial plot of the data along with all the available model forms is shown in Figure 4.20.

At this point the engineer can then view some statistics about each model equation to aid in selecting the best model form. The window showing this information is in Figure 4.21.

Table 4.12 Example Model Parameters

Overall Pavement Index Definition	
Distress	*Relative Weight (Scale of 0-1)*
Profile Distortion	0.04
Rutting	0.09
Ride	1
Raveling/Weathering	0.06
Block Cracking	0.12
Transverse/Longitudinal Cracking	0.14
Alligator Cracking	0.23
Edge Deterioration	0.17
Bituminous Patching	0.17
Potholes	0.15
Widening Dropoff	0.06
Grouping Information	
Variable	*Levels*
AADT	Low (AADT<=750), Medium (750<AADT<=3250), High (AADT > 3250)
Functional Class	Interstate Urban, Interstate Rural, NHS Urban, NHS Rural, PAS (Principal Arterial System) Urban, PAS Rural, NonPAS Urban, NonPAS Rural
Road Structure Category	
FB Rehabilitation	Overlays of Existing Asphalt Concrete with Bituminous Wearing Course FB as specified by Pennsylvania Mix Design Requirements.

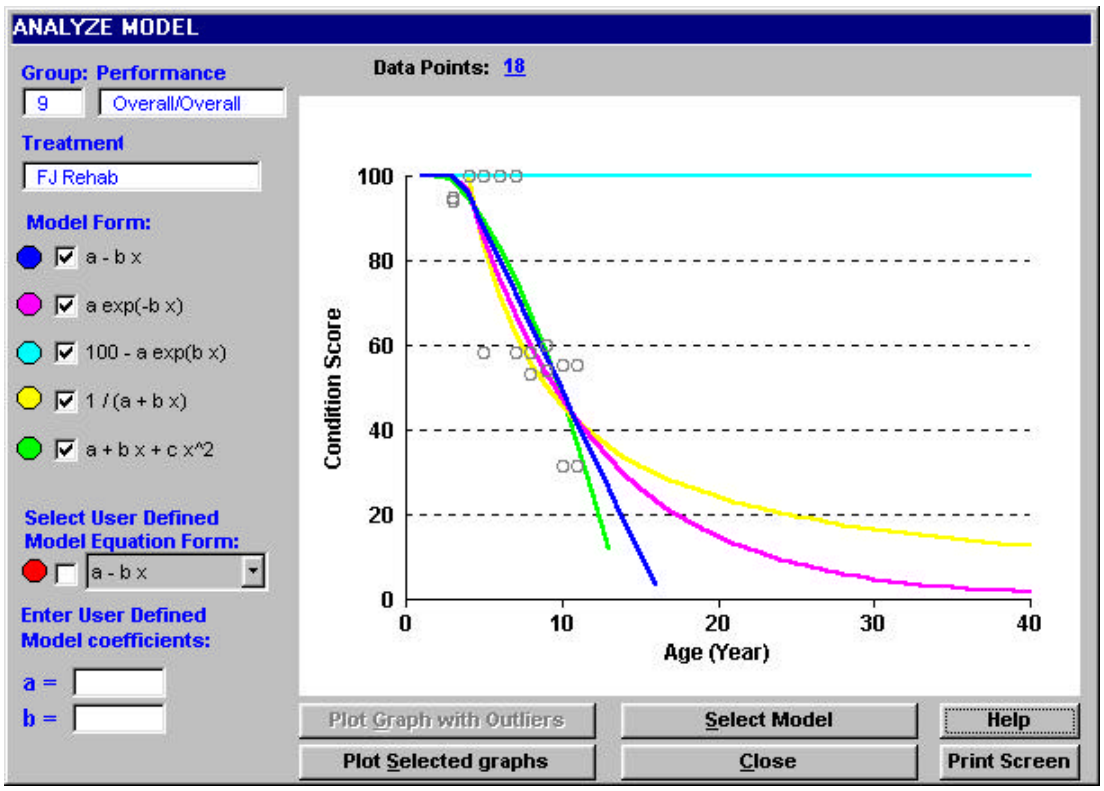
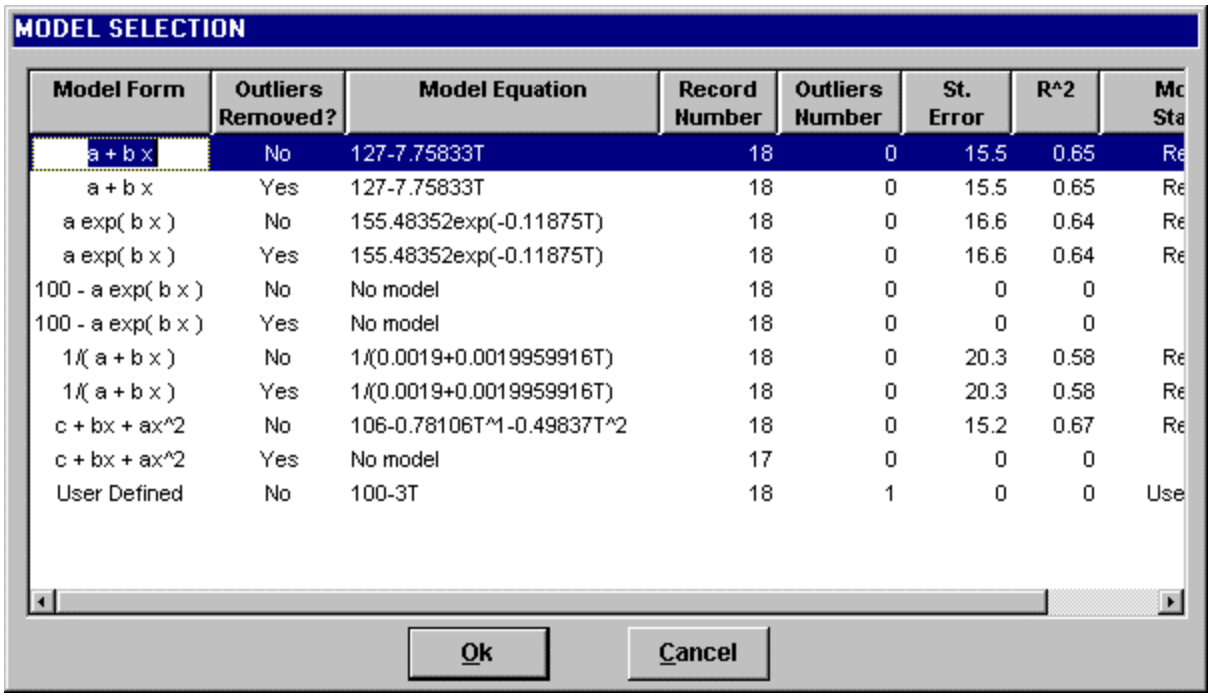


Figure 4.20 Plot of all Models for our Example



MODEL SELECTION

Model Form	Outliers Removed?	Model Equation	Record Number	Outliers Number	St. Error	R^2	Mo Sta
a + b x	No	127-7.75833T	18	0	15.5	0.65	Re
a + b x	Yes	127-7.75833T	18	0	15.5	0.65	Re
a exp(b x)	No	155.48352exp(-0.11875T)	18	0	16.6	0.64	Re
a exp(b x)	Yes	155.48352exp(-0.11875T)	18	0	16.6	0.64	Re
100 - a exp(b x)	No	No model	18	0	0	0	
100 - a exp(b x)	Yes	No model	18	0	0	0	
1/(a + b x)	No	1/(0.0019+0.0019959916T)	18	0	20.3	0.58	Re
1/(a + b x)	Yes	1/(0.0019+0.0019959916T)	18	0	20.3	0.58	Re
c + bx + ax^2	No	106-0.78106T^1-0.49837T^2	18	0	15.2	0.67	Re
c + bx + ax^2	Yes	No model	17	0	0	0	
User Defined	No	100-3T	18	1	0	0	Use

Ok
Cancel

Figure 4.21 Model Statistics Window

In this window we can see that the polynomial model (highlighted in Figure 4.21) has the largest R^2 value and the lowest standard deviation. This indicates that it is the best fit to the data. This model is shown by itself in Figure 4.22.

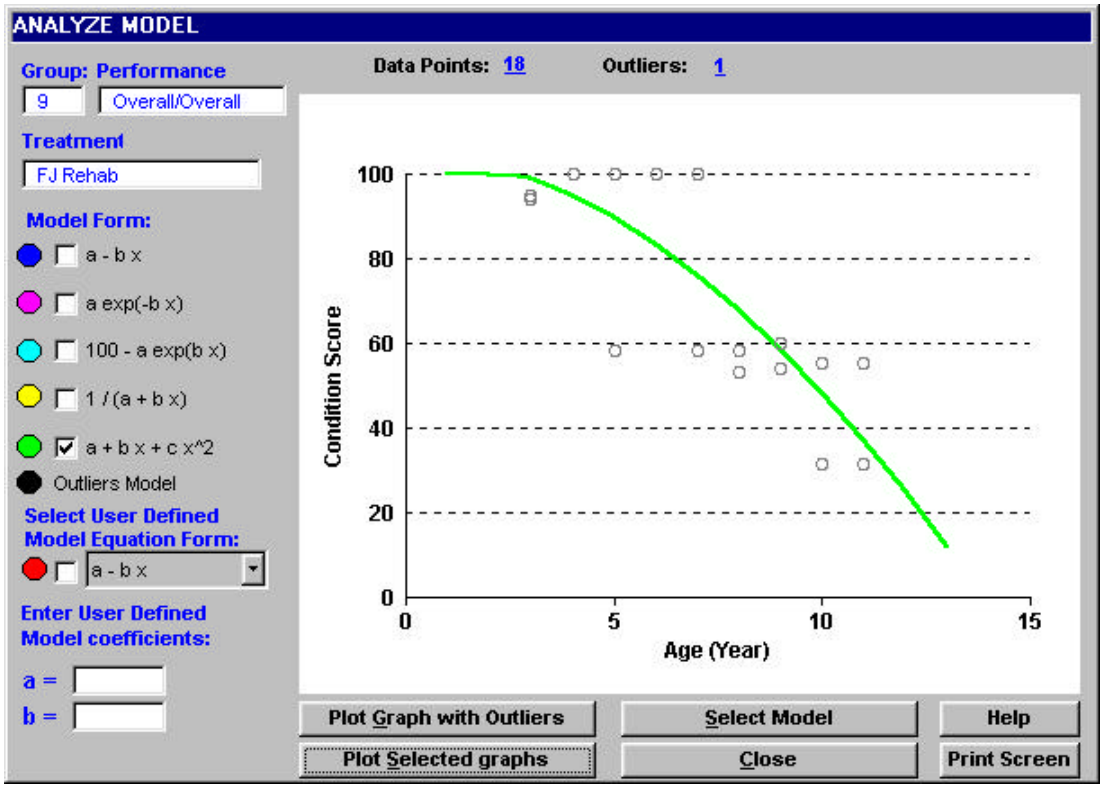


Figure 4.22 Polynomial Model Plotted Against the Data

This model can then be selected by the analyst to predict performance for analysis in the Visual/PMS™ software. The PennDOT pavement engineering staff is currently reviewing and building models from their database for use in network and project analyses. The software helps the engineers to review their data and produce accurate models.

Outlier Analysis

In addition to building models from existing data the engineer can also identify suspect data points and adjust the models accordingly. In Figure 4.23 a model is shown that contains some data points that are more than three standard deviations off the regression line.

In this case it was determined that the outlying data points were sections that had received an overlay but the construction information was not recorded in the database. The points were removed from the analysis and the model adjusted accordingly. In Figure 4.24, the effect of removing these points can be seen. The R^2 for the model approximately doubled and the standard error of prediction was reduced by approximately 25%. This is a significant improvement in the model.

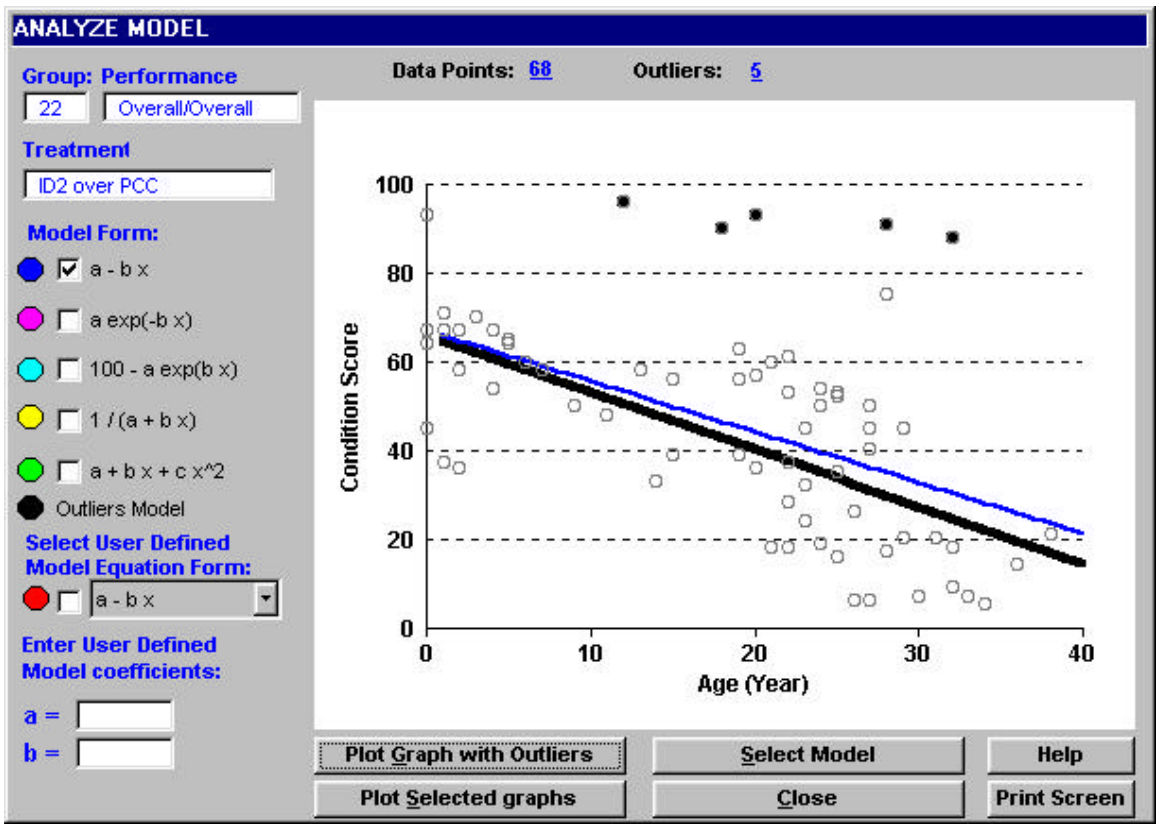
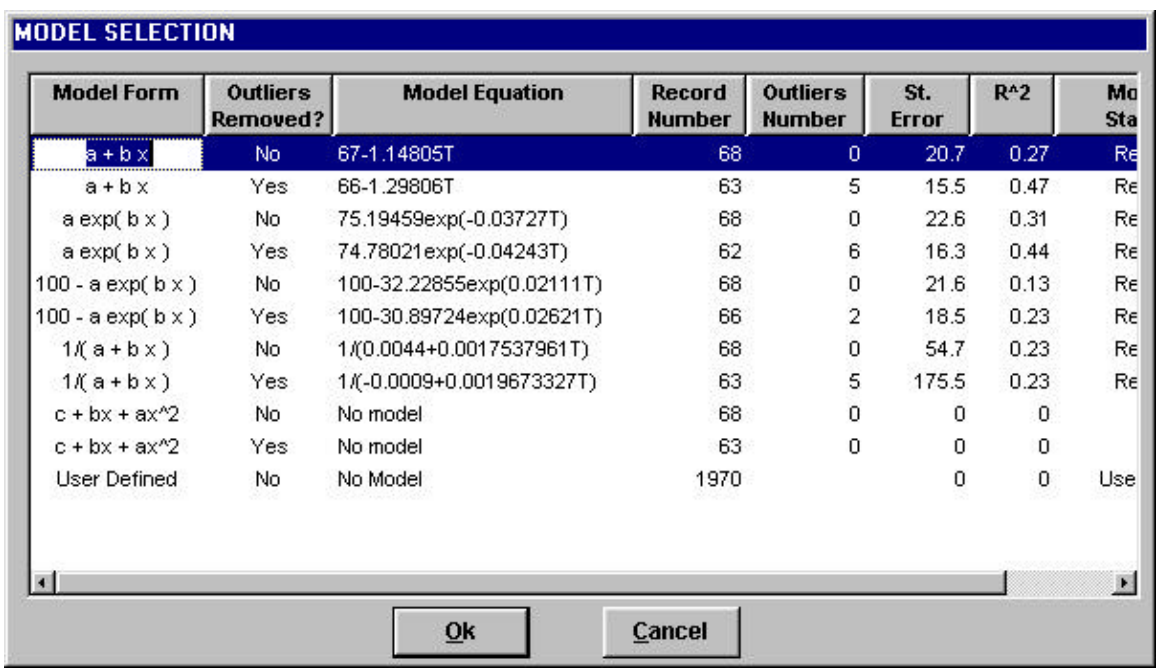


Figure 4.23 Simple Linear Model with Outlying Data Points Identified



MODEL SELECTION

Model Form	Outliers Removed?	Model Equation	Record Number	Outliers Number	St. Error	R^2	Mo Sta
a + b x	No	67-1.14805T	68	0	20.7	0.27	Re
a + b x	Yes	66-1.29806T	63	5	15.5	0.47	Re
a exp(b x)	No	75.19459exp(-0.03727T)	68	0	22.6	0.31	Re
a exp(b x)	Yes	74.78021exp(-0.04243T)	62	6	16.3	0.44	Re
100 - a exp(b x)	No	100-32.22855exp(0.02111T)	68	0	21.6	0.13	Re
100 - a exp(b x)	Yes	100-30.89724exp(0.02621T)	66	2	18.5	0.23	Re
1/(a + b x)	No	1/(0.0044+0.0017537961T)	68	0	54.7	0.23	Re
1/(a + b x)	Yes	1/(-0.0009+0.0019673327T)	63	5	175.5	0.23	Re
c + bx + ax^2	No	No model	68	0	0	0	
c + bx + ax^2	Yes	No model	63	0	0	0	
User Defined	No	No Model	1970		0	0	Use

Ok
Cancel

Figure 4.24 List of Models for the Outlier Analysis Example (note the first two models in the list are shown in Figure 4.12)

Using this type of analysis the engineer can not only produce accurate and reasonable models but also identify and correct suspect or incomplete data within the database. The data points in this example have been subsequently corrected with the missing construction history and are now actually a part of another performance class.

Engineering Input

In some cases models must be built using a considerable amount of engineering judgment and input into the modeling process. Figure 4.25 shows an example where the data set available for model analysis is not sufficient to determine which model form is best suited for prediction. In this case each model fits the data with high R^2 values and relatively low standard errors, however there is only data available for the first few years of the pavement life cycle. As shown the prediction equations differ significantly outside the range of data. At this point in the process the engineer must determine which model is the most appropriate for use in planning based upon engineering theory and local experience. It is extremely important that the engineers review and accept the models used for planning so that they reflect the actual deterioration found in the field. As PennDOT collects more data for these pavements the software can assist the engineers to update and review this model each year and adjust the equations to reflect the updated data.

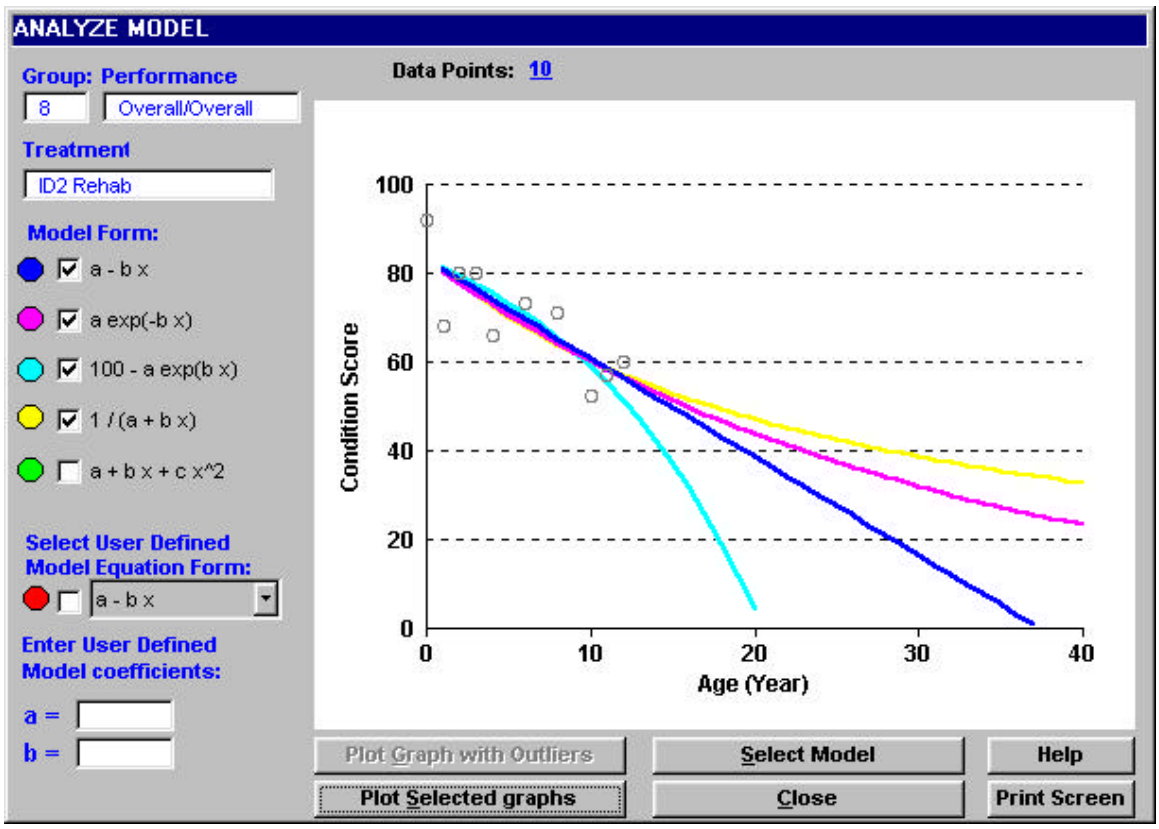


Figure 4.25 Data Set Requiring Engineering Judgment to Produce Good Models

4.8.2 WSDOT – Pavement Performance Modeling

<u>WSDOT's PMS Overview</u>

In the mid 1960s to satisfy legislative requirements for a priority programming process Washington State Department of Transportation (WSDOT) developed a pavement condition survey program as part of a pavement management system. A pavement condition survey has been conducted on 100% of the state highway system every

two years since 1969 and every year since 1988. In the late 1970s the first version of the Washington State Pavement Management System (WSPMS) was developed. In 1988 the WSPMS computer programs were completely revised to fit a new mainframe file system, to take advantage of personal computers, and local area networks.

A major update of the WSPMS was implemented again in 1993, which included a new Pavement Structural Condition Index (PSC) and a new project scoping technique for programming projects. In addition many graphical displays of data were provided, in a user-friendly environment, to make the system data much easier to view and understand [Kay 93].

The project scoping technique is discussed in detail in Module 6. The WSPMS estimates rehabilitation designs for all selected projects based on the condition of the pavement, the pavement structure and subgrade, and the projected traffic. It is a simple overlay thickness design tool built into the program. At the time of project programming the district can correct pavement structural errors found in this data file, to insure that designs will be based on valid pavement structure. Regional personnel estimate that 75 to 80% of all the scoping designs agree with the final project design. Thus, at contracting time there will be adequate funds available for building the rehabilitation or overlay.

WSPMS Structure

In the late 1970's, the WSPMS structure was developed within the WSDOT material laboratory and subsequently implemented during the 1982 project programming cycle. As the WSPMS was developed in-house, WSDOT personnel have had the capability to upgrade and modify the software. In 1988, WSPMS computer programs were completely revised to fit newly developed mainframe file systems, to take full advantage of personal computers and a local area network, and to better automate the WSPMS process. In 1995, the PMS programs were again recorded to take advantage of the Microsoft Windows environment and to convert the data files into the ACCESS database. [Pierce 96]

There are four basic components of the WSPMS:

1. File building
2. Interpreting program
3. Project selection analysis
4. Network level analysis

These components are summarized in the following sections.

File Building

The WSPMS includes several large databases. Information captured includes location referencing, roadway configurations, pavement construction history, traffic data, and planned construction schedules. Other PMS-specific data such as friction, ride, and surface distress defects are gathered and maintained by pavement management staff using PCs on a local area network with direct access to the Department's mainframe computer. Several data files are maintained as part of the WSPMS database.

The "Schematic File" is used to validate the existence of roadway location and serves as the reference for all milepost computations as well as locating and documenting parallel lanes and other highway configurations.

The "Analysis File" contains 11,000 records with details about each highway segment. Each record contains jurisdictional information, roadway configuration, pavement type, and thickness history for the pavement layers.

The "Construction Program File" accesses the mainframe databases and identifies planned projects for the next biennium (approved legislative projects).

A "Multiyear Survey File" contains the pavement condition survey data in its raw form from all surveys. The file currently contains 19 separate surveys dating back to 1969. As new surveys are completed they are added to the file.

The "PMS Projects File" keep track of beginning and ending mile points obtained from all the other files. This file is used for prioritizing and other references to project specific information.

Interpreting Program

The interpreting program takes the large arrays of data stored in the various data files and develops project specific information related to more than 3,200 sections statewide. This program calculates pavement rating scores for prior and current condition rating schemes. For the data prior to 1993 the program calculates a combined Pavement Condition Ratings Index (PCR), which included rutting and structural information. For the current data, the distresses are separated out such that a Pavement Structural Condition Index (PSC) is calculated along with functional indexes based on rutting (PRC) and roughness PPC (IRI).

PSC is based on fatigue cracking for flexible pavements and slab cracking for rigid pavements. Flexible pavements in WSDOT are placed in the rehabilitation program largely due to fatigue cracking. Few if any of the roads on the state highway system are so rough or rutted that they take priority over the cracked pavements. Any pavement distress that is time dependent, such as rutting, cracking, etc., could be used to establish a PSC index. With continued improvement in roughness and rut depth measuring equipment, WSDOT is looking toward better prediction of roughness and rutting in HMA and faulting in PCCP. The PMS programs will be modified to account for these in the future.

Project Level Analysis (Project Selection)

In the original WSPMS version, the project level optimizing program determined the optimum rehabilitation strategy for each specific project developed in the interpreting program. The optimizing program computed life cycle costs for an array of rehabilitation strategies for each project and ranked the output in order of life cycle costs for each project. Although this program worked reasonably well, it did not have much effect on WSDOT's decision-making process due to other overriding factors. With the existing priority array process and a reasonably funded rehabilitation program, projects have been identified early in the pavement deterioration cycle so that most of the rehabilitation needs are relatively thin overlays, exactly what the optimizing program recommends. Where thicker structural overlays are indicated, additional design information is needed to analyze the project. At the present time, the project level analysis is accomplished by considerable interaction between the district program development offices, the district materials staff, and the Headquarters PMS office.

Through extensive PMS experience, it has become quite clear that the process is greatly enhanced by making as much project level information as possible available to all parties during the programming period. To this end a user-friendly software package along with all of the PMS files is supplied to WSDOT managers and designers for day-to-day reference. [Kay 95]

Network Level Analysis

Network analysis has always been performed as a natural extension of the project selection analysis in WSPMS. WSDOT has been working towards network analysis processes that help optimize project selection within each region to deliver the best overall pavement condition over time for fixed funding levels. WSDOT's project selection system works well enough that there is very little disagreement between the region and WSDOT Headquarters as to project definition.

A scenario analysis allows pavement managers to predict future conditions under various circumstances. "What if?" scenarios can be run into the future years given changes in mileage, budget and other constraints. The results of these runs are converted to graphics that can be easily understood by people not familiar with the complexity of pavement strategies. Special studies and research efforts use these features to demonstrate what different strategies will accomplish.

WSPMS Performance Equations

Figure 4.26 shows typical forms for the WSPMS performance equations. These equations may be obtained by three methods.

1. A regression equation may be fit through the data for the individual pavement where the section has a number of years of known performance.
2. Defaults equations may be used that were developed previously from overall performance of similar pavements in the particular region of the state when sufficient survey data is available (such as pavements less than three years old).

Equations may be developed based on adjustments made using engineering judgment.

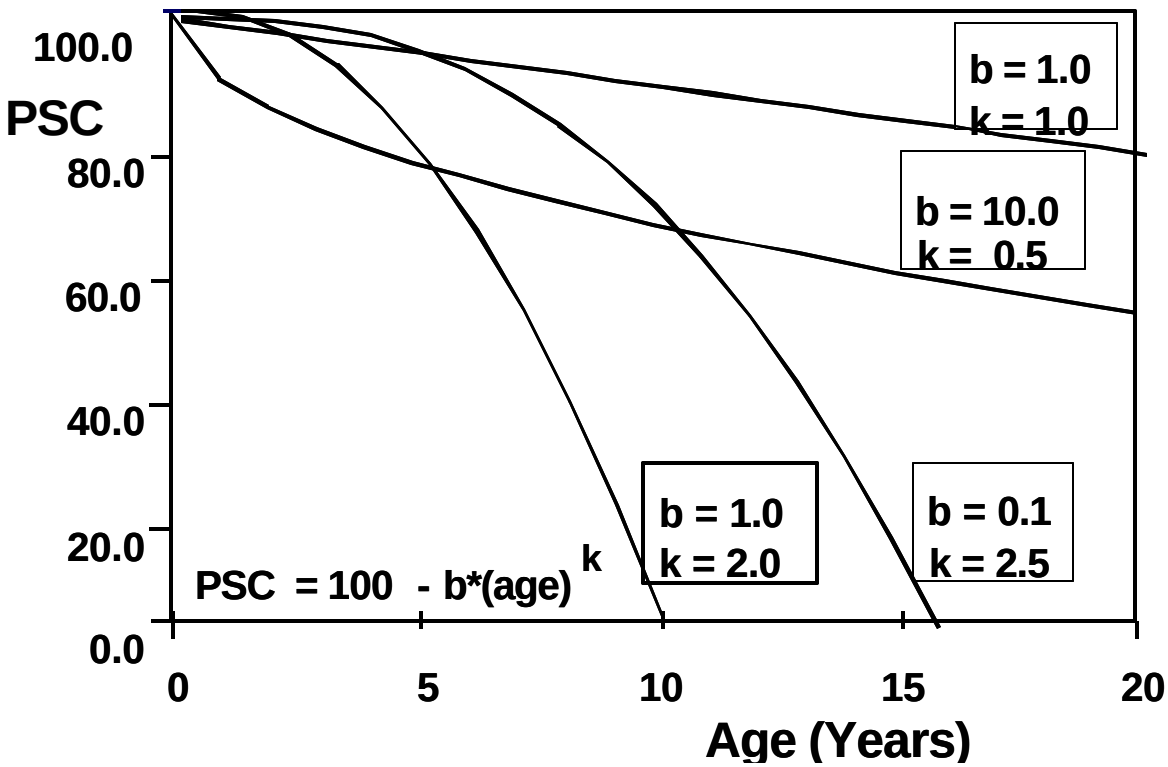


Figure 4.26 Examples of Performance Model Curve Shapes [Kay 93]

Other Engineering Applications of PMS

Use of PMS Data to Evaluate Asphalt Recycling

In 1988 Washington State DOT undertook a study to investigate the effects of aging asphalt pavements [Babaei 89]. Review of the pavement management database showed continued deterioration and need to rehabilitate. In view of these observations WSDOT began to examine the PMS data for possible clues to improvement. A potentially very cost effective method of rehabilitation was full depth cold-mix recycling and comparing the results of treated sections placed in 1982-83 on the state highway system. This was an important engineering research study. The PMS data assisted in at least three ways in the study.

1. The PMS data helped identify the problem of increasing deterioration in an aging pavement network.
2. The PMS data provided a repository for performance data and made it easier to define the test section detail given in Table 4.13 for further study.

3. The PMS database provides the necessary mechanism for follow-up on these test sections until the end of their useful life and thus reinforce or modify the finding found after seven years of observation.

The significant aspects and findings of this study are reproduced below for detailed information.

Recycling Overview

The WSDOT is investigating methods of upgrading aging pavements in the existing network of highways in the state. The majority of those highways were built with hot-mix asphalt (HMA) pavement. Often upgrading an aged HMA involves overlaying a cracked pavement with a new HMA. However, the substrate cracks often reflect through the new overlay under repeated service loading, especially when the overlay is less than one inch thick. Full-depth cold-mix in-place recycled (CIR) asphalt concrete pavement is one method to eliminate this reflective cracking. The operation is conducted at the construction site and involves milling and/or crushing the old HMA, mixing the material with an asphalt emulsion, and finally placing and compacting the cold mix. CIR is then overlaid with a new HMA. This process eliminates the existing crack pattern and the potential for reflective cracking. The process of recycling conserves natural resources and, in some cases, reduces the construction cost.

Nevertheless, the cold-mix recycling of HMA has not yet proven to be totally reliable and durable and its implementation is in its infancy. Since 1981, WSDOT has constructed three experimental cold-mix recycled projects. The further use of cold-mix recycling by WSDOT hinges on the success of those three experimental projects.

Table 4.13 Summarized Background Information on Three Washington Cold-Mix Recycled Asphalt Concrete Projects [Babaei 89]

Average property \ Project	Chewelah	County Well	Brewster
Old Pavement			
Year Built	1962	1979	1950
Type	Class "B" HMA	BST	Army Corps HMA
Thickness (in.)	3.4	1.2	3.2
Binder content (%)	5.0	4.8	4.9
Year Recycled	1982	1982	1983
Binder viscosity @ 140°F when recycled (poise)	154, 534	649	57,000
Recycled Pavement			
Thickness (in.) [1]	4.2	2.5	4.4
Binder content w/o emulsion. [1]	4.1	2.6	3.8
New Emulsion			
Type	ERA-75	CSS - 1h	CSS - 1
Viscosity @ 140°F, Residue (poise)	75	---	---
Penetration @ 77°F, Residue	300+	40 - 90	100 - 250
Content, target (%) [2]	3.2	5.5	3.25
Residue content, target (%) [2]	1.9	3.3	2.0
Water content	3% to 3.5% in RAP	6% limit	2% added, 6% limit

[1] "Thickness" and "binder content" represent the old asphalt pavement plus the portion of non-asphalt granular top course included in the recycling.
[2] Assuming the emulsion was distributed uniformly throughout the recycled mix.

WSDOT initiated this research project with the objective of evaluating the general performance of CIR and the particular performance of the three WSDOT CIR test sections to make recommendations on the further use of CIR in Washington. The first task of the project involved reviewing the available literature and gathering information on the performance of CIR constructed by other US highway agencies. After this task was completed, the research team visited Washington's three CIR test sections, visually inspected in 1988 about five to six years after the completion of the projects.

Findings

This section summarizes the findings of this report on the performance of cold-mix recycled asphalt concrete pavement (CIR). The findings are based on tests conducted in this work as well as data obtained from other research.

Performance

1. The performance of CIR in Washington supports the use of CIR as base course in flexible pavements. The overall conditions of the three Washington CIR projects after about six years of service varied from fair to good, depending on the level of traffic. The highest average daily traffic for those pavements was 4,750 vehicles per day. The factors that determined the overall pavement condition in this evaluation were wheel-track rutting and cracking, determined during visual surveys.

2. The wheel-track rutting of the Washington CIR projects was not significantly different from rutting that might be expected from conventional flexible pavements performing under the same conditions. The average rut depth for the CIR project with the highest level of traffic and rutting was only about ¼ in, which could be categorized as low to medium severity.

3. Full depth recycling of HMA eliminates the cause of reflective cracking. The type of cracking noticed in the Washington CIR projects was not reflective cracking, rather it was isolated wheel-track cracking that is usually the beginning of typical alligator (fatigue) cracking in flexible pavements.

Material Characteristics.

1. There are indications that a portion of the existing asphalt participates as part of the effective binder in the cold recycled mix. That participation occurs during the mixing and placing and/or during the service. The limited data available from tests conducted in the current work indicate that the amount of existing asphalt that participates in the mix depends on the amount and viscosity of the asphalt residue and the viscosity of the aged asphalt. Larger amounts of asphalt residue, lower viscosity's of the asphalt residue and lower viscosity's of the existing (aged) asphalt all contribute to more participation of the existing asphalt in the recycled mix as an effective binder. In the three Washington CIR projects, the amount of existing asphalt that participates in the mix as an effective binder was roughly estimated to be from 1.5% to 2.0% of the weight of total mix, depending on the factors discussed.

2. The total effective binder content in CIR (i.e., the binder that actually fills the voids between particles) is the amount of asphalt residue plus the portion of the existing asphalt that participates in the mix as an effective binder. The aggregate and the remainder of the old asphalt act as "black aggregate" in the recycled mix. The viscosity of the effective binder, thus, depends on the viscosity of the asphalt residue and viscosity of the existing (aged) asphalt.

3. The stability of the three Washington CIR projects could not be measured directly from their core samples. The pavement's average void contents, from 9.5% to 12% indicated that the stability may be marginal for the highest void content; however, this does pose a major problem, especially when CIR is used as base course. The low to moderate rutting of the three test pavements supported this opinion. Where some local severe rutting had occurred, test results showed the presence of extremely high asphalt content, which was due to an excessive

amount of emulsified asphalt in the CIR. This is probably due to a non-uniform distribution of the emulsion in the recycled mix.

4. The flexibility of the Washington's CIR projects was slightly lower than the flexibility of Washington's overlaid hot-mix recycled asphalt concrete pavements (CIR), but it was more noticeably lower than the flexibility of Washington's non-overlaid conventional asphalt concrete pavements, in light of their age. This means that CIR may be more susceptible to fatigue cracking, given the same load conditions and structural design. The average resilient modulus of the three Washington CIR projects was 347,000 psi after five to six years of service. This compares less favorably to the average resilient modulus of 258,000 psi obtained for three Washington overlaid CIR projects after six years. Also, it compares less favorably to the average resilient modulus of 350,000 psi obtained for 16 Washington non-overlaid conventional asphalt concrete pavements after 12 years considering the age and exposure factors. Higher resilient modulus values can be expected of asphalt mixes as they age making the pavements more brittle and susceptible to fatigue.

5. The durability of the three Washington CIR projects was either marginal or not satisfactory. This was because of the relatively high void content of the CIR mixes after five to six years of service (i.e., from 9.5% to 12%). Depending on the amount of void content, a reduction of 18% to 34% in the resilient modulus values was observed after the core samples of the CIR projects were water saturated. Note that CIR projects with void contents as small as 5% (after one year of service) have been constructed in the United States. The average void content of the three Washington overlaid HMR-HMA projects was 6% after six years of service.

Recommendations

The results of the current investigation using the PMS database indicate that full depth cold-mix in-place recycled asphalt concrete pavement is an effective method of rehabilitating aged flexible pavements to eliminate the potential for reflective cracking. Continuation of the use of CIR as the base course in flexible pavements is recommended. The mix design can be somewhat improved based on the findings of this study and the continued monitoring of CIR pavements through the WSPMS database.

The type and amount of the emulsion should be selected with the aim of incorporating the existing asphalt into the recycled mix as an effective binder to the extent that the field product of CIR will have a void content in the range of 6% to 8% after about five years of service. This will result in enhanced stability, flexibility, and durability of CIR.

Based on the limited information available, as a guideline, for aged asphalt's with viscosity's above 20,000 poise, the addition of 2% (residue percent) of emulsified asphalt rejuvenators with a 300+ penetration is suggested. For extremely hard, aged asphalt's with viscosities above 100,000 poise, the amount of the rejuvenator may be increased. For aged asphalt's with viscosities below 20,000 poise, addition of 2% (residue percent) of emulsified asphalt's with a 100 to 250 penetration seems appropriate. The use of high viscosity emulsified asphalt's with a penetration of below 100 seems to be effective with aged asphalt's with viscosities below 1,000 poise. In this case, the addition of 3% (residue percent) of emulsified asphalt is suggested.

Uses of PMS Data in Maintenance

The maintenance division within WSDOT is provided a hard copy of the annual pavement management program for the state, which presents future programs indicating the "due year" for each project. Projects for each of the coming six years for each region are listed. Based on this listing the maintenance engineers defer major maintenance on projects due for immediate overlay. For example, in 1997 projects are programmed for overlay in the year 2000 (three years), then major maintenance would not be conducted during 1997 - 1999. On the other hand if a project is "due" (scheduled) for rehabilitation in the year 2003 (six years off) it may be necessary to perform some type of maintenance during 1997 and 1999.

PMS Data to Define All Weather Roads

In 1994-96 the WSDOT materials laboratory was required to develop a recommendation for all weather roads in the state. This project required the definition of roads, which might suffer from freeze/thaw damage and to recommend a program of bringing those pavements up to all weather standards. A pavement would be classified as deficient if total pavement thickness (AC + Base) is ½ the thickness of maximum expected depth of freeze.

The pavement management system was used to evaluate the structural depths of all the pavements in the state and determine those, which had deficient frost-depth protection. A list was then developed of those pavement sections, which required thickening to fulfill the all weather requirements imposed by the state legislature. The PMS is now being used to monitor the effects of this legislation and the modifications made to the pavement sections. Early indications are that increased pavement lives and reduced maintenance costs will be the long-term benefits.

Use of PMS Data to Evaluate Bituminous Concrete Materials

The pavement design section uses data on aggregate gradation and asphalt type from the pavement management system to identify if any particular type of bituminous material has worked well and conversely to define any type of bituminous mixture that has not worked well. Performance life is examined and compared across material types to identify good and poor performers in various situations.

This is a good example of accessing the pavement management system in a straightforward way to understand materials related performance effects and how subsequent projects can be appropriately constructed. This has resulted in improved use of proper materials for certain situations and generally longer pavement lives.

Asphalt Grading and Early Implementation of SHRP Asphalt Grades

Since 1992 asphalt types used in construction were listed in the PMS. The new SHRP asphalt designations will be listed in the PMS database and performance trends will be developed using the PMS. USDOT has identified three climatic regions and three traffic levels that would account for up to nine possible binder types to be monitored. It is expected that this monitoring will produce a good understanding of the effects of the different binder types on pavement performance life.

Use of Database to Maximize Value of Engineering Test Data

WSDOT has initiated the use of a falling weight deflectometer (FWD), however at the initiation of the program it was found that the amount of FWD data was not suitable for incorporation into the PMS database for easy use. Continued examination of this process has clarified the need to reduce the FWD data at the project level, to either take less data or to summarize the data in an appropriate way to incorporate it into the PMS database. This approach will have the value of broadening and improving the network level PMS database to include improved structural data and will then improve WSDOT abilities to do design and to develop pavement performance models from their PMS data.

Evaluation of Superior Performing Pavements

An application has been used for estimating performance life of various rehabilitation treatments for pavements. The WSDOT PMS database and performance models were used to improve on the life cycle cost analysis process to incorporate actual experience. This led to a project to use the pavement management system to identify those pavements within the state that have provided superior performance. The PMS database will be used to locate such superior performing pavement sections so that they can be studied in more detail. As a result of this study, the pavement design models within the state will be improved and upgraded. This could not be done were it not for the high quality of the pavement management system database within the state and is an important engineering application.

Use of PMS Data to Increase State Level Budgets

An important use of the pavement management data in the state has been to provide additional information to the state legislature to show pavement performance trends. Because of the high quality of the data provided, the legislature has been willing and felt confident to increase budgets to provide funding for the preservation program. This process began in 1988, when it was possible for WSDOT to use the history of their pavement management deterioration curves to show the legislature that inadequate budgets were being made available for pavement preservation within the state. While not a purely engineering example, we believe this provides a closing keynote for the value of the excellent pavement management database, which has been developed in the state of Washington.

4.9 REFERENCES TO MODULE 4

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5 WORKSHOPS ON ENGINEERING APPLICATIONS OF PMS

5.1 INTRODUCTION

In order to stimulate the interest of course participants in engineering applications of PMS data, two workshops are held, one at each day of the course. The first one will concentrate on using PMS data to monitor the performance of various materials and techniques, with Superpave as an example. The second one will cover the question how to make use of maintenance data in a Pavement Management System.

5.2 WORKSHOP ISSUES

The issues that could be considered are the following:

- Review of examples covered during the day,
- Possibilities and opportunities for your DOT,
- What can your agency do now? Think of:
    - Problems you can solve now
    - New data that is needed in your state
    - Cost of analysis
    - Resources needed
    - Changes Needed for Your PMS
    - Improved data collection?
    - New software for reporting test data and linking various databases?
    - Adopting standard test protocols?
    - Reorganization?
    - Obtaining new and better equipment?
    - Better training of personnel?

5.3 APPROACH AND TIMETABLE FOR THE WORKSHOP

The class will be divided into groups. Each group will pick

- A chair-person or moderator
- A recorder to make notes
- A scribe to write key points on a visual pad
- A reporter (usually separate from the moderator) to report the findings for discussion to the main group, using the key points on the visual pad, at the end of the workshop.

The timetable will be roughly as following:

- One hour for intra-group discussion and preparation of presentation,
- Total of 30 minutes for presentations, 5 - 8 minute presentation for each group,
- 15 minutes for discussion amongst all participants.

6 PAVEMENT (OVERLAY) DESIGN EVALUATION ANALYSIS

6.1 INTRODUCTION

This Module will describe examples of engineering analyses carried out by Washington DOT, Texas DOT and Arizona DOT, in the area of pavement design and overlay design. The examples in Texas and Arizona will be preceded by a general description of the PMS related activities in those states. For Washington DOT this was done in Module 4.

6.2 WISDOT'S SCOPER DESIGN METHOD IN PERSPECTIVE

Prior to 1994, Washington State DOT programmed projects on a "worst first" basis. The implementation of a pavement management system improved this process, provided impetus to improving the highway system in Washington and also gained the attention of the legislature.

In 1993 the state legislature established criteria that required the use of life cycle cost analysis to prioritize pavement rehabilitation projects. The pavement management system predicts a "due year" for each pavement section based on historical, condition, and structural data in the pavement management database and performance trend equations developed for the system as a whole. An annual report on the condition of the pavement network includes those pavements, which are due for overlay or rehabilitation in the immediate year and in subsequent years for longer range planning. These reports are generated and provided to the programming division for project selection.

Even though this is a programming example, it is an excellent case study of the engineering use of the pavement management data. The data is not only being used directly for programming, but is used over a period of time to improve the way in which programming is done based on engineering analysis and criteria.

6.2.1 Subsequent Improvements Using PMS

Problems arose when using the worst - first method of programming. Once the preliminary program was set up, based on the initial budget estimates, WSDOT developed the Capital Preservation Budget. Then the individual projects were sent forward for pavement design once the budget was formulated. However, by this time the program was already set and the amount of money available for an individual project had been predefined within the budget. This situation was found to create problems because often the money budgeted for a specific project was inaccurate. Generally not enough money was programmed, although in some cases excess money was provided. In these cases it was necessary either to delay the project until more money could be made available, or to shorten the projects length so that the allotted budget was not exceeded.

6.2.2 Overlay Scoping Technique

SCOPER is a simplified overlay design method that is significantly better than the original method used for estimating overlay thickness and developing programming budgets [Pierce 96b]. SCOPER accesses PMS data for estimation of overlay thickness. SCOPER estimates required overlay thickness approximately 80% of the time to produce designs within 10 - 15% of the final required design.

Attachment 6.6, "An Overview of the WSDOT Pavement Management System Overlay Scoping Technique," describes this process in detail and gives example overlay thickness calculations.

The SCOPER process uses the Asphalt Institute's component analysis method with modification to layer coefficient based on Washington characteristics. [Asphalt 83] [WSDOT 95a]. The approach requires that the total pavement structure be developed as a new design for the specified service conditions. The method takes into account pavement condition, type, and thickness of the pavement layers. Substantial judgment is required to effectively use this type of process. The judgment is mainly associated with the selection of "weighting factors" to use in evaluating the structural adequacy of the existing pavement layers.

SCOPER uses a relationship between pavement structure and traffic to estimate the subgrade's stiffness. The existing structural integrity of the pavement is converted to an equivalent thickness of asphalt concrete, which is then subtracted from the required thickness for a new full depth asphalt concrete design to determine the required overlay thickness.

The following lists the steps taken:

1. The resilient modulus (M_r) is estimated for the subgrade. This may be accomplished using general subgrade classes, a performance based estimation technique, or deflection data when available. These three methods are described in Attachment 6.6.

2. Effective thickness of the pavement is determined by multiplying each layer thickness by a conversion factor. The factor is based on the type of material and condition of the pavement.

3. Full depth design thickness is determined using the Asphalt Institute's design chart for full depth asphalt concrete design using the expected traffic in terms of 18 kip equivalent single axle loads (80 kN).

4. Overlay thickness is calculated by subtracting the effective thickness of the pavement determined in step 2 from the full depth thickness determined in step 3.

This is an excellent example of engineering uses of pavement management data to improve network level project scoping. The availability of the pavement management database has made it possible to develop SCOPER and to produce practical, more accurate design estimates at an early date to result in improved pavement design and performance within the state highway system. The initial scoping design is then available to WSDOT regional engineers as a preliminary estimate for their full design process as discussed in the next section.

6.2.3 Review of Pavement Design Practices

In WSDOT the regional engineers have the responsibility to design their pavements. The pavement designs are then transmitted to the central office to be reviewed by the pavement design engineer at the Service Center (formally headquarters). Since the pavement management system contains history of all contract work on pavements in the state, the pavement design engineer can quickly and effectively review all of the designs submitted within the required timeframe.

As with many other aspects of a pavement management database the important thing to note is the timeliness with which the engineering process can be done because of the availability of the database. Without the database it might be possible to do the design review but it could not be done in a timely manner and as a result the design process would be less effective. The importance of a well designed and well-executed pavement management database for improving the design process should not be underestimated.

A complete example of this effort is extracted from WSDOT Pavement Guide, Volume 3 – "Pavement Analysis Computer Software and Case Studies." This section is reproduced here for completeness. [WSDOT 95a]

This case study comes from a section of State Route (SR) 395, which begins at the community of Chewelah, Washington (MP 207.81) and proceeds north to Iron Mountain Road (MP 212.67). The 1994 pavement condition survey for this project indicated 5 to 15 % of low to medium severity alligator cracking and 30 % medium to high severity longitudinal cracking. According to the WSPMS, the current PSC on this project is 58 and is projected to

reach a program level PSC of 50 in 1995; however, the PSC ranges from 0 to 80 for specific locations. In addition to a structural overlay, this project also proposes to widen the roadway by 0.6 m to provide for a 3.7 m lane and 2.4 m shoulder in each direction. This project was scheduled for construction in 1995.

The Stevens County Soils Survey shows the project site is composed of varying morphology. The predominate soil is the Hodgson Silt Loam (ML). This soil is very deep, moderately well drained and is located in undulating terraces. From February to April there is typically a perched water table at depths of 600 to 900 mm. Related engineering properties are: moderately slow permeability, low to moderate shrink swell potential, and moderate susceptibility to frost action. Typical base material, retrieved by auguring, is silty sandy gravel or sandy gravel and varies from 305 to 457 mm in depth.

This project was originally constructed in 1945 with a 20 mm bituminous surface treatment placed over 80 mm of crushed surfacing top course (crushed stone base) and from 230 to 460 mm of select roadway borrow. Construction during 1948 placed an additional 20 mm of bituminous surface treatment. Projects after 1948 rehabilitated different sections within the current project limits. A summary of AC core depths, base thickness and core description are listed in Table 6.1. The base thickness was taken from the WSPMS.

Table 6.1 Summary of Roadway Surfacing Depths

Core Location	Depth AC (mm)	Depth Base (mm)	Comments
207.85	134	457	Core taken at a crack, crack is full depth
208.00	152	457	Core taken at a crack, core not intact
208.50	119	305	Core taken at a crack, crack is full depth
209.00	116	305	Very fatigued, core broke into several pieces
209.05	107	305	Fatigued area, crack is full depth
209.40	149	335	Core taken at a crack, crack is full depth
209.80	165	396	AC core intact
210.00	113	366	Fatigue in both wheel paths, crack is full depth
210.50	98	366	Core taken at a crack, crack is full depth
211.00	229	366	Core broke into several pieces
211.50	283	366	AC core intact
212.00	299	366	AC core intact
212.50	229	366	Top 183 mm in good condition

Current traffic volumes are around 5,500 vehicles per day (two way) with 13% trucks. The design period is 15 years and the associated estimated ESALs are 2,896,000.

Table 6.2 summarizes the deflection data that was collected on this project on April 14, 1993. Though the project was over 7.8 km long and FWD testing was performed every 75 m, only the FWD data that corresponds to a core location is being evaluated as part of this case study. Knowing the AC layer thickness to within 6 mm is essential in assuring a more accurate prediction of layer moduli in the backcalculation procedure. The average pavement temperature at the time the FWD data was collected was 8°C to 10°C. The timing of the survey was about 1.5 to two months after the spring thaw in this area.

Table 6.2 FWD Deflections, Area Value and Subgrade Modulus

Core Location	Load (N)	Deflections (mm) D0	D203	D305	D610	D91 5	D1220	Area Value (mm)	M_R (MPa)
207.85	75 352	795	665	589	350	231	169		
	53 762	615	516	460	263	173	126		
	41 909	494	416	370	206	134	101		
	27 657	335	286	252	130	86	72		
Normalized Values		**467**	**394**	**350**	**193**	**127**	**97**	**540**	**99**
208.00	75 563	687	547	472	286	186	134		
	53 692	540	431	371	220	141	101		
	41 839	445	354	304	178	113	82		
	27 516	313	248	211	118	73	52		
Normalized Values		**421**	**336**	**288**	**167**	**106**	**76**	**512**	**114**
208.50	74 858	379	302	260	15O	81	58		
	54 468	296	236	202	114	54	44		
	42 403	244	194	166	92	46	33		
	28 010	171	136	114	61	32	22		
Normalized Values		**229**	**182**	**155**	**86**	**43**	**32**	**494**	**222**
209.00	72 530	1505	1234	1080	541	242	130		
	52 210	1172	953	827	396	170	91		
	41 133	938	757	651	299	126	68		
	27 375	635	505	426	185	77	44		
Normalized Values		**902**	**728**	**625**	**1 290**	**123**	**67**	**475**	**66**
209.05	71 048	1426	1140	969	556	344	236		
	51 293	1118	894	751	420	254	174		
	40 427	905	718	597	321	191	128		
	26 669	644	488	392	190	118	71		
Normalized Values		**893**	**702**	**581**	**1 311**	**189**	**125**	**481**	**61**
209.40	71 189	1597	1316	1106	631	377	244		
	51 646	1268	1044	871	491	293	189		
	40 498	1020	846	697	382	227	145		
	29 952	725	581	466	241	145	94		
Normalized Values		**1002**	**821**	**675**	**369**	**220**	**142**	**496**	**51**
209.80	76 763	677	548	475	286	182	126		
	54 397	528	429	371	220	137	94		
	42 403	426	347	298	171	107	72		
	27 869	298	240	203	111	69	44		
Normalized Values		**402**	**326**	**280**	**160**	**100**	**67**	**516**	**119**

Table 6.2 FWD deflections, area value, and subgrade modulus (continued)

Core Location	Load (N)	Deflections (mm) D0	D203	D305	D610	D915	D1220	Area Value (mm)	M_R (MPa)
210.00	74364	914	742	639	426	285	197		
	53480	706	574	490	323	213	145		
	41909	567	457	388	250	162	111		
	27446	385	307	256	160	101	70		
Normalized Values		**538**	**434**	**367**	**236**	**154**	**105**	**529**	**80**
210.5	76 339	568	488	423	265	175	120		
	54 327	441	380	329	201	128	89		
	41 980	355	306	264	159	100	68		
	27 446	244	210	179	104	63	42		
Normalized Values		**336**	**289**	**249**	**149**	**93**	**64**	**547**	**128**
211.0	76 410	344	300	278	204	154	116		
	54 820	264	228	209	155	116	85		
	42 826	203	181	166	122	90	66		
	28 433	144	121	109	80	58	43		
Normalized Values		**194**	**168**	**153**	**113**	**83**	**61**	**626**	**168**
211.50	77 680	320	266	239	169	119	82		
	56 161	234	195	175	123	87	59		
	43 955	179	150	134	93	65	44		
	28 857	113	94	83	57	40	27		
Normalized Values		**162**	**135**	**121**	**84**	**59**	**40**	**584**	**226**
212.00	78 809	570	519	483	362	269	199		
	56 161	431	392	364	272	201	149		
	44 590	340	310	287	215	158	118		
	29 280	225	204	188	139	103	76		
Normalized Values		**306**	**278**	**257**	**192**	**142**	**105**	**660**	**99**
212.50	80925	495	441	407	316	242	177		
	57502	382	337	3.10	241	183	135		
	45 790	296	267	244	191	144	102		
	30 197	206	178	161	125	93	64		
Normalized Values		**266**	**235**	**214**	**167**	**125**	**89**	**650**	**114**

The normalized (40 kN) deflections, area value [WSDOT 95, Paragraph 3.2.2] and the subgrade modulus using the normalized deflections, as determined by Equation 6.1 below [WSDOT 95, Paragraph 1.8.3.2] are also shown in Table 6.2.

$$M_r = \frac{(40\,kN)(0.2892)(1000)^2}{(610\,mm)(Defl_{610})}$$

Eq. 6.1

As shown in Table 6.2, the average normalized D0 deflection ranges from 138 µm to 1002 µm and average 466 µm with a standard deviation of 290 µm. Deflections less than about 760 µm are considered normal.

The area values shown in Table 6.2 do not suggest extremely weak AC, such as a loss of stiffness due to stripping. For example, typical theoretical area values for various uncracked AC thicknesses are:

AC Thickness (mm)	Approximate Area Parameter (mm) Normal Stiffness (MPa)	Low Stiffness
50	430	410

75	490	450
100	540	490
125	590	530
150	620	550
175	650	570
200	670	590
225	690	610
250	700	620

A quick, approximate check of the pavement structure is to compare the actual area value to see if it falls within the range (normal to low stiffness), above this range (above normal stiffness) or below this range (below normal stiffness). This comparison follows:

Core Location	AC Thickness (mm)	Actual Area (mm)	Above, Below or Within Range
207.85	134	540	Below
208.00	152	512	Below
208.50	119	494	Below
209.00	116	475	Below
209.05	107	481	Within
209.40	149	496	Below
209.80	165	516	Below
210.00	119	529	Below
210.50	98	547	Above
211.00	229	626	Within
211.50	283	-	-
212.00	299	-	-
212.50	229	650	Within

The above basically suggests MP's 207.85 to 210.00 will likely need a structural overlay. MP's 210.50 to NT 212.50 are in better structural condition suggesting the overlay requirement will be minimal.

The elastic modulus values of the subgrade range from 51 MPa to 275 MPa with an average of 108 MPa and a standard deviation of 63 MPa. On the average, a subgrade modulus of 100 MPa is typical for Washington State. This fairly large variation in stiffness is most likely due to variation in soil moisture and the proximity of underlying rock, more than actual soil or deposition changes along the roadway.

There are two fairly distinct sections occurring within the project limits. The roadway from MP 207.80 to NT 210-55 is in fair to poor condition with low to medium severity longitudinal and alligator cracking. From MP 210.55 to MT 212.67 the roadway is generally in good condition with only a few locations of low severity longitudinal and alligator cracking. Figure 6.1 graphically shows how the normalized center deflection, normalized Area value and subgrade modulus varies throughout the projects length. Figure 6.1 was created from an in-house computer program and this program has yet to be converted to metric units. Therefore, Figure 6.1 shows the center deflection in miles, the area value in inches and the subgrade modulus in ksi.

WSDOT Non-Destructive Pavement Testing **Date Tested = 04/14/93**
Case I SR 395, Chewelah CL to Iron Mt. Road

NOTE: Summary values are normalized to a 9,000 lb load and adjusted for pavement thickness and temperature. Area values are normalized to a 15,000 psi subgrade. Modulus determination is based on a deflection at the 4 sensor (2 feet from the load)

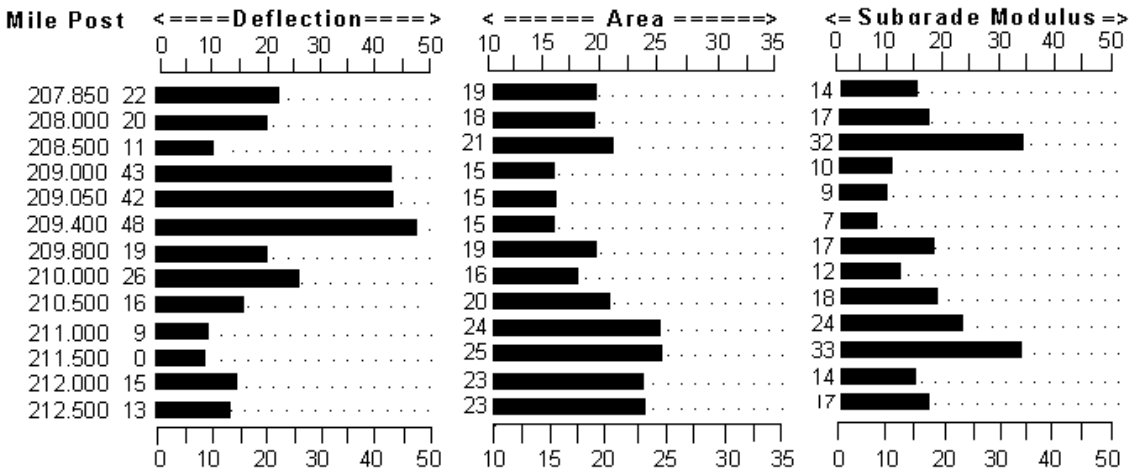


Figure 6.1 Normalized Center Deflection, Area Value and Subgrade Modulus

6.2.4 Overlay Design Procedures

This section describes three overlay design procedures that are currently used by WSDOT:

1. WSPMS SCOPER,
2. WSDOT Empirical-Mechanistic Overlay Design, and
3. AASHTO Overlay Design Procedure.

The three procedures can be used as design checks as will be more fully described later.

WSPMS SCOPER Overlay Design Procedures

WSPMS SCOPER is partially based on the Asphalt Institutes Component Analysis procedure [WSDOT 95, Paragraph 1.3]). Specific modifications made by WSDOT can be found in "Washington State Pavement Management - A 1993 Update, Research Report WA-RD 274.1, Washington State Department of Transportation, Olympia Washington, 1993.

The WSPMS SCOPER uses a component analysis approach that essentially requires the total pavement structure to be developed as a new design for the specified service conditions and then compared to the existing pavement structure (taking into account both pavement condition, type, and thickness of the pavement layers). The component design process requires substantial engineering judgment. This judgment is mainly associated with the selection of "weighting factors" or "conversion factors" to use in evaluating the structural adequacy of the existing pavement layers. The SCOPER design approach uses relationships between subgrade strength, pavement structure, and traffic to determine the overlay thickness.

Layer coefficients and ESAL's were determined using the WSPMS. The subgrade modulus as determined from the FWD analysis as shown in Table 6.2 was used in this analysis. Refer to Table 6.3 for SCOPER overlay design results. To illustrate this process, MP 207.85 will be used as an example.

MP 207.85

(134 mm, AC)(0.30)	=	40 mm.
(457 mm Base)(0.30)	=	137 mm
Effective Thickness	=	177 min

M_R = 99 MPa

ESAL = 2 896 000

Refer to Design Thickness charts in Asphalt Institutes MS- 1 Manual or Volume 2, SECTION 7.0, Paragraph 1.3.

Design Thickness	=219 mm
Effective Thickness	=177 mm
Overlay Thickness	= 42 mm

Table 6.3 WSPMS SCOPER Overlay Design Procedure

Core Locations	Depths AC (mm)	Depths Base (mm)	Layer Coefficients AC	Layer Coefficients Base	Subgrade Modulus (MPa)	Thickness (mm) Design	Thickness (mm) Effective	Thickness (mm) Overlay
207.85	134	457[1]	0.30'	0.30	99	248	177	71
208.00	152	457	0.30	0.30	114	238	183	55
208.50	119	305	0.30	0.30	222	184	128	56
209.00	116	305	0.30	0.30	66	274	127	147
209.05	107	305	0.30	0.30	61	279	124	155
209.40	149	335	0.30	0.30	51	289	146	143
209.80	165	396	0.30	0.30	119	234	169	65
210.00	113	366	0.30	0.30	80	246	144	118
210.50	98	366	0.30	0.30	128	229	139	90
211.00	229	366	0.82[2]	0.30	168	208	298	0
211.50	283	366	0.82	0.30	226	183	342	0
212.00	299	366	0.82	0.30	99	248	355	0
212.50	229	366	0.82	0.30	114	238	298	0

[1] AC "C" value based on a PSC = 0
[2] AC "C" value based on a PSC = 74

WSDOT Mechanistic-Empirical Overlay Design

EVERCALC

Currently, when a pavement structure is analyzed using FWD data and a back calculation procedure, it is recommended to vary the use and stiffness of the stiff layer. A stiff layer should be used when the underlying material is saturated or if underlying rock or other very stiff deposits are known to exist [WSDOT 95, Paragraph 3.3.4]. In most instances, the presence, the stiffness, and depth to stiff layer is not known. Therefore, it is advisable to use three back calculation approaches with regard to a potential stiff layer: 1) no stiff layer, 2) a stiff layer at 345 MPa (50,000 psi), which indicates a moist or saturated layer, and 3) a stiff layer at 6 895 MPa (1,000,000 psi), which indicates a rock layer or stiff deposit.

Input Values

General Data

Number of layers = 3 (no stiff layer), 4 (stiff layer)
Units = Metric
Indicate whether or not a stiff layer option is to be used
Temperature correction to be applied
Temperature Measurement = Direct Method
Plate radius = 15 cm
Seed Moduli = User supplied
Sensor No. 1 2 3 4 5 6

Radial Offset (cm)	0	205	305	610	915	1220

Layer Information is shown in Table 6.4.

Table 6.4 Layer Characteristic Input Values

Layer	Description	Poisson's Ratio	Modulus (MPa) Initial	Minimum	Maximum
1	AC	0.35	2758	689	13 790
2	Base	0.40	172	34	3447
3	Subgrade	0.45	103	34	3447
4*	Stiff layer (water)	0.35	345		
4*	Stiff layer (rock)	0.30	6895		

* Denotes the use of a stiff layer.

Deflection Data

Refer to Volume 3, SECTION 2.0 for the necessary procedures for entering deflection data or for converting the raw FWD deflection file into EVERCALC [WSDOT 95a].

Back calculation Results

The back calculated layer moduli, for all three cases (no stiff layer and stiff layers at 345 and 6 895 MPa) are shown in Table 6.5.

EVERPAVE

EVERPAVE is a mechanistic-empirical overlay design procedure that was developed by WSDOT. The pavement analysis is accomplished by use of EVERSTRS (used as a subroutine, see Volume 3, Section 1.0), which can account for the stress sensitive characteristics of the unbound materials. EVERPAVE uses the material properties (modulus) of each pavement layer (e.g. AC, base and subgrade). Traffic load repetitions and the environment (seasonal temperatures and moduli) to determine the appropriate overlay design thickness. The determination of the appropriate thickness is based on the two primary distresses found in flexible pavements in Washington State; rutting and fatigue cracking [WSDOT 95, Paragraph 1.6]. The EVERPAVE program calculates the overlay thickness by comparing the pavement performance lives for fatigue and rutting with the projected design traffic volume (ESALs). When the minimum repetition of the two failure criteria is greater than the traffic volume, the final overlay thickness is produced. Otherwise, the overlay thickness is increased by an incremental thickness and the analysis is repeated.

Table 6.5 Summary of EVERCALC Results

Core Location	No Stiff Layer Eadj[1] (MPa)	Eac[2] (MPa)	Ebase (MPa)	Esub (MPa)	RMS	Depth to Stiff Layer (m)	Stiff Layer @ 345 MPa Eadj (MPa)	Eac (MPa)	Ebase (MPa)	Esub (MPa)	RMS	Stiff Layer @ 6 895 MPa Eadj (MPa)	Eac (MPa)	Ebase (MPa)	Esub (MPa)	RMS
207.85	1137	3640	115	82	3.09	4.96	**979**[3]	3134	**150**	**70**	4.12	929	2975	164	63	4.53
208.00	900	2881	121	101	0.70	4.05	**735**	2352	**168**	**80**	1.80	677	2167	191	68	2.52
208.50	3657	11706	115	250	3.39	1.55	**2535**	8114	**298**	**154**	5.37	1399	4480	761	63	10.73
209.00	610	1953	34	90	13.40	1.19	**1734**[4]	5551	**117**	**34**	24.56	2630	8417	34	34	41.88
209.05	1167	3736	41	59	2.04	2.50	**619**	1981	**128**	**35**	5.29	1274	4079	81	34	10.12
209.40	416	1331	34	52	2.00	2.17	**645**	2065	**34**	**34**	14.39	652	2087	34	34	21.76
209.80	976	3125	86	117	0.96	3.61	**781**	2500	**143**	**89**	1.60	691	2211	180	73	2.41
210.00	2048	6556	95	72	2.29	3.23	**1137**	3639	**200**	**48**	0.83	1000	3200	227	42	0.99
210.50	4868	17036	115	119	0.64	8.63	**4405**	15417	**143**	**120**	0.76	4154	14536	163	110	0.85
211.00	1879	6576	148	139	0.76	5.68	**1904**	6661	**196**	**126**	0.63	1796	6287	259	105	0.68
211.50	1317	4608	81	256	0.66	3.06	1272	4450	121	204	0.67	**1158**	4053	**268**	**110**	1.13
212.00	825	2887	34	97	2.08	7.97	876	3064	34	92	1.88	**895**	3131	**34**	**86**	1.86
212.50	1968	6886	36	122	1.23	5.13	1927	6742	66	91	0.97	**1836**	6427	**102**	**74**	0.86

1 Eadj is Eac adjusted to a standard temperature of 25°C (77°F).

2 Eac is the actual backcalculated modulus for the AC at the insitu field temperatures.

3 Bold numbers indicate the moduli that were selected for input into EVERPAVE. Moduli are selected based on engineering judgment and low RMS.

4 Due to the severity of distress at this location the AC modulus was "fixed" at 690 MPa and the base and subgrade moduli were reevaluated in EVERCALC. The corresponding base and subgrade modulus are 117 and 34 MPa, respectively.

Input Values

General Input Data [WSDOT 95, Paragraph 1.6.46].

Tire Load	20 000 N
Tire Pressure	689 kPa
Shift Factor for New AC	10
Shift Factor for Existing AC	10
Dual Tire Spacing	35.6 cm

	Spring	Summer	Fall	Winter
Seasonal Variation				
Base Course	1.00	1.54	1.38	1.69
Subgrade	1.00	1.11	1.0	1.22
Traffic	1.00	1.00	1.00	1.00
Mean Air Temperature °C	7.6	18.1	7.5	-2.8
Season Period (months)	4	4	3	1

For mean air temperature refer to Volume 2, SECTION 7.0, Appendix 7.1, Table 1 and for seasonal periods (months) refer to Table 7.4, Volume 2, SECTION 7.0, Paragraph 1.6. The moduli ratios assume that the FWD deflection testing was performed during the critical spring period. Thus, the spring moduli ratios shown above were set to 1.0 (base and subgrade). The other seasonal moduli ratios were then adjusted accordingly (refer to Volume 2, SECTION 7.0, Table 7.4). [WSDOT 95]

Pavement Data

Overlay AC Moduli	= 2 758 MPa
Poisson's ratio	= 0.35
Initial Overlay Thickness	= 0.5 cm
Thickness Increment	= 0.5 cm

Traffic Data

80 kN ESAL's for Design Period (15 years)	= 2 896 000
Lane Distribution Factor	= 1.00
Total Design ESAL's	= 2 896 000

The results of the EVERPAVE overlay analysis are presented in Table 6.6. As a reminder, the initial overlay thickness was established at 5 mm for this case study. Therefore, those locations that indicate a 5 mm overlay, are more likely based on the initial overlay thickness set by the user.

Table 6.6 EVERPAVE Overlay Thickness Results

			Selected Layer Moduli			EVERPAVE
Core Location	AC (cm)	Base (cm)	Eac (MPa)	Ebase (MPa	Esub (MPa)	Overlay (mm)
207.85	13.4	45.7	979	150	70	50
208.00	15.2	45.7	735	168	80	40
208.50	11.9	30.5	2535	298	154	10
209.00	11.6	30.5	690	117	34	95
209.05	10.7	30.5	619	128	35	100

209.40	14.9	33.5	645	34	34	110
209.80	16.5	39.6	781	143	89	35
210.00	11.3	36.6	1137	200	48	60
210.50	9.8	36.6	4405	143	120	35
211.00	22.9	36.6	1904	196	126	10
211.50	28.3	36.6	1158	268	110	10
212.00	29.9	36.6	895	34	86	10
212.50	22.9	36.6	1836	102	74	10

AASHTO - DARWin Pavement Design System

DARWin is the computerized version of the pavement design models presented in AASHTO's "Guide for Design of Pavement Structures 1993." From the DARWin User's Guide, "In the AASHTO overlay design procedure, the structural capacity for future traffic (SC_f) and the effective structural capacity (SC_{eff}) of the existing pavement are calculated using one of up to four available methods. These structural capacities are then used to determine the required overlay structural capacity." For this case study, the non-destructive testing method will be used for overlay determination. DARWin uses the deflection data to backcalculate the subgrade modulus (M_r) and the effective pavement modulus (E_p). These values are then used to determine the structural numbers for the existing pavement and for future traffic volumes. The required overlay structural number is the difference between SC_f and SC_{eff}.

Input Values

18 k ESAL's	2 896 00
Initial Serviceability	4.5
Terminal Serviceability	3.0
Reliability Level (O/o)	50
Overall Standard Deviation	0.50
Overlay Layer Coefficient (aol)	0.44

Calculated Overlay Structural Number (SN_{oi}) = Non-Destructive Method

Point-by-Point Backcalculation	
FWD Load	9000 lbs
Resilient Modulus Correction Factor	0.5
Base Type	Granular
Mid-depth Pavement Temperature	50°F (MP 207.85 - MP 210.05)
	47°F (MP 210.05 - MP 212.67)

The normalized load (9000 lbs) and corresponding deflections are shown in Table 6.7. WSDOT uses a FWD sensor spacing of 0, 8, 12, 24, 36, and 48 inches. DARWin will accommodate deflection data collected at any deflection spacing and at any load level. The FWD sensor spacing was selected such that the deflection basin would be adequately identified. Typically, the pavement structural design is based on a legally loaded axle of 18,000 lbs (9000 lbs per one-half of the axle). Therefore, the normalized deflections at 9000 lbs are used in this procedure. Results of this analysis are shown in Table 6.8.

Table 6.7 Normalized Deflection Data

MP	Load (lbs)	$D_{at\ 0"}$ (mils)	$D_{at\ 8"}$ (mils)	$D_{at\ 12"}$ (mils)	$D_{at\ 24"}$ (mils)	$D_{at\ 36"}$ (mils)	$D_{at\ 48"}$ (mils)
207.85	9000	18.37	15.51	13.77	7.59	4.99	3.83
208.00	9000	16.59	13.21	11.32	6.56	4.16	2.98
208.50	9000	9.01	7.16	6.10	3.37	1.68	1.24

209.00	9000	35.51	28.65	24.62	11.40	4.84	2.65
209.05	9000	35.14	27.62	22.88	12.24	7.43	4.90
209.40	9000	39.46	32.33	26.59	14.54	8.68	5.58
209.80	9000	15.83	12.83	11.00	6.30	3.93	2.62
210.00	9000	21.18	17.07	14.45	9.31	6.05	4.15
210.50	9000	13.21	11.39	9.79	5.86	3.68	2.52
211.00	9000	7.63	6.61	6.02	4.45	3.28	2.41
211.50	9000	6.36	5.31	4.75	3.31	2.31	1.57
212.00	9000	12.05	10.95	10.13	7.55	5.58	4.14
212.50	9000	10.49	9.27	8.44	6.57	4.94	3.50

Table 6.8 AASHTO DARwin Overlay Thickness Results

Core Location	Pavement Depths AC (in)	Base (in)	Total (in)	SN for Future Traffic	Effective Existing Pavement SN	Calculated Overlay SN (SN01)	Overlay Thickness (in)
207.85	5.3	18.0	23.3	3.72	3.78	0.00	0
208.00	6.0	18.0	24.0	3.46	3.96	0.00	0
208.50	4.7	12.0	16.7	2.70	3.56	0.00	0
209.00	4.6	12.0	16.6	4.37	2.19	2.18	5.0
209.05	4.2	12.0	16.2	4.48	2.19	2.29	5.2
209.40	5.9	13.2	19.1	4.77	2.43	2.34	5.3
209.80	6.5	15.6	22.1	3.38	3.73	0.00	0
210.00	4.4	14.4	18.8	4.05	3.21	0.99	2.3
210.50	3.9	14.4	18.3	3.37	3.51	0.00	0
211.00	9.0	14.4	23.4	3.14	5.45	0.00	0
211.50	11.1	14.4	25.5	2.73	5.99	0.00	0
212.00	11.8	14.4	26.2	3.87	5.16	0.00	0
212.50	9.0	14.4	23.4	3.70	5.02	0.00	0

6.2.5 Overlay Thickness Design Summary

As shown in Table 6.9, the three overlay design procedures indicate various required overlay thickness at any given location. The variations between the three overlay design procedures are due in part by the various characteristics and design criteria used by each design method. However, using multiple design procedures allows for a check and balance of the necessary overlay thickness and requires the use of engineering judgment for the final decision.

Table 6.9 Summary of Overlay Thickness

Core Location	EVERPAVE (mm)	AASHTO DARWin (mm)	WSPMS SCOPER (mm)
207.85	50	0	71
208.00	40	0	55
208.50	10	0	56
209.00	95	127	147
209.05	100	132	155
209.40	110	135	143
209.80	35	0	65
210.00	60	58	118
210.50	35	0	90
211.00	10	0	0
211.50	10	0	0
212.00	10	0	0
212.50	10	0	0

Due to the difficulty in constructing many changes in overlay thickness, the project should be constructed with logical overlay thickness breaks. For this case study, MP 207.85 to MP 208.50 would require a 45 mm overlay, MP 208.50 to MP 209.40 would require a 100 mm overlay, MP 209.40 to MP 210.50 would require a 45 mm overlay and finally, no structural overlay would be required from MP 211.00 to MP 212.50.

Even though the three overlay design procedures indicate that no overlay is required on the last 2.4 km, it is important to consult the WSPMS to verify that there is no need for rehabilitation on the last section of this project.

The WSPMS and core data indicate that rehabilitation would be needed on this section of roadway in 1997, albeit to correct surface cracking. The adjacent project (20 km) to the north was overlaid with 40 mm HMA Class B in 1994. Therefore, for timing purposes, the last 2.4 km should be overlaid with this project. The recommended overlay thickness for that portion of the project would be 45 mm.

The actual rehabilitation of this project consisted of two options:

1. Cold in-place recycling from NT 207.80 to MY 210-55, to a maximum thickness of 105 mm and then overlaying with HMA Class F at the following depths:

Milepost	Depth
207.81 to 209.00	45 mm
209.00 to 209.65	60 mm
209.65 to 212.67	45 mm.

2. Grind a maximum depth of 90 mm and replace with 90 mm HMA Class F. Then overlay the roadway as indicated by the overlay depths in Option 1.

6.3 TXDOT'S ANALYSIS TO RELATE RUTTING TO SUBGRADE PROPERTIES

6.3.1 Texas DOT Pavement Management Experience

The Texas Department of Transportation (TxDOT) manages about 79,000 centerline miles of roadway that include about 3,000 miles of Interstate highways, 12,000 miles of U.S. highways, and 16,000 miles of State highways. The remainder is made up of about 41,000 miles of Farm to Market roads, 6,000 miles of frontage roads, and 300 miles of park roads.

TxDOT is divided into 25 districts, and each district manages about 3,000 centerline miles of roads. The districts are supported by a central office where the State level pavement management function falls under the supervision of the Construction Division. Most districts are further subdivided into about 4 divisions managed by Area Offices. Maintenance is generally managed by about 250 County Maintenance Offices. The Central office develops State-wide programs including addressing legislative questions, providing support for funding requests and allocation decisions, and other strategic level analysis in addition to standard network-level analysis. The 25 districts develop and implement district programs, including selecting sections for funding. The Area Offices provide input to the Districts during the District network-level and project selection analysis. The County Maintenance Offices are often consulted about preventive maintenance and other pavement related maintenance and rehabilitation programs.

Historically TxDOT inspected their pavements a minimum of once every two years, but recently they have inspected most of the pavements annually. Roughness and rutting are collected with equipment operated and maintained by TxDOT personnel; other distress types are collected manually, normally by contract for each district. Six additional distress types are collected for asphalt surfaced pavements, with two optional distress types. Five types of distress are collected for continuously reinforced concrete pavements, and six types of distress are collected for jointed concrete pavements. Skid and deflection measurements are made using TxDOT operated and maintained equipment.

Texas DOT has a strong pavement management program with both network and project level components. Because Texas is such a large, decentralized agency, it has chosen to let the districts take the lead in finding ways to use pavement management and the current pavement management information system (PMIS) software. This has the advantage of keeping the system close to users in the field. It is therefore easy for the central headquarters to

identify what parts of PMIS are most useful, and to develop improvements as needed to meet ever-changing district requirements. It also ensures that newly developed enhancements to PMIS provide quick benefit, because those enhancements are typically the ones that the districts themselves have requested. This decentralized approach provides for strong engineering related applications to be carried out based on PMIS data, on a routine basis by the Districts. The next section summarizes the components of the PMIS followed by several engineering applications that have been carried out based on PMIS data.

What is the Texas PMIS?

This section is taken from a TxDOT pamphlet dated June 1996, by the same title [TxDOT 96].

The Texas PMIS is "an automated system for storing, retrieving, analyzing and reporting information to help with pavement-related decision-making processes."

PMIS helps TxDOT monitor statewide trends in pavement condition, and is part of the allocation formulas of the new Unified Transportation Program (UTP) conceptually depicted in Figure 6.2. District personnel have used PMIS to monitor, select, and set priorities for preventive maintenance programs and determine future needs.

The primary elements and products of PMIS are:

- An inventory of pavements in the network.
- A database of past and current pavement conditions.
- Budget requirements.
- Methods for optimizing and prioritizing projects.

Pavement management is not a new idea. TxDOT has been managing pavements since its inception in 1917. Decisions in the past were based on engineering judgment and experience. However, with the number of experienced personnel declining and funding becoming more restricted old methods needed refinement.

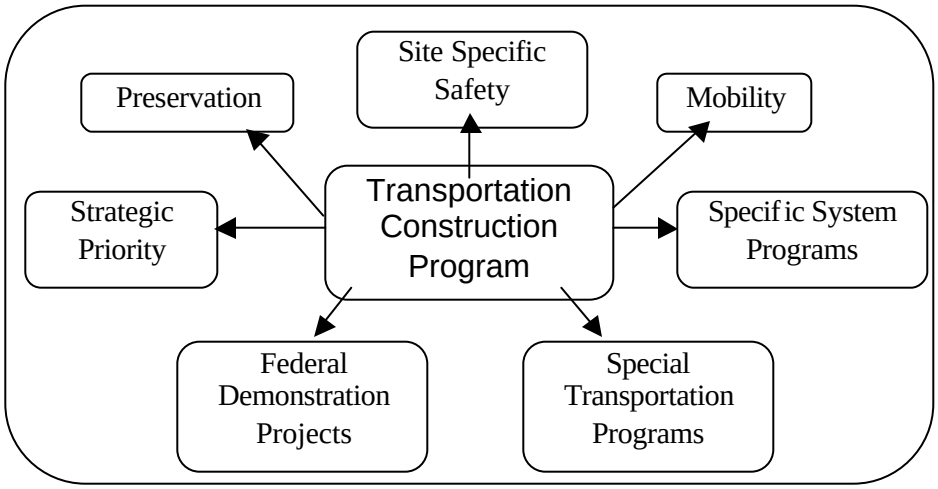


Figure 6.2 Overall Unified Transportation Program (UTP)

This section describes the major elements of PMIS:

- Surveys and Scores
- Reports
- Needs Estimate
- Optimization

- Impact Analysis.

Visual Distress Data

Trained raters do a visual distress survey each year from September to December on the following three major pavement types:

1. Flexible Pavement
2. Continuously Reinforced Concrete Pavement (CRCP)
3. Jointed Concrete Pavement (JCP).

The distress types rated for flexible pavements are shallow rutting, deep rutting, patching, failures, block cracking, alligator cracking, longitudinal cracking, and transverse cracking. On September 1, 1995, TxDOT automated rutting measurements with two levels, the shallow rutting and deep rutting distress data collection (see ride quality and automated rut data). This will speed the manual data collection effort, improve the reliability of ratings, while also making it safer. Two optional distress types are raveling and flushing, which are collected at the option of each district office.

CRCP and JCP are grouped together as rigid pavement. These pavements are a product of Portland cement concrete. The CRCP distress types are spalled cracks, punchouts, asphalt patches, concrete patches, and average crack spacing. The JCP distress types are failed joints and cracks, failures, slabs with longitudinal cracks, shattered slabs, concrete patches, and apparent joint spacing.

These distress ratings combine to make the **Distress Score**, which ranges from 1 (most distress) to 100 (no or very little distress), with a score below 80 indicating problems. A low Distress Score may result from multiple distresses (such as shallow rutting and alligator cracking) or from a severe distress (such as deep rutting). Table 6.10 describes the classes of the Distress Score.

Table 6.10 Distress Score Classes

DISTRESS SCORE	CLASS	DESCRIPTION
90-100	"A"	Very Good
80-89	"B"	Good
70-79	"C"	Fair
60-69	"D"	Poor
1-59	"F"	Very Poor

Ride Quality and Automated Rut Data

Ride quality, along with automated rut data, is collected each year from September to December. Currently, TxDOT uses twelve SIometer/Rutbar vehicles to measure pavement roughness, shallow rutting, and deep rutting.

The SIometer/Rutbar device uses an accelerometer, a processing computer, and a data storage computer installed in a vehicle to measure ride quality. To improve the measurement of ride quality, TxDOT is replacing the SIometer/Rutbar vehicle with the Profiler/Rutbar vehicle. The Profiler/Rutbar device will use lasers and accelerometers to measure the inertial profile of each wheelpath. To ensure data consistency, TxDOT calibrates each ride quality device during the summer. TxDOT does calibration verifications throughout the data collection season to ensure quality data.

To measure rutting, five ultrasonic sensors, mounted on an eight-foot wide bar, measure the distance between the bar and pavement surface. These distances are then used to measure average rutting. Sensors are in each wheelpath, at the center of the vehicle, and eighteen inches outside each wheelpath. These sensors measure distance to the pavement by sending out sound waves and measuring the time it takes the waves to return. The same processing computer that measures ride, also captures the rut depth information. The processing computer then sends the rut depth info to the data storage computer where it is summarized for each 0.1 miles and stored.

PMIS reports ride quality as **Ride Score**. Ride Score is based on a user panel rating that ranges from 0.1 (very rough) to 5 (very smooth). Table 6.11 shows the classes of Ride Score. A Ride Score below 3.0 suggests a rough road to the average person.

Table 6.11 Ride Score Classes

RIDE SCORE	CLASS	DESCRIPTION
4.0 - 5.0	"A"	Very Smooth
3.0 -3 .9	"B"	Smooth
2.0 - 2.9	"C"	Medium Rough
1.0 - 1.9	"D"	Rough
0.1 - 0.9	"F"	Very Rough

Deflection Data

The deflection survey is an optional survey that helps the pavement manager determine the structural integrity of the pavement. Deflection data can be collected throughout the fiscal year in either network- or project-level format and can be stored in PMIS. The Falling Weight Deflectometer (FWD) collects this data. The FWD drops a weight on the pavement and measures the resulting deflection. TxDOT operates thirteen FWDs for the collection of deflection data.

A parameter, called **Structural Strength Index (SSI) Score**, reports deflection data. The SSI score varies from 1 (very weak) to 100 (very strong). An SSI Score below 80 shows that the pavement structure is inferior and may require extensive monitoring and frequent maintenance to maintain suitable driving conditions. This is based on TxDOT studies and experience. Table 6.12 shows the classes of the SSI Score.

Table 6.12 SSI Score Classes

SSI SCORE	CLASS	DESCRIPTION
90 - 100	"A"	Very Strong
80 - 89	"B"	Strong
70 - 79	"C"	Fair
60 - 69	"D"	Weak
1 - 59	"F"	Very Weak

Skid Resistance Data

The other optional PMIS survey type is skid resistance. This data can be collected through out the fiscal year and stored in PMIS. Engineers use this data to evaluate the surface friction properties of aggregate types, asphalt mix design, and pavement construction methods.

A locked-wheel skid trailer and a tow vehicle collect the data at a constant speed. The tow trailer's left wheel locks at periodic intervals while a metered amount of water is sprayed on the pavement surface. TxDOT operates seven skid units for data collection. The score that classifies skid resistance data is the **Skid Score**. The Skid Score ranges from 1 (very poor) to 100 (very good), however scores above 70 are rare. The Skid Score does not indicate the stopping characteristic of a vehicle, driver, or climate condition.

Condition Score

PMIS uses the data from the ride and distress surveys in the calculation of the **Condition Score**. The Condition Score corresponds with the average person's perception of pavement quality. Factors such as climate, traffic, and functional class are not factored into the calculation. Condition Score ranges from 1 (very poor) to 100 (very good), as described in Table 6.13.

Table 6.13 Condition Score Ranges

CONDITION SCORE	CLASS	DESCRIPTION
90 - 100	"A"	Very Good
70 - 89	"B"	Good
50 - 69	"C"	Fair
35 - 49	"D"	Poor
1 - 34	"F"	Very Poor

The distress, ride, and auto rutting surveys are the fundamental surveys of the PMIS. These surveys provide the basic information for decision-making used in managing pavements. However, PMIS has two other surveys - pavement deflection and skid resistance - that can also be used in managing pavements.

Data Collection and Management Sections

PMIS summarizes and reports data on specified Data Collection Sections (usually ½ mile). In the future, when TxDOT converts to the metric system most sections will be one kilometer long. Visual, ride, and automated rutting data are collected from the beginning to the end of a highway within a county once every two years, while deflection and skid data collection is optional.

When analyzing and selecting projects, the pavement manager creates ***Management Sections***. A management section is a segment of pavement, which is treated homogeneously and may be thought of as a candidate project. The pavement manager can link data collection sections together to form management sections or allow PMIS to create them based upon county and maintenance section boundaries, number of roadbeds, and pavement type. The rating and scores are then summarized for each management section.

Both data collection sections and management sections can be used to estimate overall pavement needs and list proposed projects. An example of a management section is shown in Figure 6.3.

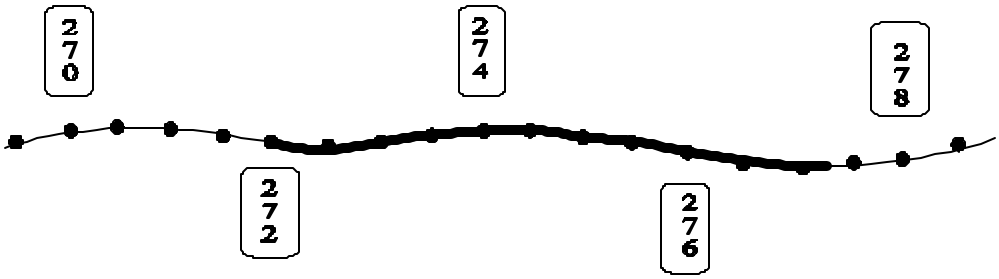


Figure 6.3 Example of Management Section 272 =00 to 277 =0.30

Standard Reports

Standard reports are designed to assist pavement managers with data collection, verification, and analysis. These reports are divided into the three categories listed in the box below.

STANDARD REPORTS

Data Collection Reports
- List of Pavement Sections to be Rated
- Status of Data Collection Survey
- Modified Section Length & Pavement Type

Raw Data Reports
- Distress Data
- Ride Quality Data
- Deflection Data
- Skid Resistance Data
- Automated Rutting Data

Current and Historical Ratings and Scores
- Single Year Ratings and Scores
- Critical Value Ratings and Scores
- Multi-Year Ratings and Scores

Administrative Reports

Administrative reports provide summary data to evaluate the highway network condition over time. These reports show what effect past budgets, weather, and maintenance treatments have had on the pavements; they are listed in the box below.

ADMINISTRATIVE REPORTS

Statewide Average PMIS Scores
- By District and Year
- By Highway System and Year

District Average PMIS Scores
- By Highway System and Year

Statewide Maintenance Level of Service
- By District and Year
- By Highway System and Year

District Maintenance Level of Service
- By Highway System and Year

Needs Estimate

The needs estimate determines the funding needed to repair all pavements in a single year. The estimate is reported in terms of lane miles and funding. The needs estimate classifies pavement needs into the following broad treatment categories:

- Needs Nothing (NN)
- Preventive Maintenance (PM)
- Light Rehabilitation (LRhb)
- Medium Rehabilitation (MRhb)
- Heavy Rehabilitation or Reconstruction (HRhb).

The needs estimate can be run using data collection sections or management sections. The needs estimate reports are listed in the box below.

NEEDS ESTIMATE REPORTS

Statewide Needs Estimate
- by District
- by Highway System

District Needs Estimate
- by Highway System

Executive Summary Needs Estimate
- by District
- by County
- by Maintenance Section

Optimization

Optimization gives the pavement manager a tool to get the most benefit from the available budget. The pavement manager enters the available budget in one of the following ways:

1. For one year (either current fiscal year or a future fiscal year).
2. For one year with an inflation factor.
3. For multiple years (one to ten years).

The program also allows the user to change the percent of truck traffic per year.

The program uses management sections or data collection sections and evaluates the treatment as determined by the needs estimate. If funding is sufficient, the section receives the needs estimate ("needed") treatment. If funding is not sufficient, the program gives the section a ***Stopgap*** treatment. A ***Stopgap*** treatment is repair maintenance needed to preserve a pavement section until the next fiscal year or when sufficient funding becomes available. The number of lane miles that require ***Stopgap*** treatment is an indicator of over- or under-funding.

The optimization program then ranks all of the funded sections according to benefit. Multi-year optimization ages the treated and untreated lane mileage one year. PMIS then applies the next fiscal year's budget to the aged sections.

The needs estimate can be run using data collection sections or management sections. Management sections used in the optimization program are changeable. The pavement manager can change these management sections to match pavement condition, candidate projects, or any other limits.

Even with the ability to alter management sections, PMIS cannot consider all factors. Thus, PMIS allows districts to "force" treatments by:

1. Choosing a section whose treatment did not receive a high enough benefit to be selected with the available budget.
2. Canceling a section recommended for treatment.
3. Changing a treatment.

One example would be a US highway due for widening in three years. The optimization program proposes *Light Rehabilitation*. The district could force this treatment to *Preventive Maintenance* and rerun the optimization program to see the effects on other sections.

Another example is a Farm-to-Market (FM) road that calls for *Preventive Maintenance*. The district knows that a major shipping center will be located on the road. The district could force the treatment on this road to a*Heavy Rehabilitation* and even specify when to apply the treatment.

The optimization reports are listed in the box below.

OPTIMIZATION REPORTS

Statewide Optimization
- By District
- By Highway System

District Optimization
- By Highway System

Impact Analysis

Impact analysis shows the effects of four different factors on current and future pavement condition. These factors are:

1. Budget
2. Management section limits
3. Management section treatments
4. Truck traffic

Impact analysis determines if the current funding will allow pavement conditions to improve, deteriorate, or remain the same.
The impact analysis looks at:

- Average distress score
- Average ride score
- Average condition score
- Pavement maintenance level of service

The following three points are examined in the impact analysis:

1. Current condition (before treatments).
2. After needs estimate treatments.
3. After optimization treatments.

For example, suppose the needs estimate shows that a district needs a budget of $50 million for pavement rehabilitation, and the current average condition score is 87.5. The impact analysis would suggest that if the budget:

- Was reduced to $30 million, the average condition score would be 83.7.
- Was reduced to $40 million, the average condition score would be 86.1.
- Remained at $50 million, the average condition score would be 90.3.

Thus a request for increased funding could accompany the impact analysis report, with the impact analysis report used as justification.

Impact analysis reports are listed in the box below.

IMPACT ANALYSIS REPORTS

Statewide Impact on Average PMIS Scores
- by District
- by Highway System

District Impact on Average PMIS Scores
- by Highway System

Statewide Impact on Maintenance Level of Service
- by District
- by Highway System

District Impact on Maintenance Level of Service
- by Highway System

Summary

PMIS is a set of tools to help monitor current pavement condition and plan ahead for future conditions. It can be used to estimate pavement needs, select preventive maintenance and rehabilitation projects, monitor the effectiveness of existing treatments, and improve future pavement designs. PMIS can help pavement managers improve the overall condition of Texas pavements within given funding by using longer-lasting pavement treatments applied at the right place and at the right time.

6.3.2 Analysis of Rutting as a Function of Subgrade Properties

Approximately one-third of the mainlane Texas Pavement Management Information System (PMIS) flexible pavement sections have measurable amounts of rutting (i.e., ruts at least ½" deep). Analysis of data collected between 1983 and 1987 shows a steady increase in the frequency of observed rutting – a large part of which occurred just in the last year of the analysis period. Considering that the Western Association of State Highway and Transportation Officials (WASHTO) has suggested that any pavement with a rut depth of ½" or more has reached the end of its design life, the PMIS rutting values represent a sizable problem.

Such high rutting values would normally be expected only on seal coats, however, hot mix pavements also have a substantial amount of rutting. In 1987, rutting on hot mix pavements increased rapidly, surpassing the frequency observed on flexible pavements as a whole for the first time in five years. Not only did the frequency of rutting increase, but the rutting was becoming more localized, at times even following exactly along district boundaries.

Comparison of the PMIS pavement distress ratings and deflection data indicates that, although structurally weak subgrade and base/surface layers are no doubt contributing to the increased rutting, enough rutting is occurring on "strong" subsurface layers to warrant examination of pavement material selection, mix design, and paving practices.

Introduction

Although many types of distress may be observed on an asphalt pavement, rutting has historically been one the most important since it suggests the presence of a structural deficiency. Truck traffic is most commonly blamed for causing ruts, however, poor mix design and construction practices can increase a pavement's susceptibility to rutting.

With observed increases in truck traffic, load, and tire pressures, the "rutting problem" has become the subject of frequent study, especially in the Western states of the USA. The WASHTO published a report that suggested that any pavement section rutted greater then 0.5" had reached the end of its design life. A 1988 report by the American Association of State Highway and Transportation Officials (AASHTO) referenced the WASHTO report as part of a nationwide survey on rutting of asphalt pavements. The AASHTO report concluded that rutting is a national problem.

In light of such alarming statistics, the pavement management section examined PMIS data collected from 1983 – 1988 to determine whether Texas has a "rutting problem." This report specifically compares the amount and type of rutting on hot mix pavements with that of all asphalt pavements.

General Statewide Rutting Trends

PMIS ratings describe a noticeable increase in the percent of mainlane sections with rutting in 1987 as compared to that observed in 1983, as shown in Figure 6.4. Not only are the 1987 rutting values the highest observed in the last five years, they also show large increases for flexible and hot mix sections. The hot mix sections, which used to have less frequent rutting than flexible pavements as a whole, now show more frequent rutting. Such a result is troubling for several reasons:

- Hot mix pavements are generally thicker than other flexible pavements, thus representing a greater investment of time, labor, material, and money at risk.

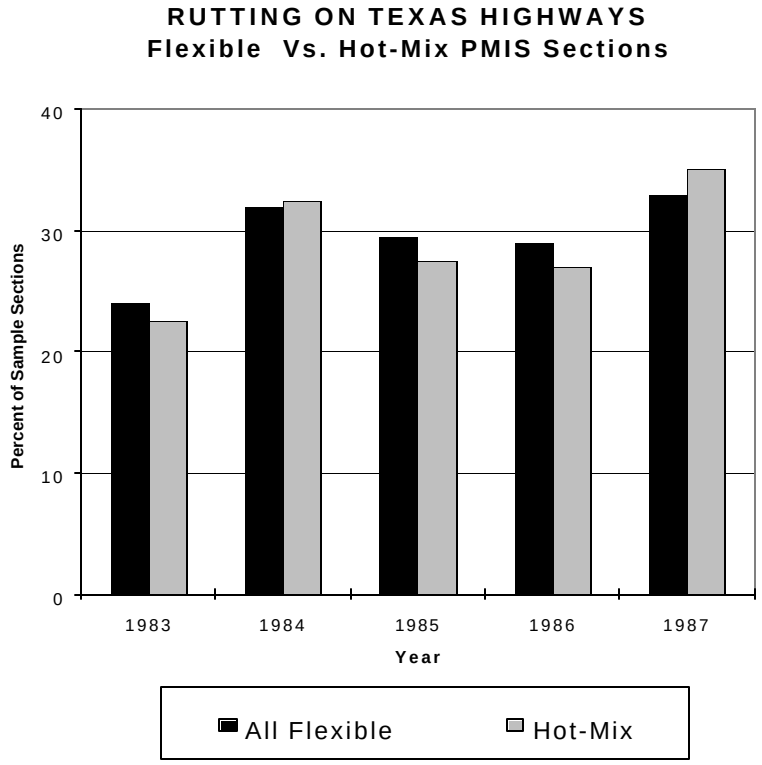


Figure 6.4 Frequency of Rutting on Texas Highways

- Hot mix pavements, because of their enhanced structural properties, are typically used for more important higher-traffic highway sections. As a result, deficiencies in hot-mix pavements have a greater visibility but, because of the traffic, scheduling corrective work is more difficult.

- Hot mix pavements, especially when compared to seal coats, are supposed to be more carefully designed and constructed. To the average taxpayer, deficiencies (especially premature ones) observed in such a pavement, no matter how unavoidable they may be technically, suggest that the department either is not doing its job or does not know how to do its job.

Analysis of Sub-Surface Factors Affecting Rutting

Because rutting is not just a surface distress type, it is worth considering the subsurface factors that may contribute to the observed increases in rutting frequency.

Subgrade Support and Base/Surface Stiffness

FWD data collected for PMIS in 1987 provides an insight into underlying pavement properties that could contribute to the observed increase in rutting. A pavement's susceptibility to rutting can be assessed by measuring subgrade and base/surface properties. Weakness in any of these layers can set the stage for rutting to occur. Figure 6.5 shows the computed average subgrade support value for each county, as taken from FWD data collected on flexible mainlane sample sections. The subgrade support categories are defined in terms of subgrade modulus, as shown below:

Category	Modulus Values (psi)
"Very Poor"	0 – 12,999
"Poor"	13,000 – 17,999
"Fair"	18,000 – 22,999
"Good"	23,000 – 27,999
"Very Good"	28,000 – 199,999
STATE AVG.	20,652 All 20,495 Hot Mix

Major geologic features are readily apparent, especially the blue-shaded Balcones Escarpment, Llano Uplift, and Edwards Plateau regions of central Texas (the white regions indicate counties where subgrade support could not be computed, either due to the lack of deflection data or the absence of hot mix sections). Figure 6.5 demonstrates that subgrade support variations occur independently of district boundaries, as would be expected [TxDOT 88b].

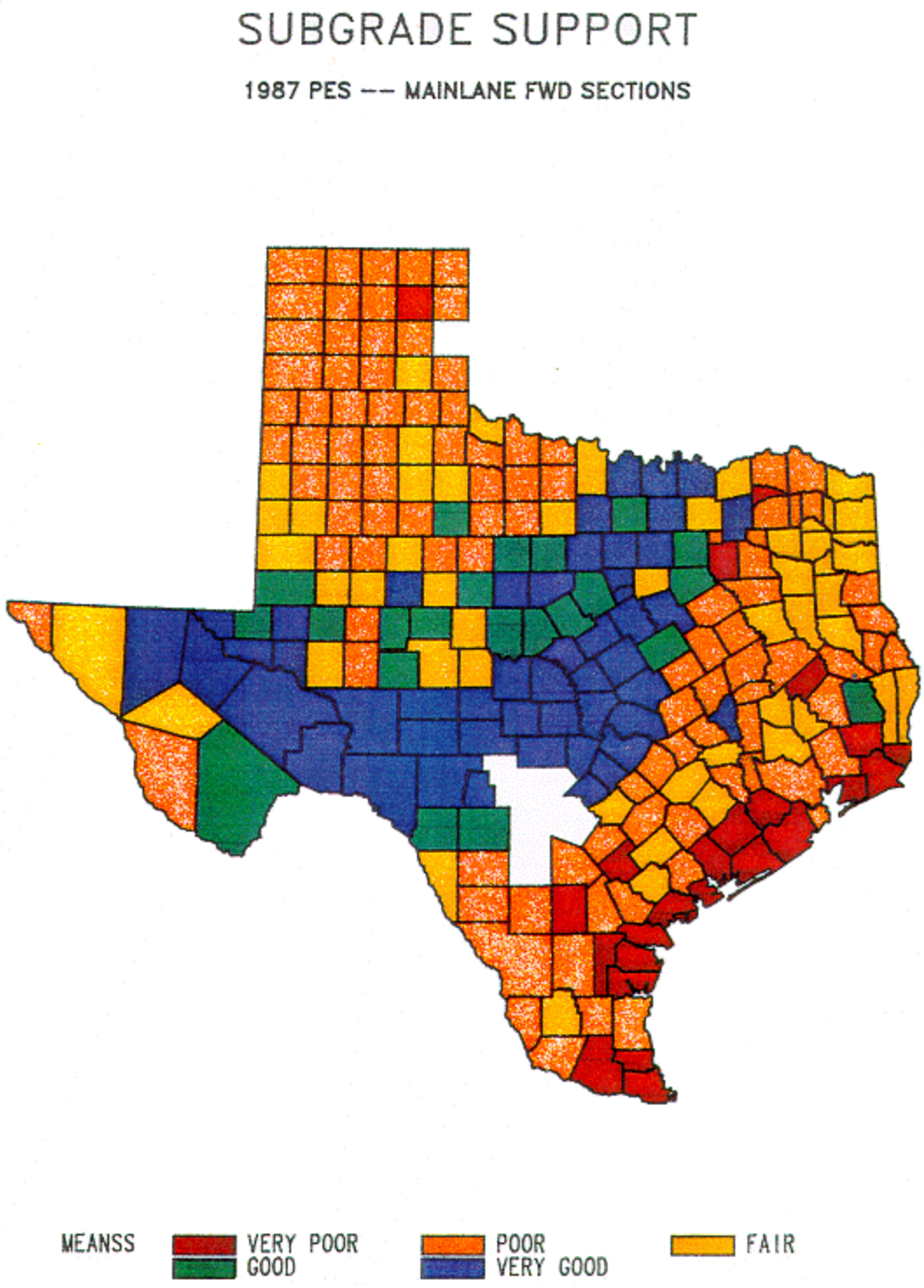


Figure 6.5 Average County Subgrade Support- Flexible Pavments

Base/surface properties may also be assessed using the 1987 FWD data. Figure 6.6 depict the observed variations in average county surface curvature index (SCI) for all flexible sections, which is a measure of base/surface stiffness. SCI categories were defined as shown below:

Category	Range of SCI Values
"Very Good"	0 – 6.99
"Good"	7.00 – 12.99
"Fair"	13.00 – 18.99
"Poor"	19.00 – 24.99
"Very Poor"	25.00 – 99.99
STATE AVG.	16.58 All
	12.24 Hot Mix

The base/surface stiffness values reflect the relative quality of the "man-made" roadbed materials, as opposed to the "natural" subgrade materials. The SCI values do display some of the same trends as the subgrade values, suggesting areas where local subgrade materials are being used for base.

Summary of Subsurface Factors

As expected, the subgrade support values paralleled major geologic trends. Although the hot mix data for subgrade support is somewhat limited by the absence of hot mix sections in central Texas, it is clear that the subgrade properties are relatively unaffected. The base/surface stiffness plots provide the most interesting insight, where the preponderance of "Good" values in the hot mix plot suggests the widespread use of stiff upper layers in hot mix pavements.

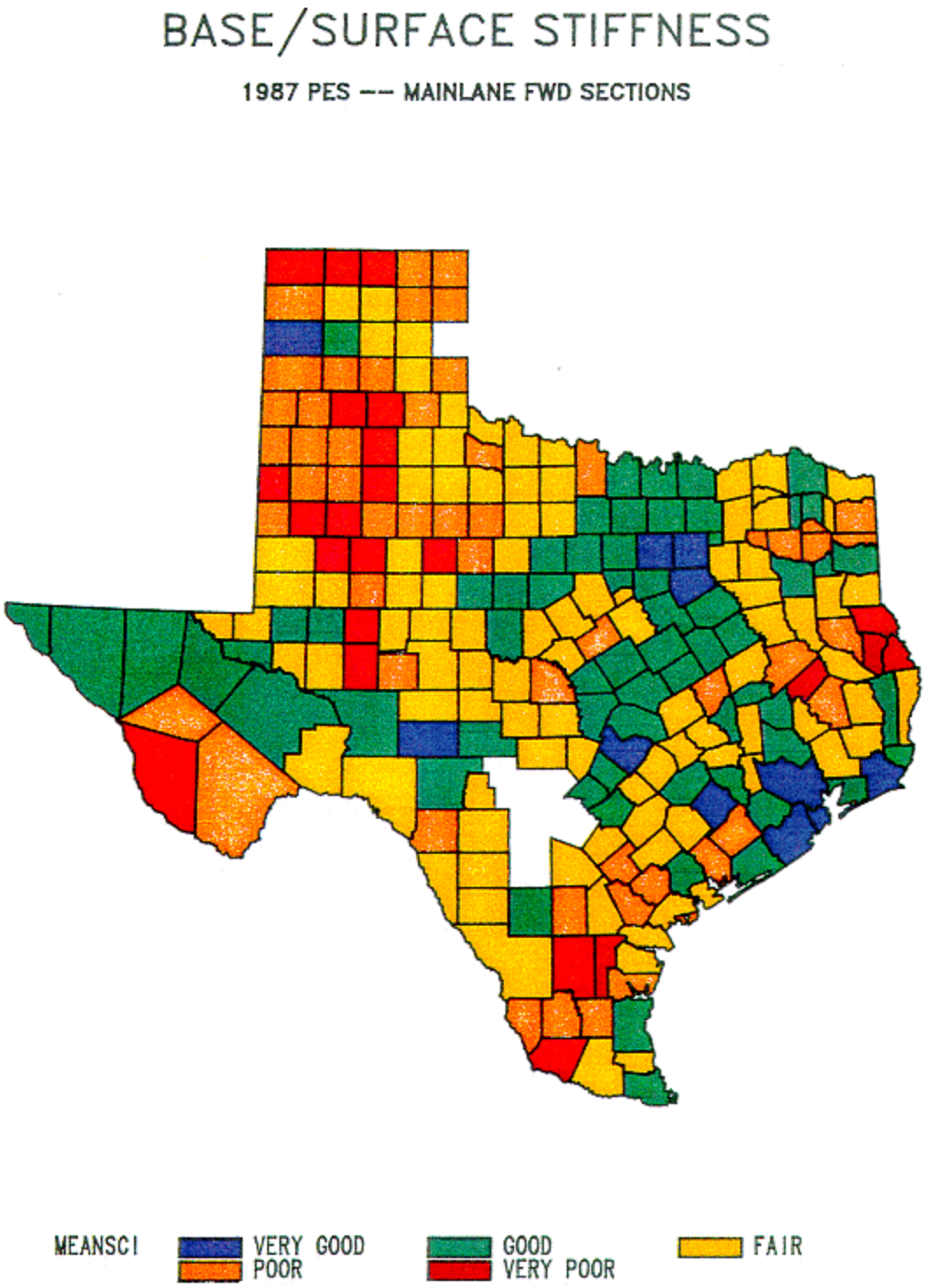


Figure 6.6 Average County Base/Surface Stiffness-Flexible Pavements

Analysis of Surface Rutting

As shown earlier in Figure 6.4, rutting was more frequently observed in 1987 than in the previous four years, especially on hot mix pavements. Because the PMIS ratings distinguish between ½" and 1" ruts, it is possible to determine how much of the observed rutting falls within each group. Figure 6.7 demonstrates, for example, that the observed rutting frequency of 33.09% for all flexible pavement sections in 1987 is made up of 29.08%, ½" rutting and 4.01%, 1" (or deeper) rutting. The general trends however, are the same for both rut depths – the larger changes in the 1" rut graph are due to its reduced vertical scale.

Maps of Rutting on Flexible Pavements

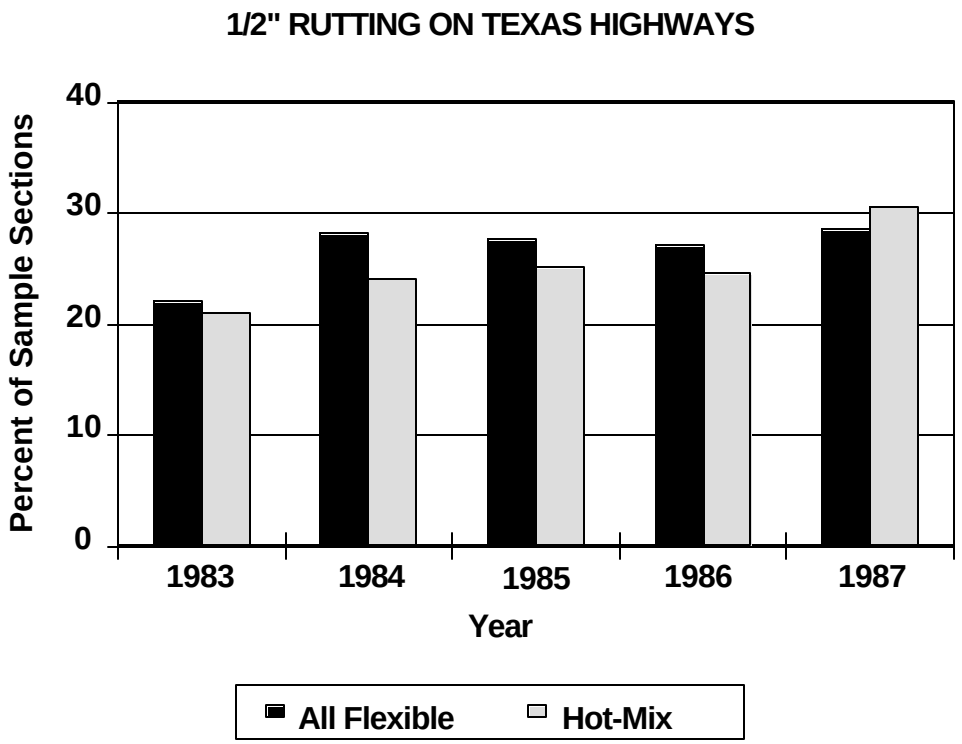
The study showed rutting values scattered widely across the state, with little apparent pattern. However, closer examination revealed that certain areas do typically display more rutting than others. What is especially disturbing is that these areas are so closely aligned with district boundaries, in some cases exactly so. Partial, or even entire, districts were identified in the study. In 1983, the number of districts that could be said to have been in control of their rutting outnumbered those that were being overwhelmed by rutting. By 1987, that trend had been completely reversed (Figure 6.8).

The study also suggests that rutting is persistent in its occurrence (and reoccurrence). For example, rutting in Northwest Texas was almost total by 1985. The improvements made in 1986 however, do not appear to have completely solved the basic problem, as suggested by the 1987 increases. South central Texas displays a similar trend.

In general, the data indicates that, although subsurface structural problems would contribute to the observed increases in rutting frequency, there are a sufficient number of stronger areas to suggest that surface layer deficiencies are also responsible.

Reliability of PMIS Rutting Data

Each annual PMIS survey includes a one-week audit of the pavement distress ratings, in which raters from each district travel to a neighboring district. The PMIS Audit consists of comparing the distress ratings and scores from the visiting "audit" team with those of the home "district" team. Analysis of data from the 1986 and 1987 PMIS audits provides a measure of the reliability of the rutting trends documented in this report. Errors in the results could be due to bias in either the audit sample or the rutting ratings. Both of these possible sources are examined below.



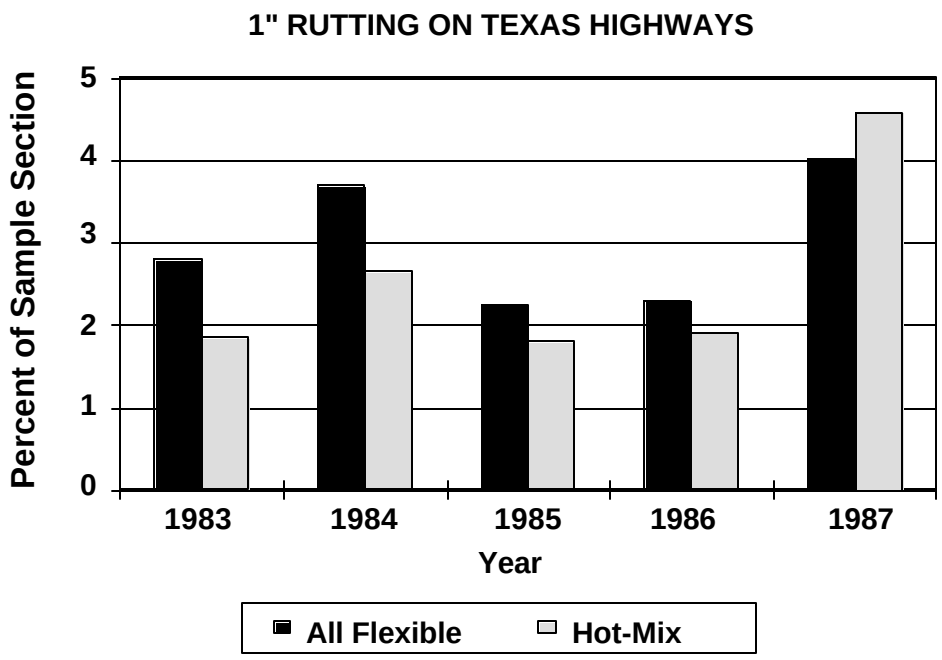


Figure 6.7 Rutting on Texas Highways by Rut Depth

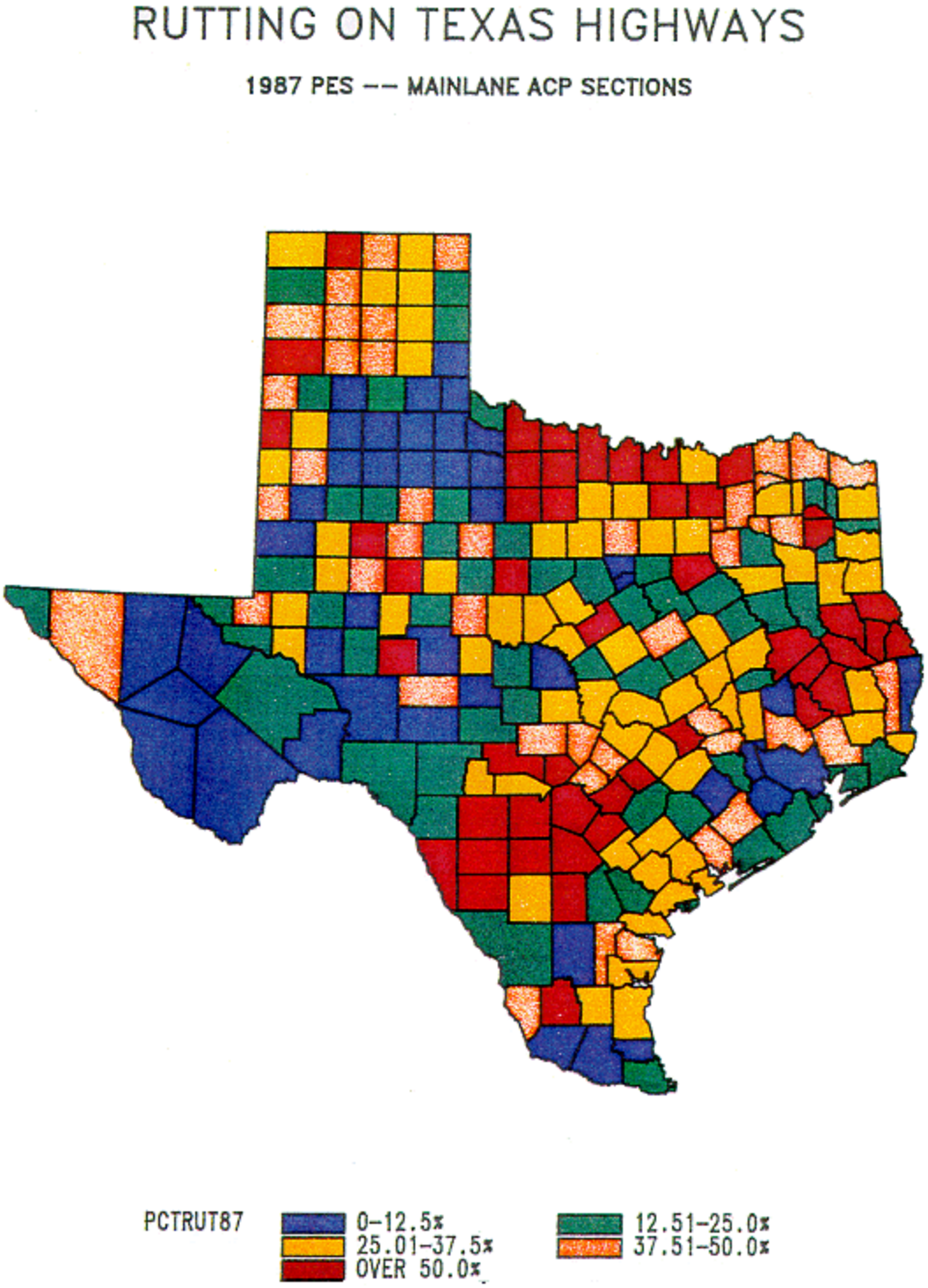


Figure 6.8 1987 Rutting on Flexible Pavments

Summary of this Engineering Application

Approximately 33% of the mainlane PMIS flexible pavement sections have measurable amounts of rutting (i.e., ruts at least ½" deep). Analysis of data collected between 1983 and 1987 shows a steady increase in the frequency of observed rutting – a large part of which occurred just in the last year. Even if the 1987 results are just an aberration (instead of the start of a higher plateau, as in 1984) the 33% rutting pavements in Texas is still a large one. Compared to this, Texas highways have a definite rutting problem – and have had one for some time.

Fortunately, only about 10% of the rutting observed since 1983 has been greater than 1" deep, however ruts in both depth categories follow the same increasing trend as all rutting.

Rutting on hot mix pavements is slightly less frequent than it is on all flexible pavements, except in 1987 when hot mix sections displayed far more frequent rutting. Again, this may be an aberration, however with so much "important" mileage potentially at risk, such an observation warrants careful consideration.

Review of FWD pavement structural data indicates large areas of the state where weak subgrade and base/surface layers are certainly aggravating the observed rutting. However, stronger subsurface layers also exist in high-rut areas, especially on hot mix pavements. This observation demonstrates the need for review of the mix design, materials, and construction techniques, which go into the pavement surface.

Without exact measurements of rut length and detailed maintenance histories of rutted sections, it is difficult to assess the true magnitude of the rutting problem on Texas highways. However, the current PMIS data clearly identifies the relative nature of the problem and is helpful in deriving first-cut solutions.

Asphalt Specification Changes

This study using the PMIS data on rutting as presented in detail in the previous sections was an important input in determining the need for asphalt and asphalt concrete specification changes. As a result, specification changes were made and a move was initiated toward performance-based specifications for asphalt concrete.

Currently additional work is being done to follow-up on the specification changes. This study will involve TxDOT personnel, the Asphalt Institute, the Association of General Contractors and the Federal Highway Administration. This steering committee will evaluate data presented to them from the PMIS database. This is an excellent example of the use of pavement management data to improve pavement design and performance. This follow-up study is currently underway.

6.4 ADOT'S EXPERIENCE WITH OVERLAY DESIGN TECHNIQUES

6.4.1 Introduction

The contents of this Section are based on a study that TRDI carried out for ADOT in 1998 [ADOT 99]. Part of that study was an evaluation of the implications of using AASHTO's DARWin (Design, Analysis, and Rehabilitation for Windows) to design overlays in Arizona, instead of the system used earlier.

6.4.2 Background

In the early eighties ADOT developed SODA (Structural Overlay Design for Arizona) with which they have been designing pavement overlays since. With the arrival of more modern and flexible alternative design options ADOT is interested in the possible use of one of these modern systems for Arizona, with particular interest expressed in the DARWin methodology that is based on the *1993 AASHTO Design Guide*. Additional background information on SODA and DARWin is given below.

Approach

The approach covered the following points:

1. A comparison has been made between SODA and DARWin for the structural design of overlays. The main features of both methods have been compared and discussed with regard to needs for 1) input data (DARWin uses different and more input parameters than SODA), 2) engineering applicability, and 3) flexibility (e.g., treatment of non-conventional materials, etc.).

2. SODA computer runs have been carried out using TRDI created SODA equation simulation software with several levels for the five input parameters. A sensitivity analysis for the parameters that are not used in DARWin, and vice-versa, has been made as discussed in Section 6.4.7.3.

3. DARWin runs have been carried out for a number of representative overlay design case studies where SODA was used by ADOT to design an overlay and where all parameters required for DARWin were also available or could be estimated well. Information was also made available on thicknesses actually constructed. Comparisons of these runs and those from Item 2 for SODA are made at the end of section 6.4.6 and compared with actual selected thicknesses.

6.4.3 The Development of SODA

The development of SODA has been documented by [Eisenberg 84]. ADOT acquired a Dynaflect in 1972 and started measurements soon thereafter. Originally use was made of an empirical relationship developed by California [CalTrans 79] which use the (center) deflection between the wheels of a Dynaflect and the traffic level to estimate the overlay thickness needed for a cracked AC pavement.

In the early eighties nine Interstate and thirteen non-Interstate projects were selected to develop a new overlay design method, which is called SODA (Structural Overlay Design for Arizona). Deflection data were available for all sites before and after an overlay was applied, as was PSI and performance data.

Data from the 24 sites was assembled into a regression matrix of 170-mile post locations and 31 variables, representing either physical measurements or calculated values of strains, moduli, etc. Mostly mechanistic models were applied for the calculations.

From the multiple regression Eisenberg established the following general form for the overlay equation:

$$Pt = f(R, D, Po, L, T)$$

where

Pt = a pavement performance characteristic over time t
R = an environmental factor
D = deflection measurements
Po = level of pavement performance characteristic before overlay
L = expected traffic loading after overlay during time t
T = AC overlay thickness

After substitution, combining terms, simplification and rearranging the following equation was obtained for SODA:

$$T = \frac{\log L + 0.104 \times R + 0.000578 \times Po - 0.0653 \times SIB}{0.0587 \times (2.6 + 32.0 \times D5)^{0.333}}$$

where

T = AC overlay thickness (inch)
L = number of 18 kip ESAL's (thousands)
R = Regional factor (AASHTO)
Po = Mays Ridemeter roughness (inches)
SIB= spreadibility index (related to shape of deflection bowl) before overlay
D5 = Dynaflect sensor # 5 deflection (mils)

As part of this development a sensitivity analysis of the independent variables in the SODA equation was carried out by ADOT about the relative effects on overlay thickness. This showed that L (ESAL's) and SIB (spreadibility index) each have double to triple the effect of any of the other three variables. D5 and Po both have an equally low impact on overlay thickness.

The following observations made by [Eisenberg 84] are worth noting:

1. It was recommended to use ride as the controlling failure mode. A value of 260 inches of roughness was selected as the pavement terminal roughness (this corresponds to a terminal serviceability of 2.5). Since the overlay was usually applied at a roughness level of 170 inches/mile (well before the terminal level was reached), its thickness tends to be on the conservative side.

2. The deflection under the load center is not used in SODA since the authors found that this variable was statistically insignificant during the multiple regression analysis. (With the previously used California-based method [CalTrans 79] this was the only deflection variable used).

3. The spreadibility index (SIB) alone does not sufficiently reflect the load carrying capacity of a pavement. SIB is a function of the ratio of the moduli of the various pavement layers, consequently a pavement with three weak layers can have the same SIB value as one with three proportionally stronger layers.

4. The structural properties or stiffness of the existing AC and the unbound pavement layers are not separately represented by an independent variable. The subgrade modulus on the other hand is directly related to D5.

5. In SODA the structural properties of the AC overlay (modulus) have been treated as a constant (200 ksi), but particularly with thick overlays the temperature effect on the pavement's bearing capacity can be significant.

6. SODA cannot address the structural requirements of recycling or the use of innovative materials.

6.4.4 Experience with and Limitations of SODA in Practice

During the development of SODA in the early eighties, and also during the following years a number of limitations, restrictions and changes were implemented in the SODA method by ADOT in order to get better results. Most of these have been documented in their engineering and design manual [ADOT 89]. The following points are relevant:

- Initially ADOT found that SODA occasionally produced unrealistically thick overlays, the range of thickness solutions was therefore constrained to less than six inches.
- If the value of D5 is less than 0.1 mil the overlay thickness becomes too large, if the D5 value exceeds 0.4 the overlay thickness will be underestimated. In both cases the designer was advised to compensate.

- Soon after the introduction of SODA ADOT acquired a Dynatest FWD. Since SODA was designed for Dynaflect deflection data, fixed conversion factors were devised and are used for FWD deflection data. This introduces potential error.
- The SODA overlay thickness equation was later revised by introducing corrections for design loads (L) and roughness (Po). The equation now reads as follows:

$$T = \frac{(\text{Log } L - 3.255) + (0.104 \times SVF) + [0.000578 \times (Po - 4.255) / 0.54] - (0.0653 \times SIB)}{0.0587 \times [2.6 + (32.0 \times D5)]^{0.333}}$$

where:

L	= Design 18 kip ESAL's
SVF	= Seasonal Variation Factor
Po	= Roughness, inches/mile
SIB	= Spreadability Index Before Overlay, for FWD: $SIB=2.7x(FWD\ SI)^{0.82}$
D5	= #5 Dynaflect sensor reading in mils, for FWD: $D5 =0.16x(FWD\ D7)^{1.115}$

- For the FWD the deflection level of the 7th sensor D7 is used to determine subgrade properties (D7 is converted to D5, the 5th geophone of the Dynaflect).
- When the design overlay resulting from SODA is 2.5 inches or less, or traffic loading is less than 100,000 18Kip ESAL's, a minimum thickness of 2.5 inches is selected for use, unless the roughness, percent cracking, lack of friction and/or rut depth data are in the low range. In such cases another rehabilitation treatment (seal coat, milling, friction course) etc., or "no action" can be chosen. In these cases the decision also depends on the annual maintenance cost of the road section concerned (low annual cost: no action, high annual cost: minimum two inches overlay). There is no systematic treatment of these decision rules.
- The effect of milling (reducing pavement thickness) is currently incorporated in the thickness design of the overlay
- For rigid pavement rehabilitation, deflection testing is also conducted, but instead of SODA a graph and a table are used to estimate and determine the degree of structural damage and the type of rehabilitation to be considered.

6.4.5 DARWin as an Overlay Design Technique

DARWin is a computerized release of the pavement design models presented in the *AASHTO Guide for the Design of Pavement Structures* [AASHTO 93]. It has modules for flexible and rigid structural design, for overlay design, and for life cycle cost. It also contains a library with material cost, pay items, etc.

With the overlay design module seven different types of overlays can be designed using up to three different methods. The overlay design types are:

- AC overlay of AC pavement
- AC overlay of fractured PCC slab
- AC overlay of PCC pavement
- AC overlay of AC/PCC pavement
- Bonded PCC overlay of PCC pavement
- Unbonded PCC overlay of PCC or AC/PCC pavement
- PCC overlay of AC pavement.

For the first and most commonly used overlay design type, AC overlay on an existing AC pavement, the thickness design is based on the difference between the structural number required to carry the projected future traffic loadings (SN_f) and the calculated effective existing pavement structural number (Sn_{eff}). SN_f is determined using the same procedure used for the design of new AC pavements. A value can be entered directly by the user, or calculated with DARWin from inputs such as ESAL's, initial and terminal serviceability and design resilient modulus. Sn_{eff} can be determined with three evaluation methods:

1. The Component Analysis Method, in which the structural number equation is solved using actual drainage coefficients and layer thicknesses and modified structural coefficients. DARWin provides suggested coefficients for existing materials.

2. The Remaining Life Method, based on the concept of reducing the structural capacity of the pavement due to accumulated fatigue damage. The actual loadings carried are compared to the total loadings the pavement was designed to carry to its terminal serviceability and this ratio is subtracted from 100 to give a percent remaining life (RL). The RL is used to obtain a condition factor (CF) using info from the AASHTO Guide. The Sn_{eff} is the product of CF and the SN of the pavement when new.

3. The Nondestructive Testing Method, where Sn_{eff} is determined with the assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The effective pavement modulus is a backcalculated measure of the structural contribution of the existing pavement. Its calculation requires a subgrade resilient modulus and deflection measurements obtained with an FWD. Milling is taken into account when calculating the structural number.

When using all inputs, DARWin calculates the structural number of the overlay (Sn_{ol}) as the difference between SN_f and Sn_{eff}. The thickness of the overlay (D_{ol}) is obtained by dividing Sn_{ol} by the structural coefficient a_{ol} of the overlay material.

The *AASHTO DARWin User's Guide* recommends the use of the nondestructive testing method in all cases, but suggests results be regularly checked with the other two methods. Although the overlay thickness values should be similar, there is no reason to expect them to be identical.

For the design of flexible pavements the properties of asphalt layers are characterized with "layer coefficients" that are based on their elastic (resilient) modulus at a temperature of 68°F. Consequently, the significant effects of temperature on the asphalt modulus are not fully taken into account. It also means that the thickness for innovative materials that also demonstrate temperature-dependent properties in the design method will produce questionable results if there is not good empirical data to predict the layer equivalency of such materials.

For the design of AC overlays for AC pavements based on deflection measurements, the temperature of the AC layer should be determined at the time of testing. During the analysis of the deflection measurements the deflection at the center of the load plate is adjusted to a standard temperature of 68°F. No information is given on the adjustment requirements for innovative materials.

6.4.6 Comparison between SODA and DARWIN

General

SODA has the advantage of being calibrated to actual ADOT experience over two decades, however its overlay thickness results depend on only five parameters and are insensitive to some situations. Since DARWin has more inputs it can be used with more sensitivity to varied conditions. Calibrating DARWin to SODA would combine

ADOT's experience with SODA with results from DARWin, and obtain the flexibility and additional sensitivity of DARWin for ADOT.

Table 6.14 gives an overview of the two different overlay design calculation techniques. For DARWin the methodology is given for the non-destructive testing technique.

From this overview it can be seen that both methods are sensitive to ESAL's, regional factors and deflection basin. SODA is sensitive to roughness prior to overlay while DARWin is not. DARWin is sensitive to pavement thicknesses, initial and terminal serviceability and several other parameters and coefficients while SODA is not.

Comparison in Sensitivity to Input Parameters

SODA is a thickness formula composed of five influences each of which are a function of these independent variables:

1. CESALs cumulative 18 kip equivalent axles (10 years),
2. SIB deflection spreadability index,
3. SVF environmental factor,
4. Po roughness prior to overlay, and
5. D5 subgrade deflection

Table 6.14 Comparison between Overlay Design Equations for SODA and DARWin

SODA OVERLAY DESIGN	DARWin OVERLAY DESIGN
$T = \frac{\log L + a \times R + b \times Po - c \times SIB}{d \times (e + g \times D5)^{0?333}}$ where: T = AC overlay thickness (inch) L = number of 18 kip ESAL's (thousands) R = Regional factor (AASHTO), later called SVF (Seasonal Variation Factor) Po = Mays Ridemeter or Profilometer roughness prior to overlay (inches per mile) SIB= Dynaflect spreadibility index (related to shape of deflection bowl) before overlay. For FWD: $SIB = 2.7 \times (FWD\ SI)^{0.82}$ D5 = Dynaflect sensor #5 deflection (mils). For FWD: $D5 = 0.16 \times (FWD\ D7)^{1.115}$ a,b,c,d,e and g are constants	$SN_{0l} = a_{0l} \times D_{0l} = SN_f - Sn_{eff}$ where: Sn_{0l} = Required overlay Structural Number (inch) a_{0l} = Structural coefficient for AC overlay D_{0l} = Required overlay thickness (inch) SN_f = SN required for future traffic SN_{eff} = Effective SN of existing pavement Determine SN_f from: 1. Nr of 18 kip ESALs in design lane over design period 2. Effective design subgrade resilient modulus (FWD, lab, graphs) 3. Initial and Terminal serviceability 4. Overlay design reliability R (percent) 5. Overlay standard deviation Determine Sn_{eff} from: 1. FWD testing (calculate effective modulus of all pavement layers and of subgrade) 2. Total thickness of pavement layers above subgrade 3. Include the effect of milling. Determine a_{0l} for overlay material (0.44 for AC at 68 ºF).

All of these variables except Po are also used in DARWin. In order for DARWin and SODA to be compatible, each of these independent variables must exert the same type of influence in both methods. In comparing SODA to DARWin the main question is "Are the influences of these variables compatible"? The answer is "No", because the effect of the D5 variable in SODA is counter intuitive and cannot be aligned with the DARWin effects. Table 6.15 compares the effects.

Because of the incompatibility of deflection between SODA and DARWin a direct mathematical comparison is not valid.

Table 6.15 Comparison of Effects of Variables on Thickness

#	Variable	Effect on thickness in SODA	Effect on thickness in DARWin	Compatible?
1	CESALs	Increasing CESALs (more expected traffic) produces thicker overlays	Increasing CESALs (more expected traffic) produces thicker overlays	Yes

2	SVF	Increasing SVF (poorer environment) produces thicker overlays	Increasing SVF (poorer environment) produces thicker overlays	Yes
3	P0	Increasing P0 (more roughness prior to overlay) produces little or no effect on thickness	P0 (more roughness prior to overlay) produces no effect on thickness	Yes
4	SIB	Decreasing SIB (weaker pavement prior to overlay) produces thicker overlays	Decreasing SIB (weaker pavement prior to overlay) produces thicker overlays	Yes
5	D5	Decreasing D5 (stronger subgrade) produces thicker overlays	Decreasing D5 (stronger subgrade) produces thinner overlays	**No**

Analysis of SODA

The method used to investigate the SODA thickness formula was to split it by components and then estimate the effect of each component on the thickness.

In general, the SODA thickness formula is the sum of four of these influences, all of which is multiplied by the fifth influence. The SODA thickness formula is of the algebraic form

$$T = (B_1 + B_2 + B_3 + B_4) * A$$
$$T = B_1*A + B_2*A + B_3*A + B_4*A$$
$$T = \Sigma B_i * A,$$

where:

T = thickness
$B_1 = \log_{10} CESAL - 3.255$
$B_2 = 0.0653 * SIB$
$B_3 = 0.104 * SVF$
$B_4 = 0.00107 * (Po - 4.2)$
$A = [0.0587 * (2.6 + 32 * D5)^{0.333}]^{-1}$

The "A" influence has a multiplicative effect on thickness. For a given "A", the B_1, B_2, B_3, B_4 influences each have an additive effect on thickness.

Trends

The definitive trend on thickness for each variable is as follows:

1. Increasing CESALs (more expected traffic) produces thicker overlays
2. Increasing SVF (poorer environment) produces thicker overlays
3. Increasing Po (more roughness prior to overlay) produces thicker overlays
4. Decreasing SIB (weaker pavement prior to overlay) produces thicker overlays
5. Decreasing D5 (stronger subgrade) produces thicker overlays

Four of these trends are expected and can also be found in DARWin, however number 5, the D5 influence, is counter-intuitive.

Sensitivity

The effect on thickness of each influence is as follows.

1. Influence of the variable CESALs is B_1 * A where "A" is set at a given value
2. Influence of the variable SVF is - B_2 * A where "A" is set at a given value
3. Influence of the variable Po is B_3 * A where "A" is set at a given value
4. Influence of the variable SIB is B_4* A where "A" is set at a given value
5. Influence of the variable D5 is ΣB_i * A where every "B_i" is set at a given value

To estimate the maximum effect of a variable on thickness, the thickness for a low value of the variable was calculated, and then at a high value of the variable for the same "given set" of the other variables. The difference of these two thicknesses is the maximum effect of the variable for the "given set" of the other variables. The maximum effect is found when the given set of variables is set for the highest thickness effect. We show the full range of maximum effects by estimating each effect

- Where the given variables are set to produce large thicknesses
- Where the given variables are set to produce small thicknesses

These values are given in Table 6.16.

Table 6.16 Values of variables for small and large thickness in SODA

	Produce small thicknesses	**Produce large thicknesses**
CESAL	100,000	10,000,000
SIB_{Dynaf} (SIB_{FWD})	98 (80)	50 (35)
SVF	1	4
Po	50	175
$D5_{Dynaf}$ ($D7_{FWD}$)	0.1 (0.67)	0.4 (2.4)

The effects of introducing the "low" and "high" influence values are listed in Table 6.17.

The results from Tables 6.16 and 6.17 attest to the conclusions documented at the beginning of this section, in particular:

- The maximum thickness effect of Po (roughness prior to overlay) is very small
- A weak subgrade (D5=.4) yields thinner overlays than a strong subgrade (D5=.1), whenever the other four variables CESALs, SIB, SVF, Po are held constant

Table 6.17 Effect of "Low" and "High" Influence Values on Thickness in SODA Calculations

(1) value	"low influence" D5 level	(2) "low influence" value	(3) effect of the "low influence" value (B_i * A) on thickness	(4) "high influence" value	(5) effect of the "high influence" value (B_i * A) on thickness	(6) max net effect on thickness for D5=.4 (5) - (3)
CESAL	.4	100,000	11.76"	10,000,000	25.24"	13.48"
SIB_{Dyn}	.4	98	-43.2"	50	-21.93"	-21.27"
SVF	.4	1	.701"	4	2.804"	2.1"
Po	.4	50	.33"	175	1.23"	.9"

Table 6.17 Effect of "low" and "high" influence values on thickness in SODA calculations (continued).

(1) value	"high influence" D5 level	(2) "low influence" value	(3) effect of the "low influence" value on thickness	(4) "high influence" value	(5) effect of the "high influence" value on thickness	(6) max.net effect on thickness for D5 = .1 (5) - (3)
CESAL	.1	100,000	16.48"	10,000,000	35.38"	18.9"
SIB_{Dyn}	.1	98	-60.54"	50	-30.74"	-29.8"
SVF	.1	1	.9824"	4	3.9296"	2.95"
Po	.1	50	.4625"	175	1.7264"	1.27"

(1) value	"high influence" level of given values of CESALs, SIB, SVF, Po	(2) "low influence" value	(3) effect of the "low influence" value on thickness	(4) "high influence" value	(5) effect of the "high influence" value on thickness	(6) max.net effect on thickness for "high given values" Col (5) - (3)
D5	10000000, 50, 4, 175	.4	7.35"	.1	10.29"	2.94"

(1) value	"low influence" level of given values of CESALs, SIB, SVF, P0	(2) "low influence" value	(3) effect of the "low influence" value on thickness	(4) "high influence" value	(5) effect of the "high influence" value on thickness	(6) max. net effect on thickness for "low given values" Col (5) - (3)
D5	100000, 98, 1, 50	.4	-30.41"	.1	-42.62"	-12.21"

Comparison for Historic Overlay Projects in Arizona

ADOT forwarded information about 34 historic SODA overlay design projects carried out between 1986 and 1998. A list with details of these projects, such as route number, milepoints, region, date of report, NDT testing method (Dynaflect or FWD) and design of the overlay, is given in Table 6.18.

For a number of these projects TRDI carried out runs with DARWin to determine the overlay thicknesses as projected by DARWin in comparison with the SODA projections and with the actual thicknesses selected by ADOT engineers. A total of 22 DARWin runs were carried out to completion while dozens more were run in preparation and testing the levels of test variables.

The DARWin runs were carried out using the "Non-Destructive Testing" program; the following input variables were taken from the project information:

- Layer thicknesses (granular, asphalt) prior to overlay
- Temperatures at time of testing
- Future 18-kips ESAL's over the design period
- Deflection data (average values were used as in SODA)
- Milling thicknesses

For the following inputs the recommended values from the DARWin Guide and the AASHTO Design Method were used:

- Initial Serviceability: 4.0-4.5
- Terminal Serviceability: 2.5
- Reliability Level: 90 or 95%
- Overall Standard Deviation: 0.34-0.39
- Resilient Modulus Correction Factor: 0.33
- FWD data converted to 9000 lbs, plate radius 5.9”
- Structural Coefficient for the Overlay Material a_{0l} = 0.44 (relation between Structural Number and Overlay Thickness)

Table 6.19 gives the results of the DARWin runs, together with the original results from SODA, and the actual overlay thicknesses selected by the ADOT design team.

It is difficult to discern the exact trends from Table 6.19. Therefore the results have been grouped into three categories:

1. Projects where SODA and DARWin gave a similar thickness (Table 6.20)
2. Projects where SODA yielded a greater thickness (Table 6.21)
3. Projects where SODA gave a smaller thickness (Table 6.22)

In the first and the third projects in Table 6.20 the calculation assumed milling, whilst a choice was made not to mill. In the second, fourth and fifth projects the reverse situation occurred. There is therefore no comparison between the calculated outputs and the actual overlay thickness selected by ADOT designers.

Table 6.18 Representative ADOT Overlay Projects during Period 1986-1998

SODA Overlay Design Projects							
Project	Route	Begin mp	End mp	Region	Date	Design of pavement section	NDT
						(explanation in table 6.19)	method
IM-40-1 (82)	I-40	54.0	62.8	Transition	3/17/95	5’’R&R+3’’AC+AR-FC	FWD
RS-316(22)	SR-177	157.5	164.6	Desert	6/11/91	2" Olay+ SC	FWD
F-069-1-902	SR-277	305.8	312.4	Mountains	12/24/86	1.5"AC + SC	Dynaflect
F-060-1-510	SR-264	403	426	Mountains	1/21/92	1.5"AC+1"ARAC, PBA-3	FWD
F-051-2(33)	US-191	160	162.1	Transition	8/24/89	2"R&R+2"Olay+SC	No Data
F-037-3-507	US89A	524	532	Mountains	7/23/90	2" AC + SC	FWD
F-035-1-512	US-93	91.5	95.8	Transition	10/9/91	2.5" AC + FC	FWD
IM-10-1(87)	I-10	11.7	24	Desert	12/18/97	5"R&R+2.5"AC+AR-FC	FWD
IM10-5(468)	I-10	290.2	296.2	Desert	6/19/96	4"R&R+3"AC+AR-FC	FWD
F-026-1(21)	US-60	255.8	260.2	Mountains	6/4/86	2.5"R&R+2"AC Recycle+SC	Dynaflect
F-016-1-922	SR-80	332	339	Transition	2/26/87	2"AC+SC	Dynaflect
IR-10-3(232)	I-10	195.1	200	Desert	7/23/87	3"R&R+2"AC+½” FC	Dynaflect
IR-10-6(88)	I-10	331	336	Transition	7/23/87	3"R&R+2"AC+½” FC	Dynaflect
IR-40-4(122)	I-40	224.7	239.6	Transition	9/17/87	2"R&R+2.5"AC+½” FC	Dynaflect
IR-17-2(32)	I-17	290	299	Transition	12/16/87	3"R&R+2"AC+½” FC	Dynaflect
IR-8-1(86)	I-8	29	37.1	Desert	6/24/88	3.5"R&R+2"AC+½” FC	FWD
IR-10-6(101)	I-10	310	316	Transition	7/22/88	4"R&R+2.5"AC+½” FC	FWD
IR-15-1(36)	I-15	0	8	Desert	8/9/88	3.5"R&R+3"AC+½” FC	FWD
IR-10-2(131)	I-10	80.0	90.0	Desert	8/29/88	3.5"R&R+2"AC+½” FC	FWD
IR-8-1(91)	I-8	72.0	82.0	Desert	3/29/89	4.5”RE+4”AC+¾”FC	FWD
IR-10-5(66)	I-10	272.0	275.5	Desert	3/27/90	3"R&R+3"AC+½” FC	FWD
IR-40-5(92)	I-40	307.3	318.7	Transition	7/2/91	5"R&R + 2”AC+½” AR-FC	FWD
IR-40-4(139)	I-40	249.8	259.6	Desert	9/23/91	3”RE+2.5”AC+½” FC	No Data
IR-40-4(141)	I-40	268.0	277.4	Transition	6/26/92	3”RE+2.5”AC+½” FC	FWD
IM17-1(228)	I-17	245.0	250.4	Transition	7/29/96	3.5”RE+3”AC+½” FC	FWD
STP060-1(6)	S-264	340	344	Transition	3/6/97	2"R&R+2"AC+DC	FWD
IM-40-1(83)	I-40	8.3	16	Desert	3/12/97	5"R&R + 3”AC+½” AR-FC	FWD
STP026-1(44)	US-60	252.61	255.84	Transition	9/5/97	2”AC+½” AR-FC	FWD
STP022-3(83)	US-60	208	212	Desert	10/16/97	2.5”RE+2”AC+½” AR-FC	FWD
IM17-1(329)	I-17	232	238	Desert	12/18/97	4.5”RE+6.5”AC+½” AR-FC	FWD
STP013-1(15)	SR-80	328.6	336.3	Transition	12/24/97	3”AC+SC	FWD
STP035-1(47)	US-93	177.08	182.43	Desert	2/4/98	2"R&R + 2”AC+½” AR-FC	FWD
IM-40-2(122)	I-40	86.2	95.4	Transition	3/12/98	4.5”RE+4”AC+½” AR-FC	FWD
IM-40-2(122)	I-40	95.4	103.1	Transition	3/12/98	5.5”RE+5”AC+½” AR-FC	FWD

Table 6.19 Results of DARWin and SODA Calculations, Compared with Actually Applied Thicknesses

Project	Lane	Treatment	DARWin	SODA	Actual Overlay
Based on FWD deflection data					
IM-40-1 (82) Interstate	W 1	No milling	2.8”	0”	3” AC + ½” AR-FC
		5” milling	5.5”	5”+0.2”	
	W 2	No milling	6.6”	5.6”	5”R&R+3” AC + ½” AR-FC
		5” milling	9.0”	5”+2.5”	
	E 1	No milling	3.6”	0”	3” AC + ½” AR-FC
		5” milling	6.0”	5+0.1”	
	E 2	No milling	6.9”	5.6”	5”R&R+3” AC + ½” AR-FC
		5” milling	9.2”	5”+2.7”	
IM-10-1(87) Interstate	E 1	3” milling	0.9”	3.5”	3”R&R+2.5” AC + ½” AR-FC
	E 2	5” milling	4.4”	> 6”	5”R&R+2.5” AC + ½” AR-FC
	W 1	3” milling	0.8”	3.1”	3”R&R+2.5” AC + ½” AR-FC
	W 2	5” milling	4.8”	>6”	5”R&R+2.5” AC + ½” AR-FC
IM-40-2(122)	E-1	No milling	0”	0.3”	3”RE+2.5”AC+½” AR-FC
	E 2	No milling	0”	3.1”	4.5”RE+4”AC+½” AR-FC
	W 1	No milling	0”	0.1”	3”RE+2.5”AC+½” AR-FC
	W 2	No milling	0”	3.9”	4.5”RE+4”AC+½” AR-FC
RS-316(22)	N	No milling	4.9”	1.1”	2” AC + SC
STP-13-1(15)	E 1	No milling	1.1”	4.2”	3” AC + chip seal
STP-35-1(47)	S 1	No milling	5.1”	3.0”	2”R&R+2”AC+½” AR-FC
F-037-3-507	1	No milling	3.4”	1.8”	2” AC + chip seal
F-035-1-512	1	No milling	5.9”	2.3”	2.5” AC + ½” FC
Based on Dynaflect deflection data					
F-069-1-902	N 1	No milling	0”	1.7”	1.5” AC + chip seal

Codes used for the overlays are:

AC: Asphalt concrete
R&R: Remove and replace
AR-FC: Rubberized AC friction course
SC: Sealcoat
FC: Asphalt Concrete Friction Course
RE: Remove (mill)
DC: Double Chip Seal

Table 6.20 Projects where SODA& DARWin came up with Similar Overlay Thickness (plus or minus 1 inch)

Project	Lane	DARWin	SODA	Actual Overlay Thickness
IM-40-1 (82) Interstate	W 1	5”RE+5.5”AC	5”RE+5.2”AC	3” AC + ½” AR-FC
	W 2	6.6”AC	5.6”AC	5”RE+8” AC + ½” AR-FC
	E 1	5”RE+6.0”AC	5”RE+5.1”AC	3” AC + ½” AR-FC
IM-40-2 (122) Interstate	E 1	0”AC	0.3”AC	3”RE+2.5”AC+½” AR-FC
	W 1	0”AC	0.1”AC	3”RE+2.5”AC+½” AR-FC

For the first project in Table 6.21 (IM-10-1) the design engineers selected for all four lanes overlay thicknesses that were greater by an average of one to two inches than calculated with SODA, and by an average of three to five inches greater than calculated with DARWin. For the second project (IM-40-2) a comparison between calculated and actual thickness is difficult, because for the actual situation milling was carried out, but DARWin seems to be much closer to the actual selection than SODA. For the last project (STP-13-1) the actual thickness was less than calculated by SODA by 1.2 inches, but greater than calculated by DARWin by two inches.

Table 6.21 Projects where SODA Yields a Larger Thickness than DARWin

Project	Lane	DARWin	SODA	Actual Overlay Thickness
IM-10-1(87) Interstate	E 1	3”RE+0.9”AC	3”RE+3.5”AC	3”RE+5.5” AC + ½” AR-FC
	E 2	5”RE+4.4”AC	5”RE+> 6”AC	5”RE+7.5” AC + ½” AR-FC
	W 1	0.8”AC	3.1”AC	5.5” AC + ½” AR-FC
	W 2	4.8”AC	>6”AC	7.4” AC + ½” AR-FC
IM-40-2(122) Interstate	E 2	0”AC	3.1”AC	4.5”RE+4”AC+½” AR-FC
	W 2	0”AC	3.9”AC	4.5”RE+4”AC+½” AR-FC
STP-13-1(15)	E 1	1.1”AC	4.2”AC	3” AC + chip seal

It is interesting to note from Table 6.22 that in almost all these type three cases the design engineers chose thicknesses greater than required by SODA, and were often very close in their engineering judgment to the thickness prescribed by DARWin.

Table 6.22 Projects where SODA came up with a Lower Thickness than DARWin

Project	Lane	DARWin	SODA	Actual Overlay Thickness
IM-40-1 (82)	W 1	2.8”AC	0”AC	3” AC + ½” AR-FC

	W 2	5”RE+9.0”AC	5”RE+7.5”AC	5”RE+8” AC + ½” AR-FC
	E 1	3.6”AC	0”AC	3” AC + ½” AR-FC
	E 2	6.9”AC	5.6”AC	5”RE+8” AC + ½” AR-FC
	E 2	5”RE+9.2”AC	5”RE+7.7”AC	5”RE+8” AC + ½” AR-FC
RS-316(22)	N	4.9”AC	1.1”AC	2” AC + SC
STP-35-1(47)	S 1	5.1”AC	3.0”AC	2”RE+4”AC+½” AR-FC
F-037-3-507	1	3.4”AC	1.8”AC	2” AC + chip seal
F-035-1-512	1	5.9”AC	2.3”AC	2.5” AC + ½” ACFC

When one takes the average thicknesses from all tables for the thirteen runs where the three thicknesses can be compared (i.e. milling or no milling for both calculated thicknesses and for actual thickness) the following emerges:

- Average thickness for actually selected overlays: 4.5 inches
- Average thickness for SODA calculations 3.5 inches
- Average thickness for DARWin calculations 3.9 inches

It can thus be concluded that for these projects on average the thicknesses found with DARWin are between the SODA recommendations and the actual thicknesses applied.

As stated earlier for these comparisons a selection was made from the total number of available projects. There are three general reasons why these analytical comparisons were not carried out for all 34 projects:

- As previously explained a detailed one-to-one comparison between SODA and DARWin is not possible, there is little added value of carrying out lots of additional comparisons.
- The results of the SODA calculations were not readily available in electronic format. Consequently the data had to be fed to DARWin manually, this is certainly possible, but as stated in the DARWin Guide, a “daunting task”.
- The DARWin program, through its “Pavement Deflection Data Exchange”, offers data format specifications for five different FWD’s (JILS-20, Phonix ML1000, Dynatest Editions 20 and 25, and KUAB 2m) and one vibratory load device (Road Rater), but not for the Dynaflect. The load configuration data were put in manually to the best of our ability, but the output data could be suspect. Consequently only one run was carried out for a project based on Dynaflect data.

6.4.7 Advantages of DARWin over SODA

- DARWin is a comprehensive system for designing new flexible or rigid pavements, different types of overlays and life cycle cost analysis of pavements. It offers full integration of overlay design in overall pavement design, and it is entirely based on the AASHTO design philosophy while SODA is a stand-alone system for overlay design only.
- DARWin can be used for flexible and rigid pavements and for combinations such as AC overlays of various types of PCC pavements, and PCC overlays over asphalt pavements, while SODA only works for AC overlays on asphalt pavements
- DARWin offers considerable flexibility. Various types of equipment can be used for non-destructive testing, and it is also possible to design overlays without NDT.

- By adjusting input variables such as reliability level, resilient modulus correction factor, overall standard deviation, and structural coefficient for the overlay, the effects of special circumstances and or innovative materials can be incorporated.
- Through "Pavement Deflection Data Exchange" (PDDX) Dynatest FWD deflection data can be used without conversion while SODA was developed for the Dynaflect and a Dynatest FWD conversion factor is used. Dynaflect data can also be entered into DARWin through PDDX once a reliable load configuration format is available.
- Temperature effects are taken into account in DARWin where a layer equivalency factor is adjusted for temperature and used in the thickness design of the overlay.
- Remaining pavement life is directly incorporated in the overlay DARWin design methodology.
- DARWin runs in a modern Windows environment that is fully compatible for use on a PC with or without networking, whilst SODA is a dated DOS application that needs a connection to a mainframe computer.

Although there seem to be many advantages of DARWin one should not fail to overlook the value of extensive practical experience gained by ADOT with SODA.

6.5 REFERENCES FOR MODULE 6

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ATTACHMENT

6.6 WSDOT OVERLAY SCOPING TECHNIQUE

6.6.1 Why?

Until the advent of the technique described in this appendix, the WSPMS did not "scope" project specific pavement rehabilitation. What was done instead was to "apply" a "standard fix" to each pavement segment requiring rehabilitation (based on a pre-selected minimum Pavement condition Rating (PCR) level). The goal of the scoping technique is to incorporate into WSPMS an asphalt concrete overlay thickness estimate for each pavement segment requiring rehabilitation (or BST requirement). This will provide, hopefully, more realistic initial rehabilitation estimates from WSPMS.

6.6.2 How?

The approach taken is to use a technique quite similar to The Asphalt Institute component analysis method [Ref. 6A-1]. This method was further modified by Ritchie and Mahoney [Ref. 6A-2] and used in the computer program OVERDRIVE. The Asphalt Institute component analysis overlay design method and the modifications thereto will be presented in the following sections.

6.6.3 The Asphalt Institute Component Analysis Method

The component analysis approach to overlay design essentially requires that the total pavement structure be developed as a new design for the specified service conditions and then compared to the existing pavement structure (taking into account both pavement condition, type, and thickness of the pavement layers). A review of all current component design procedures quickly reveals that substantial judgment is required to effectively use them. This judgment is mainly associated with selection of "weighting factors" or "conversion factors" to use in evaluating the structural adequacy of the existing pavement layers.

The Asphalt Institute component analysis design approach (termed "effective thickness") uses relationships between subgrade strength, pavement structure, and traffic. The existing structural integrity of the pavement is converted to an equivalent thickness of asphalt concrete, which is then compared to that required for a new design. The structural evaluation procedure developed by The Asphalt Institute allows for determining the required thickness of asphalt concrete overlay or to estimate the length of time until an overlay is required.

The three essential parts of this overlay design procedure will be briefly described and include

110. subgrade analysis,
111. pavement structure thickness analysis, and
112. traffic analysis.

Subgrade Analysis

Testing of the subgrade materials is encouraged even if original design records are available. Use of resilient modulus (M_r), soaked CBR, or R-value tests appear to be the easiest to use with this procedure. For actual design, the design stiffness of the subgrade must be characterized in terms of resilient modulus. Associated correlations for CBR and R-value are

M_r (psi) = 1500 (CBR) Eq. 1

M_r (psi) = 1155 + 555 (R-value) Eq. 2

If test data in terms of M_r, CBR, or R-value are not available, subgrades can be placed into one of three classes for design purposes as follows:

(a) **Poor soils**. Soft and plastic when wet, generally composed of silts or clays, typical properties: M_r - 4,500 psi, CBR = 3, R-value = 6.

(b) **Medium soils**. Include soils such as loams, silty sands, and sand-gravels, which contain moderate amounts of clay and silt. These soils can be expected to lose only a moderate amount of strength when wet. Typical properties: M_r = 12,000 psi, CBR = 8, R-value = 20.

(c) **Good soils**. These soils can be expected to retain a substantial amount of their strength when wet and include clean sands and sand-gravels. Typical properties: M_r = 25,000 psi, CBR = 17, R-value = 43.

Pavement Structure Thickness Analysis

The goal of this portion of the design method is to determine the "Effective Thickness (TE)" of the existing pavement structure. The Asphalt Institute (A.I.) has two approaches, which can be used; only one will be illustrated in this report. First, the significant pavement layers are identified and their condition determined. Second, "Conversion Factors" are selected for each layer (judgment by the designer is very important at this point). Third, the Effective thickness for each layer is determined by multiplying the actual layer thickness by the appropriate Conversion Factor. The Effective Thickness of the complete pavement structure is the sum of the individual Effective Thicknesses. Typical layer thickness Conversion Factors are shown in Table 6.23.

Table 6.23 Example of the A.I. Conversion Factors for Estimating Thickness of Existing Pavement Components to Effective Thickness [Ref. 6A-1]

Description of Layer Material		Conversion Factor[1]
1.	Native subgrade	0.0
2.	a. Improved subgrade — predominantly granular materials b. Lime modified subgrade of high PI soils	
3.	a. Granular subbase or base — CBR not less than 20 b. Cement modified subbases & bases constructed from low PI soils	0.1 - 0.3
4.	a. Cement or lime-fly bases with pattern cracking b. Emulsified or cutback asphalt surfaces and bases with extensive cracking, rutting, etc. c. PCC pavement broken into small pieces	0.3 - 0.5
5.	a. Asphalt concrete surface and base that exhibit extensive cracking	0.5 - 0.7
6.	a. Asphalt concrete — general uncracked b. PCC pavement — stable, undersealed and generally uncracked pavement	0.9 - 1.0
7.	Other categories of pavement layers listed in Ref. 7A-1	

Traffic Analysis

The Asphalt Institute pavement design procedures require the use of 18,000 lb. (80 kN) equivalent single axle loads (ESALs) to characterize the traffic loading input. A variety of techniques can be used to estimate ESALs, and the Asphalt Institute provides information, which is helpful; however, the ESAL related information that follows is based on WSDOT data sources. For example, in general terms, Table 6.24 shows "typical" ESALs per year for various WSDOT routes. If a ten-year overlay design life was required, then ESALs ranging from 250,000 (SR21) to 20,000,000 (SR5) would be used. Naturally, values such as those shown in Table 6.24 are too general to be of much value for specific project locations.

An alternative approach for estimating ESALs is to multiply ESAL factors by specific classifications of trucks and buses and sum the results. Such ESAL factors have been compiled by the FHWA (Table 6.25) and for WSDOT conditions (Table 6.26). This approach can accommodate a wide variety of truck information ranging from only an estimate of the percent of the Average Daily Traffic (ADT), which constitutes trucks to estimates of trucks broken into categories of single and multi-units as illustrated in Table 6.26.

Table 6.24 Approximate ESAL Estimates for Various WSDOT Routes

Highway Type and Route	Estimated 18,000 lb ESALs per Year

[1] Equivalent thickness of new asphalt concrete.

Interstate	
113. SR5 (North Seattle)	2,000,000
114. SR5 (Centralia)	2,000,000
115. SR90 (Moses Lake)	750,000
116. SR90 (Spokane)	1,000,000
US	
117. SR2 (Stevens Pass)	150,000
118. SR97 (Swauk Pass)	125,000
119. SR195 (Pullman South)	175,000
State	
120. SR17 (Leahy South)	50,000
121. SR20 (Washington Pass)	50,000
122. SR21 (Republic South)	25,000

The term "truck factor" or "ESAL factor" represents the average ESAL per truck (or axle). The truck factors shown in Table 6.26 suggest an overall ESAL per truck of about 1.2 (this represents both loaded and unloaded trucks). Thus, if a project were expected to have 1,000,000 "trucks" during the design period, the resulting ESALs would be about 1,200,000.

Specifically, the WSPMS "imports" truck classification data in terms of single units (two axles), combination units (tractor-semi-trailer), and tractor-multi-trailer units. The associated ESAL factors are 0.25 ESAL per single unit, 1.0 ESAL per combination unit (assumes four axles), and 1.75 ESALs per multi-trailer unit (assumes seven axles). Further, an annual growth rate of 1.6 % is used for all cases.

Table 6.25 Average Truck Factors Compiled from FHWA Data [Ref. 6A-1]

Vehicle Types	Truck Factors (ESALs/truck)				
	Rural Highways			Urban Highways	Combined
	Interstate	Other	All	All	All
1. Single-units					
(a) 2-axle, 4-tire	0.02	0.02	0.03	0.03	0.02
(b) 2-axle, 6-tire	0.19	0.21	0.20	0.26	0.21
(c) 3-axles or more	0.56	0.73	0.67	1.03	0.73
(d) All single-units	0.07	0.07	0.07	0.09	0.07
2. Tractor semi-trailers					
(a) 3-axle	0.51	0.47	0.48	0.47	0.48
(b) 4-axle	0.62	0.83	0.70	0.89	0.73
(c) 5-axles or more	0.94	0.98	0.95	1.00	0.95
(d) All multiple units	0.93	0.97	0.94	1.00	0.95
3. All trucks	0.49	0.31	0.42	0.30	0.40

Table 6.26 Summary of ESAL Factors [Ref. 6A-5]

Highway System	ESALs Factors				
	Single Units	Combination Units	Buses	Individual Axle	Over Truck*
Interstate	0.30	1.25	1.60	0.25	1.20
Non-Interstate Rural	0.50	1.50	1.60	0.25	1.40
Non-Interstate Urban	0.25	1.20	1.60	0.25	1.00

* Excludes buses

Example of the Unmodified Asphalt Institute Procedures

A two-lane highway has the following characteristics and resulting overlay requirement.

1. **Traffic**
   (a) Average Daily Traffic = 4,000
   (b) Percent trucks (total all units) = 10%
   (c) Traffic growth rate = 4%

2. **Existing pavement structure and condition**
   (a) Asphalt concrete = 0.35 ft (4.2 in.)
   (b) Crushed stone base = 0.80 ft (9.6 in.)
   (c) Subgrade strength value: CBR = 8 or M_r = 12,000 psi
   (d) Overall, the pavement structure is in poor condition with asphalt concrete exhibiting well-defined crack patterns.

3. **Determine overlay thickness for a 10-year period**
   (a) 18,000 lb. Equivalent single axle loads

(i) Number of trucks in the design lane per day = (4,000) (0.50) (0.10) = 200

(ii) ESALs/day = 200 (1.4 ESALs/truck) = 280/day

(iii) If annual ESAL growth rate = 1.6 % for 10 years, then

(b) Effective pavement thickness

Layer Thickness (in.)		Conversion Factor (Table 6A-1)		Effective Thickness (in.)
4.2	x	0.5	=	2.1
9.6	x	0.2	=	1.9
		Total T_e	=	4.0

(c) Required new "Full-Depth" asphalt concrete pavement thickness (T_n) = 8.0 in. (refer to Figure 6A-1).

(d) Thickness of AC overlay = T_n - T_e = 8.0 - 4.0 in. = 4.0 in.

6.6.4 WSDOT Overlay Scope Method

The following subsections will overview three of the needed parameters to estimate AC overlay thickness: conversion factors for existing pavements, subgrade modulus, and full-depth AC thickness.

Conversion Factors

There are several elements from The Asphalt Institute component analysis method that have been modified to better conform to WSDOT conditions and practice. The primary modification is the layer conversion factors used to convert existing layers into equivalent new asphalt concrete thicknesses. These "initial" conversion factors ("C") were made a function of surface distress (both extent and severity). These factors are shown in Table 6.27 (Asphalt Concrete Surface Courses and Asphalt

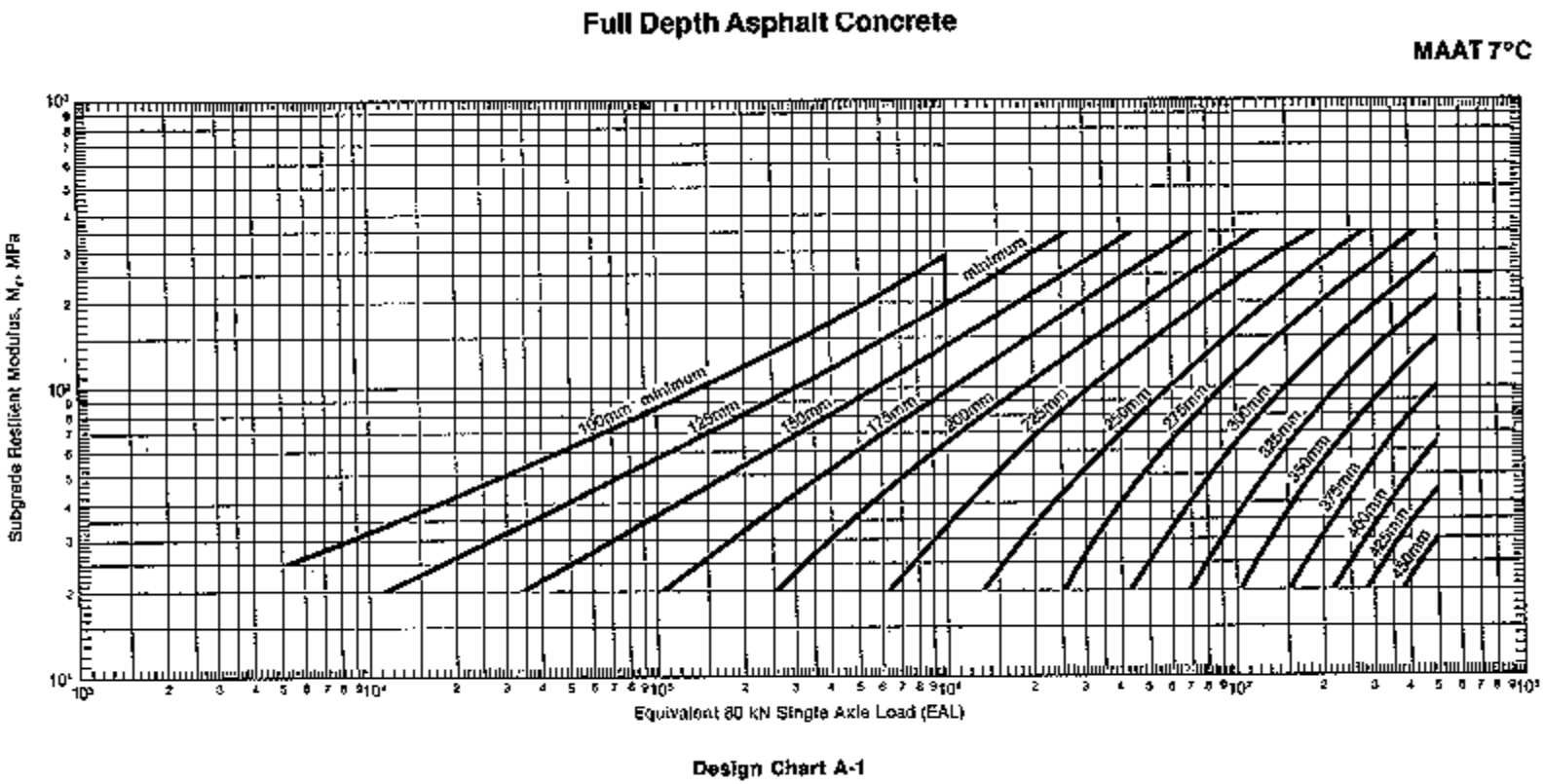

Figure 5A-1. The Asphalt Institute Design Chart for Full-Depth Asphalt Concrete Pavement (from Ref. 5A-1).

Figure 6.9 The Asphalt Institute Design Chart for Full-Depth Asphalt Concrete Pavement (from Ref. 6A-1)

Treated Bases (ATB)), Table 6.28 (Emulsified or Cutback Asphalt Surface Courses and Bases), and Table 6.29 (Base and Subbase Courses Other than ATB).

The initial implementation of the scoping technique resulted in numerous enhancements and refinements. A major enhancement was the development of the Pavement Structural Condition (PSC) value, which replaces the Pavement Condition Rating (PCR). The PSC can be used as a direct predictor of past, current, or future conversion factors ("C") for ACP or BST wearing courses. (Note: the details associated with the calculation of PSC are contained in the main body of the report). The relationship between the conversion factors and PSC are:

(a) ACP Wearing Course

"C" = 0.3 + [(PSC)(0.007)]

whereby if PSC = 0 then C = 0.3

PSC = 100 then C = 1.0

(b) BST Wearing Course

"C" = 0.3 + [(PSC)(0.005)]

whereby if PSC = 0 then C = 0.3

PSC = 100 then C = 0.8

These equations are simply linear functions scaled between the maximum and minimum values. For ACP or BST layers below the wearing course, the conversion factors fall into ranges as follows:

(a) "Buried" ACP Wearing Course: C = 0.65 (max) to 0.30 (min)

(b) "Buried" BST: C = 0.55 (max) to 0.30 (min)

The maximum "C" assumes a PSC = 50 when "buried" ("buried" implies being overlaid or resurfaced). The minimum "C" assumes a PSC = 0. The exception to this is that the "buried" ACP and BST "C" reverts to the wearing course "C" if the wearing course value is less than the maximum limits (ACP = 0.65, BST - 0.55) for the buried layers.

Table 6.27 Initially used Conversion Factors for Determining Equivalent Thickness of Asphalt Concrete for Asphalt Concrete Surface Courses and Asphalt Treated Bases

Type of Distress			Conversion Factor, C
Fatigue Cracking		Rutting	
Extent (%)[1]	Severity[2]		
0	—	0	1.0
0	—	< 1/4”	0.9
0	—	≥ 1/4”, < 1/2”	0.8
0	—	≥ 1/2”, < 3/4”	0.7
0	—	≥ 3/4”, < 1-1/2”	0.6
0	—	≥ 1-1/2”	Reconst or Mill and Overlay
< 10%	H	0, <1/2”	0.7
< 10%	S	0, <1/2”	0.5
≥ 10%, < 25%	H	0, <1/2”	0.6
≥ 10%, < 25%	S	0, <1/2”	0.4
≥ 25%	H	0, <1/2”	0.5
≥ 25%	S	0, <1/2”	0.3
< 10%	H or S	≥ 1/2”, < 3/4”	0.5
< 10%	H or S	≥ 3/4”, < 1-1/2”	0.4
< 10%	H or S	≥ 1-1/2”	Reconst or Mill and Overlay
≥ 10%, < 25%	H or S	≥ 1/2”, < 3/4”	0.4
≥ 10%, < 25%	H or S	≥ 3/4”, < 1-1/2”	Reconst or Mill and Overlay
≥ 10%, < 25%	H or S	≥ 1-1/2”	Reconst or Mill and Overlay
≥ 25%	H or S	≥ 1/2”, < 3/4”	0.3
≥ 25%	H or S	≥ 3/4”, < 1-1/2”	Reconst or Mill and Overlay
≥ 25%	H or S	≥ 1-1/2”	Reconst or Mill and Overlay

Table 6.28 Initially used Conversion Factors for Determining Equivalent Thickness of Asphalt Concrete for Emulsified or Cutback Asphalt Mixtures (Surfaces and Bases)[1]

Type of Distress			Conversion Factor, C
Fatigue Cracking		Rutting	
Extent (%)[2]	Severity[3]		
0	—	0, < 1/2”	0.8

1 Percentage of wheel track per station.
2 H = hairline; S = spalling
2 Percentage of wheel track per station.
3 H = hairline; S = spalling

0	—	≥ 1/2”, < 3/4”	0.6
0	—	≥ 3/4”, < 1-1/2”	0.5
0	—	≥ 1-1/2”	Reconst or Mill and Overlay
< 10	H	0, < 1/2”	0.6
< 10	S	0, < 1/2”	0.4
≥ 10, < 25	H	0, < 1/2”	0.5
≥ 10, < 25	S	0, < 1/2”	0.3
≥ 25	H	0, < 1/2”	0.4
≥ 25	S	0, < 1/2”	0.2
< 10	H	≥ 1/2”, < 3/4”	0.55
< 10	H	≥ 3/4”, < 1-1/2”	0.4
< 10	H	≥ 1-1/2”	Reconst or Mill and Overlay
< 10	S	≥ 1/2”, < 3/4”	0.3
< 10	S	≥ 3/4”, < 1-1/2”	0.25
< 10	S	≥ 1-1/2”	Reconst or Mill and Overlay
≥ 10, < 25	H	≥ 1/2”, < 3/4”	0.4
≥ 10, < 25	H	≥ 3/4”, < 1-1/2”	0.3
≥ 10, < 25	H	≥ 1-1/2”	Reconst or Mill and Overlay
≥ 10, < 25	S	≥ 1/2”, < 3/4”	0.3
≥ 10, < 25	S	≥ 3/4”, < 1-1/2”	0.2
≥ 10, < 25	S	≥ 1-1/2”	Reconst or Mill and Overlay
≥ 25	H	≥ 1/2”, < 3/4”	0.3
≥ 25	H	≥ 3/4”, < 1-1/2”	0.2
≥ 25	H	≥ 1-1/2”	Reconst or Mill and Overlay
≥ 25	S	≥ 1/2”, < 3/4”	0.25
≥ 25	S	≥ 3/4”, < 1-1/2”	Reconst or Mill and Overlay
≥ 25	S	≥ 1-1/2”	Reconst or Mill and Overlay

Table 6.29 Initially used Conversion Factors for Determining Equivalent Thickness of Asphalt Concrete for Base and Subbase Courses other than ATB

	Type of Material	Conversion Factor, C
•	Portland Cement Concrete (overlaid by ACP)	0.50
•	Cement Stabilized Granular Material	0.4
•	Cement stabilized, low PI soil	0.2
•	Lime or Lime-flyash stabilized crushed stone or gravel	0.3
•	Other lime stabilized layer excluding high PI subgrade soils	0.15
•	Crushed Stone – Crushed Surfacing Top Course – Crushed Surfacing Base	 0.35 0.30
•	Any other granular material with CBR >20	0.2
•	Sand	0.1

Subgrade Modulus

The subgrade modulus is required to obtain a thickness for a "new" full-depth asphalt concrete pavement (along with the design ESALs). There are at least three options for estimating this modulus value.

Deflection Data

The WSDOT FWD deflection data can be used along with the following equation to estimate the subgrade modulus (equation from a recent NCHRP study [Ref. 6A-6], which will be used to revise Part III of the AASHTO Pavement Guide [Ref. 6A-7]):

$$M_R = P\,(1-\mu^2) / (\pi)(D_r)(r) \qquad \text{Eq. 3}$$

Where

M_R = backcalculated subgrade resilient modulus (psi),
P = applied load (lbs.),
D_r = pavement surface deflection a distance r from the center of the load plate (inches),
μ = Poisson's ratio for the subgrade (usually fixed at 0.45), and
r = distance from center of load plate to D_r (inches).

An earlier version of the WSDOT scoping technique used the equations developed by Newcomb [Ref. 6A-3].

Subgrade Classes

The general subgrade classes can be used as shown in Table 6.30 (also described in Paragraph 6.6.3). In order words, one can select the basic subgrade class for each specific pavement segment. In lieu of this, the single best category to choose in Table 6.30 is 12,500 psi. This recommendation is based n review of extensive amounts of backcalculated layer moduli throughout the state and WSDOT triaxial laboratory tests. Actually, use of 12,500 psi is somewhat conservative since 12 subgrade samples tested by WSDOT averaged 19,300 psi with a standard deviation of 8,600 psi. Thus, 12,500 psi is about the average of the samples tested minus one standard deviation. This range of subgrade moduli undoubtedly appears somewhat "high." Most of the WSDOT triaxial testing to date on "subgrades" is borrow materials used to create the roadway embankment.

Table 6.30 Subgrade classes [Ref. 6A-1]

Soil Class	Characteristics	Design Modulus
Poor	Soft and plastic when wet, generally composed of silts and clays	4,500 psi
Medium	Includes loams, silty sands, and sand-gravels, which contain moderate amounts of clay and silt. Can be expected to lose only a moderate amount of strength when wet.	12,500 psi
Good	Expected to retain substantial amount of strength when wet. Includes clean sands and sand-gravels	25,000 psi

Performance Based

If one assumes that the PSC - Age relationship for each pavement segment is primarily influenced by subgrade modulus, then it is possible to develop a "rule of thumb" from this relationship to estimate subgrade modulus. One must caution that the following can at best be only approximately correct and at worst very wrong.

An examination of Figure 6.9 shows that for various levels of design ESALs, halving or doubling the subgrade modulus from some nomial value increases or decreases the required AC depth by about 1.5 inches.

For example:

Subgrade Modulus (psi)	AC Thickness		
	Design ESALs		
	100,000	*1,000,000*	*10,000,000*
6.250	6.3"	9.3"	14.3"
12.500 (nominal)	4.9"	7.9"	12.9"
25,000	--	6.1"	10.9"

Further, if one assumes a nominal subgrade modulus of 12,500 psi for a design ESALs of 1,000,000, then the range of ESALs is 2,400,000 (+ 1.5 inch AC) to 350,000 (-1.5 inch AC). Dividing the range of subgrade moduli (25,000 - 6,250 psi) by the associated ESAL range (2,400,000 - 350,000) results in 9.1 psi per 1,000 ESALs (or round off to 10 psi per 1,000 ESAL).

The way the above rule of thumb can be used is illustrated as follows:

If normal Age to PSC = 50 is, say, 12.5 years (from standard (default) equation), but actual Age at PSC = 50 is 10 years, then ΔAge= 2.5 years (12.5 - 10). Multiply ΔAge x ESALs/year = ESALs. This provides an estimate of the "reduced" ESALs. If ESALs/year = 100,000, then

ΔAge x ESALs/yr = 2.5 yr x 100,000/yr = 250,000 ESALs (or 250 1,000 ESALs).
Thus, estimated subgrade modulus
= nominal modulus - (250 1,000 ESALs). = 12,500 - 2,500 = 10,000 psi

Thus, M_r = 10,000 psi for the purpose of obtaining a full-depth, new AC thickness from Figure 6.9.

The above calculations, again, assume any loss in performance is associated with a lower than nominal subgrade modulus. This assumption is, of course, overly simplistic; however, some attempt must be made to account for less than nominal pavement performance. Further, this method of estimating subgrade resilient modulus need only be considered until such time as all WSDOT pavements have representative FWD deflection data and, hence improved estimates of subgrade modulus.

Calculation of Full-Depth Asphalt Concrete Pavement

O'Neil [Ref. 6A-4] developed the following regression equation which can be used to estimate full-depth AC as a function of subgrade modulus and ESALs:

$$\text{AC Thickness (in)} = -3.845 + 5.672\,(\ln(M_r)) - 0.4390\,(\ln(M_r))^2 - 2.197\,(\ln(\text{ESAL})) + 0.1455\,(\ln(\text{ESAL}))^2 \qquad \text{Eq. 4}$$

$R^2 = 0.996$
RMSE = 0.299 inch
n = 67 points taken from Figure 6A-1.

For example, from Figure 6A-1, for ESALs = 1,000,000 and M_r = 12,500 psi, the full-depth AC thickness is about 7.9 inches. Inserting these ESALs and M_r into Equation 4:

$$\text{AC (in)} = -3.845 + 5.672\,(\ln(12{,}500)) - 0.4390\,(\ln(12{,}500))^2 - 2.197\,(\ln(1{,}000{,}000)) + 0.1455\,(\ln(1{,}000{,}000))^2 = 8.0 \text{ inches}$$

A 0.1-inch difference is only 0.0083 ft of AC overlay, an acceptable variation.

Illustration of Method

For a pavement segment that requires an overlay,

125. Design ESALs = 1,000,000 (assumed)
126. Subgrade resilient modulus = 12,500 psi (known or calculated)
127. Full-depth new AC = 8.0 inches (calculated from Equation 4)
128. Existing pavement structure
    - 4.2 inches AC (PSC = 60; thus "C" = 0.72)
    - 9.6 inches crushed stone base (CSTC/CSB)
    - Subgrade (nominal)
132. Convert existing to full-depth AC
    - AC: 4.2 inches x 0.72 = 3.0 inches
    - Base: 9.6 inches x 0.30 (Table 7A-6) = 2.9 inches

    5.9 inches
135. Scoped" overlay thickness

    8.0 inches - 5.9 inches = 2.1 inches (or about 0.18 ft)

Discussion

To initially evaluate the overlay scoping technique, three separate pavement sections were evaluated at different ESAL subgrade moduli, and distress levels. The ESAL levels were set at 1×10^5, 1×10^6, and 1×10^7; the subgrade moduli at 4,500 psi, 12,500 psi, and 25,000 psi; the distress levels were a function of fatigue cracking only (none, less than 10 % hairline cracking, greater than 10 % but less than 25 % hairline cracking, and greater than 25 % spalling cracks). The distress levels were used along with Tables 6.27 and 6.29 (conversion spalling cracks) to calculate the "existing" full-depth AC thickness (t_x). The new AC full-depth thickness (t_n) was calculated by use of Equation 6 (recall that this equation was developed from Figure 6.9). The difference between (t_n) and (t_x) is the required overlay thickness (t_o). These results are shown in Tables 6.31, 6.32, and 6.33.

The overlay thickness appears to be somewhat typical. However, one interesting observation is the influence of extensive pavement distress (as illustrated by the ≥ 25 % fatigue cracking) on the required overlay. The overlay thickness is nearly independent of the original pavement structure if extensive distress is allowed to accumulate (based on t_o values in Tables 6.31, 6.32, and 6.33). This tends to reconfirm what the authors have previously observed on a few, prior, relatively thick overlay designs for the WSDOT route system.

Table 6.31 Overlay Thickness for Various Levels of ESAL's Pavement Thickness and Distress Levels (Subgrade M_r = 4,500 psi)

Existing Pavement Structure	Distress Level[1]	ESALs								
		100,000			1,000,000			10,000,000		
		Calculated Thickness			Calculated Thickness			Calculated Thickness		
		t_x (2) (in.)	t_n (3) (in.)	t_o (4) (in.)	t_x (2) (in.)	t_n (3) (in.)	t_o (4) (in.)	t_x (2) (in.)	t_x (2) (in.)	t_o (4) (in.)
1.5" AC	None	5.1	6.8	1.7	5.1	10.2	5.1	5.1	15.2	10.1
	< 10% H	4.6	6.8	2.2	4.6	10.2	5.6	4.6	15.2	10.6
12.0" BASE	≥ 10 < 25% H	4.5	6.8	2.3	4.5	10.2	5.7	4.5	15.2	10.7
(CSTC)	≥ 25% S	4.0	6.8	2.8	4.0	10.2	6.2	4.0	15.2	11.2
3.0 " AC	None	5.4	6.8	1.4	5.4	10.2	4.8	5.4	15.2	9.8
	< 10% H	4.5	6.8	2.3	4.5	10.2	5.7	4.5	15.2	10.7
8.0" BASE	≥ 10 < 25% H	4.2	6.8	2.6	4.2	10.2	6.0	4.2	15.2	11.0
(CSTC)	≥ 25% S	3.3	6.8	3.5	3.3	10.2	6.9	3.3	15.2	11.9
6.0" AC	NONE	8.4	6.8	0.0	8.4	10.2	1.8	8.4	15.2	6.8
	< 10% H	6.6	6.8	0.2	6.6	10.2	3.6	6.6	15.2	8.6
8.0" BASE	≥ 10 < 25% H	6.0	6.8	0.8	6.0	10.2	4.2	6.0	15.2	9.2
(CSTC)	≥ 25% S	4.2	6.8	2.6	4.2	10.2	6.0	4.2	15.2	11.0

[1] Distress levels for fatigue cracking only
(2) t_x: Equivalent thickness of new AC based on "C" factors from Tables 7A-4 and 7A-6.
(3) t_n: From Equation 5 (fixed M_r = 4,500 psi)
(4) $t_o = t_n - t_x$

Table 6.32 Overlay Thickness for Various Levels of ESALs Pavement Thicknesses and Distress Levels (Subgrade M_r = 12,500 psi)

Existing Pavement Structure	Distress Level[1]	ESALs								
		100,000			1,000,000			10,000,000		
		Calculated Thickness			Calculated Thickness			Calculated Thickness		
		t_x[(2)] (in.)	t_n[(3)] (in.)	t_o[(4)] (in.)	t_x[(2)] (in.)	t_n[(3)] (in.)	t_o[(4)] (in.)	t_x[(2)] (in.)	t_x[(2)] (in.)	t_o[(4)] (in.)
1.5" AC	None	5.1	4.6	0.0	5.1	8.0	2.9	5.1	13.0	7.9
	< 10% H	4.6	4.6	0.0	4.6	8.0	3.4	4.6	13.0	8.4
12.0" BASE	≥ 10 < 25% H	4.5	4.6	0.1	4.5	8.0	3.5	4.5	13.0	8.5
(CSTC)	≥ 25% S	4.0	4.6	0.6	4.0	8.0	4.0	4.0	13.0	9.0
3.0 " AC	None	5.4	4.6	0.0	5.4	8.0	2.6	5.4	13.0	7.6
	< 10% H	4.5	4.6	0.1	4.5	8.0	3.5	4.5	13.0	8.5
8.0" BASE	≥ 10 < 25% H	4.2	4.6	0.4	4.2	8.0	3.8	4.2	13.0	8.8
(CSTC)	≥ 25% S	3.3	4.6	1.3	3.3	8.0	4.7	3.3	13.0	9.7
6.0" AC	NONE	8.4	4.6	0.0	8.4	8.0	0.0	8.4	13.0	4.6
	< 10% H	6.6	4.6	0.0	6.6	8.0	1.4	6.6	13.0	6.4
8.0" BASE	≥ 10 < 25% H	6.0	4.6	0.0	6.0	8.0	2.0	6.0	13.0	7.0
(CSTC)	≥ 25% S	4.2	4.6	0.4	4.2	8.0	3.8	4.2	13.0	8.8

[1] Distress levels for fatigue cracking only

(2) t_x: Equivalent thickness of new AC based on "C" factors from Tables 7A-4 and 7A-6.

(3) t_n: From Equation 5 (fixed M_r = 4,500 psi)

(4) $t_o = t_n - t_x$

Table 6.33 Overlay Thickness for Various Levels of ESALs, Pavement Thicknesses and Distress Levels
(Subgrade* M_r 25,000 psi*)

Existing Pavement Structure	Distress Level[1]	ESALs								
		100,000			1,000,000			10,000,000		
		Calculated Thickness			Calculated Thickness			Calculated Thickness		
		t_x(2) (in.)	t_n(3) (in.)	t_o(4) (in.)	t_x(2) (in.)	t_n(3) (in.)	t_o(4) (in.)	t_x(2) (in.)	t_x(2) (in.)	t_o(4) (in.)
1.5" AC	None	5.1	4.0(5)	0.0	5.1	6.0	0.9	5.1	11.0	5.9
	< 10% H	4.6	4.0	0.0	4.6	6.0	1.4	4.6	11.0	6.4
12.0" BASE	≥ 10 < 25% H	4.5	4.0	0.0	4.5	6.0	1.5	4.5	11,.0	6.5
(CSTC)	≥ 25% S	4.0	4.0	0.0	4.0	6.0	2.0	4.0	11.0	7.0
3.0 " AC	None	5.4	4.0	0.0	5.4	6.0	0.6	5.4	11.0	5.6
	< 10% H	4.5	4.0	0.0	4.5	6.0	1.5	4.5	11.0	6.5
8.0" BASE	≥ 10 < 25% H	4.2	4.0	0.0	4.2	6.0	1.8	4.2	11.0	6.8
(CSTC)	≥ 25% S	3.3	4.0	0.7	3.3	6.0	2.7	3.3	11.0	7.7
6.0" AC	NONE	8.4	4.0	0.0	8.4	6.0	0.0	8.4	11.0	2.6
	< 10% H	6.6	4.0	0.0	6.6	6.0	0.0	6.6	11.0	4.4
8.0" BASE	≥ 10 < 25% H	4.0	4.6	0.0	6.0	6.0	0.0	6.0	11.0	5.0
(CSTC)	≥ 25% S	4.2	4.0	0.0	4.2	6.0	1.8	4.2	11.0	6.8

[1] Distress levels for fatigue cracking only
(2) t_x: Equivalent thickness of new AC based on "C" factors from Tables 6A-4 and 7A-6.
(3) t_n: From Equation 6 (fixed M_r = 4,500 psi)
(4) $t_o = t_n - t_x$
(5) The Asphalt Institute recommends 4.0 inches of AC as a minimum (Equation 6 indicates 2.6 Inches)

6.6.5 Future Enhancements

The scoping technique does not have any type of reliability to account for project uncertainties (and hence uncertainly relative to pavement performance). The addition of reliability can be added to the technique if desired.

6.6.6 Summary

The previously described overlay scoping method can only be considered as approximate; however, prior experience with OVERDRIVE (and limited comparisons to other design methods) has shown the resulting overlay thickness to be reasonable. Only experience with the proposed approach will prove its value or show that further modifications are required.

6.6.7 References to Attachment 6.6

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6A-2. Ritchie, S.G. And J.P. Mahoney, "Development and Performance of an Expert System for Pavement Overlay Design," Symposium on Knowledge-Based Systems in Civil Engineering, Monash University, Australia, August 1988.

6A-3. Newcomb, D.E., "Development and Evaluation of a Regression Method to Interpret Dynamic Pavement Deflections," Ph.D. Dissertation, University of Washington, Seattle, Washington, 1986.

6A-4. O'Neil, D., "Regression Analysis for the Full-depth Asphalt Pavement Nomograph," Paper submitted for CETS 599, Special Topics, University of Washington, Seattle, Washington, March 2, 1992

6A-5. WSDOT, WSDOT Pavement Guide, Draft Document, Washington State Department of Transportation, Olympia, Washington, May 1992

6A-6. Darter, M.I., Elliot R.P. , and Hall, K.T., "Revision of AASHTO Pavement Overlay Design Procedures." Draft Final Report, Project 20-7/39, National Cooperative Highway research Program, Transportation Research Board, Washington, D.C., June 1991.

6A-7. AASHTO, AASHTO Guide for Design of Pavement Structures, American Association of State Highway and transportation Officials, Washington, D.C., 1986

7 PAVEMENT MATERIALS AND CONSTRUCTION PERFORMANCE

7.1 INTRODUCTION

The main objective of this chapter is to review the relationship of materials, construction details, and specifications, to PMS. It will be demonstrated how historical PMS pavement structure files can be used with performance data to evaluate construction specifications or materials. Illustrations of a pavement history file will be given also.

After a general overview of materials and construction methods, examples of engineering applications will be given from Arizona, Texas, and Kansas. Since Kansas DOT is featuring for the first time in this Reference Manual, a brief description of their PMS will precede their example.

7.2 MATERIALS AND CONSTRUCTION OVERVIEW

7.2.1 Introduction to Materials

The physical inputs to the pavement management process include material properties, traffic characteristics, and environmental factors. The purpose of this section is to describe, in a general manner, the nature of these inputs, particularly materials, with respect to the pavement performance and management process. The focus with respect to material characterization is on the fundamental evaluation of the material properties rather than on empirical testing [Haas 94].

A pavement management system can provide information required to evaluate materials performance and its relationship to pavement performance. Details regarding materials characteristics, type, and source can be stored in the structural history database of the PMS. As pavement performance is tracked over time these factors can be analyzed as to their effects on deterioration rates, load carrying capacity, durability, serviceability, and overall performance of pavements under various levels of traffic and environment.

Performance of new materials can also be tracked and evaluated if they are properly recorded in the PMS database. The effects of using crumb rubber asphalts, superfine mixes, stone matrix asphalts, and other new technologies can be identified with a suitable engineering analysis. Such materials must be set up as specific material types within the PMS and each project constructed with the material must be tracked in the normal PMS processes. Performance modeling techniques may then be used to evaluate and compare performance of projects constructed with the new materials against projects constructed with conventional materials under similar conditions.

7.2.2 Material Properties

The primary materials used for pavement construction are the asphalt concrete or portland cement concrete surface, granular bases and subbases, asphalt, cement, and lime stabilized bases, and steel used for reinforcement and dowel bars in concrete pavements. In addition, due to the importance of the subgrade in the performance of pavements any discussion of pavement materials must also address subgrade soil properties.

Fundamental Material Characterization

Many pavement design methods use empirical tests of material quality as opposed to methods that quantify the engineering properties of the materials. Examples of empirical tests include the California Bearing Ratio (CBR) and the Marshall stability test. Empirical material characterization has many drawbacks, the primary one being the inability to extrapolate historical knowledge to changing conditions. Since pavement design is a dynamic process with new materials being introduced and traffic loads changing, there is increasing interest in the use of mechanistic analysis procedures. Mechanistic evaluation requires testing procedures that determine the load-response properties of the materials, usually under test conditions that simulate field conditions.

Mechanistic models of a structural response are derived based on assumptions about the response of the material to loads. The terminology used to describe the load-response characteristics of materials are defined in many textbooks of the subject of strength of materials and will not be reviewed here. Materials can display a wide range of response to load ranging from elastic to plastic and viscous; the properties can be dependent on the rate of loading and temperature during testing. The simplest type of behavior is where the deformation of the material is proportional to the stress level and the material returns to its original shape when the load is removed. This proportional response would be independent of other factors that can affect material response such as the rate of loading, the temperature of the test, and the age of the material. While this type of behavior is commonly assumed for model simplification, it rarely occurs in nature. Thus, the mechanistic evaluation of materials always requires compromises between the complexity of the testing and analytical procedures and the ability of engineers to perform the tests and the calculations in a cost effective manner; hence, the need to perform tests under simulated field conditions. However, in-service conditions can never be fully duplicated in the lab [Sentler 71]:

> The strength characteristic of materials is to a large extent based on the results obtained in standardized tests. Such information is valuable because it is often the only information available. But very few structural members, if any, fail in a way, which resembles a standardized test. Instead, other types of failures like fatigue play a much more important role in practice. It is also obvious that the environmental influence has to be considered in a more appropriate manner.

Deacon, in the conclusion to an in-depth literature survey of pavement material properties states [Deacon 71]:

> One may characterize the behavior of pavement materials in numerous ways depending in part on the nature of the problem and in part on personal preferences. It must be emphasized, however, that in most cases pavement materials do not possess idealized properties and that the measured properties are often significantly influenced by the test procedures and equipment. It is important, therefore, for laboratory procedures to simulate to as great a degree as possible actual field loading conditions. Test procedures that result in nearly homogeneous stress and strain states are necessary to investigate the properties of a small volume element.

As the characteristics of materials are reviewed in the following sections, it is important to realize that these discussions are based on the state-of-the-art methods of analysis for pavement design and do not necessarily represent the "true" response of the material that would be observed by a materials specialist. It is also important to recognize that the state-of-the-art in pavement design is limited by the development of modeling assumptions rather than representing the ultimate development in pavement analysis techniques. In other words, application of current mechanistic testing and analysis techniques represents an improvement over empirical methods but there remains a need to further refine our analytical capability.

Steel

Steel is used for pavement reinforcement and load transfer devices. Compared to other materials used in pavements, the properties of steel are relatively easy to quantify. For the conditions encountered in pavement performance, steel is a linear elastic material. Structural design of the steel in pavements is relatively straightforward as Hooke's law applies and the modulus of elasticity and Poison's ratio are well defined.

Generally, the corrosion is the limiting factor in the performance of the steel used for reinforcing and load transfer devices in pavements. Epoxy coatings can be used to limit the corrosion of reinforcing steel. Load transfer devices can either be epoxy coated or made with stainless steel.

Portland Cement Concrete

Portland cement concrete is a combination of cement aggregates and water. The chemical reaction between the cement and water forms the bonds in the concrete that determine the properties of the hardened material.

The failure mode of concrete depends on the stress state of the material. Uniaxial compressive tests are generally used in concrete design and quality control tests to determine the ultimate compressive strength. However, concrete is much weaker in tension than in compression. Since pavements carry traffic loads in a flexural mode, the tensile-flexural test is usually recommended for the design and quality control of concrete used in pavements.

In addition to mechanical failures, many concrete pavements fail due to deterioration of the concrete as a result of environmental and chemical attach. Freezing water in the pores and cavities of the concrete expands about nine percent, producing hydrostatic pressure [Derucher, 88]. Generally an air-entraining admixture is added to the concrete to provide protection against freeze-thaw deterioration. Deicing salts and chemicals can increase the water retention of the concrete and contribute to recrystallization and weathering. These effects will cause the properties of the concrete to vary with time.

The stress-strain behavior of concrete is nonlinear. Generally the modulus of elasticity that is used for design is determined as a secant modulus with the end points of the cord being the origin and the point on the curve where the stress equals one-half of the compressive strength.

Asphalt Concrete

Asphalt concrete mix design consists of selecting the aggregate gradation and asphalt binder type and content required to meet design criteria, such as the Marshall or Hveem stability and flow, and void content. In the past, little attention has been paid to the relationship between the asphalt concrete quality and the thickness of the pavement. Based on the Mashall or Hveem mix design method, the asphalt concrete was either acceptable or not. A new asphalt concrete mix design procedure was proposed based on evaluation of asphalt mixtures using engineering measures of the properties [Monismith 85]. One of the unique features of this method is simultaneous consideration of the mix design and pavement design. In other words, greater consideration of the application of the material needs to be included in the mix design process. The mixture properties that should be considered during mix design include:

1. Mixture stiffness
2. Resistance to permanent deformation
3. Durability
4. Fatigue resistance
5. Low temperature response (including stiffness at long loading times and fracture characteristics)
6. Permeability

With the exception of durability and permeability, it is recommended that the properties of the mixes are measured in a form, which permits mechanistic analysis [Monismith 85]. This mechanistic approach makes it possible to improve the predictive capability of the design models, particularly with respect to changes in materials and traffic loading conditions. Recently this approach has led to the development of Superpave asphalt mixes [SHRP 94a and 94b].

Stabilized Materials

Granular materials can be stabilized with various chemicals such as portland cement, asphalt cement, lime, fly ash, sodium chloride, and others. Generally the amount of cement used for stabilization and the specifications for the aggregate gradation are lower for the production of the stabilized materials than for concrete. Therefore, the quality of the stabilized materials will not be equal to asphalt concrete or portland cement concrete. However, the nature of the load response curves is similar, for example, cement-stabilized bases are nonlinear elastic and asphalt-stabilized bases are viscoelastic nonlinear.

Soils and Granular Materials

The properties of the granular materials depend on gradation, moisture content, density, stress state, and the aggregate shape and texture. Depending on the amount and type of fine material in the aggregates, they may be classified as either cohesive or cohesionless. From a general mechanistic response viewpoint, soils and aggregate materials can be considered within these two classifications. However, the properties of unstabilized aggregate materials are discussed separately due to the extensive use of these materials in bases and subbases.

The characteristics of aggregates and soils are strongly influenced by the moisture content. During construction, care is taken to compact these materials at the optimum moisture content. However, during the life of the pavement various mechanisms such as percolation of water through cracks in the pavement surface and capillary action tend to allow water to enter the base and the subgrade. As a result, many pavement foundations, even in arid regions, are saturated for a major portion of the time. In essence, this means the strength of the foundation in field conditions is less than the optimum strength.

Cedergren developed a damage factor for comparing the damage to pavements with saturated bases and subgrades to well drained pavements. This damage factor ranged from 10 to 70,000 demonstrating the importance of subgrade moisture on the strength of the pavement. Cedergren also demonstrated that pavements can remain saturated for up to 22 days following a rain storm [Cedergren 88].

Cohesive Soils

Most investigators describe the behavior of cohesive soils as highly nonlinear [Deacon 71]. Clays show immediate and time dependent recoverable and permanent strains, the immediate strains being predominant under short duration loads and the permanent strain per cycle decreasing to an insignificant amount after many cycles of stress. Stress history may have a significant effect on response. The nonlinear response to load varies in two ways:

1. The stiffness of these materials is dependent on the initial stress state and increases as the effective mean principal stress increases.
2. The stiffness decreases with an increase in the incremental stress amplitude (deviator stress in triaxial tests).

The effects of load, mixture features and environment on the stiffness of cohesive soils are summarized in Table 7.1 [Deacon, 71]. As this table demonstrates, the load deformation characteristics, or stiffness, of cohesive soils are very complex. In addition, cohesive soils are cross-isotropic with the horizontal stiffness exceeding the vertical stiffness.

Cohesionless Soils

The stiffness of cohesionless soils, such as sand, are affected by many of the same factors as cohesive soils. However, the response to an increase in the number of cycles differs for cohesive and cohesionless soils. Table 7.2 is a summary of how the various factors affect the stiffness of cohesionless soils [Deacon 71]. Many investigators relate the stiffness, *S*, of cohesionless soils to the mean effective stress, σ_0 as

$$S = K s_0^n \qquad \text{Eq. 7.1}$$

where

K and *n* are experimentally determined constants. Cohesionless soils by definition do not have tensile strength. However, they are probably more isotropic than other paving materials [Deacon 71].

Untreated Granular Materials

The effect of various factors on the stiffness of granular materials is summarized in Table 7.3 [Deacon 71]. The major effect is the initial confining pressure on the sample. The modulus of resilient deformation, M_R, to the initial stress state is

$$M_R = K s_3^n \qquad \text{Eq. 7.2}$$

and

$$M_R = K s_0^n \qquad \text{Eq. 7.3}$$

These equations are similar to Equation 7.1 for the cohesionless soil, except σ_3 is the initial confining pressure in a triaxial test.

Table 7.1 Effect of Variables on the Stiffness of Cohesive Soils [Deacon 71]

Variable that is increased	**Effect on Stiffness**	*Remarks*
Loading		
Number of cycles	Decrease	
	Minimum	Minimum at 1 to 5000 cycles
Incremental strain amplitude	Decrease	Rate of decrease depends on maximum stiffness and shear stress
Effective mean initial principal stress	Increase	Effect depends on stress or strain amplitude
Transverse stress		No effect
Initial octahedral shear stress		Effect negligible after 10 cycles
Frequency of loading Increase	Increase	Effect minor above 10 cps
Strain rate	Increase	
Over consolidation ration	Increase	Any effect can be explained on basis of effective pressure and void ratio
Stress path		Large dependency
Mixture		
Soil disturbance	Decrease	
Void ratio	Decrease	Maximum effect at low confining pressure At small strains
Dispersion Structure	Decrease	Little effect on max. shear modulus Modulus of resilient deformation
Degree of saturation at compaction	Decrease	
Plasticity	Decrease	Impact compaction
Compaction Energy	Maximum	
Environmental		
Aging	Increase	
Degree of saturation	Decrease	
Time (thixotropy)	Increase	Recovery after high amplitude cyclic loading or many load cycles
Densification	Increase	
Time (during secondary compression)	Increase	Bentonite

Table 7.2 Effect of Variables on the Stiffness of Cohesionless Soils [Deacon 71]

Variable that is Increased	Effect on Stiffness	Remarks
Loading		
Number of cycles	Increase	Approaches a maximum
Incremental strain amplitude	Decrease	Rapid decrease
Incremental stress amplitude	Decrease	
Load duration	Decrease	Pulsating loads
Load rate or frequency	Constant	No effect after the first few cycles
Initial effective mean principle stress	Increase	
Initial octahedral shear stress	Decrease	Very small effect after 10 load cycles
Mixture		
Void ratio	Decrease	
Environmental		
Degree of saturation	Constant	Effective stresses must be used

Table 7.3 Effect of Variables on the Stiffness of Untreated Granular Aggregates [Deacon 71]

Variable that is Increased	Effect on Stiffness	Remarks
Loading		
Number of cycles	Constant	After 50 to 100 cycles
Initial confining pressure	Increase	Triaxial compression
Initial effective mean principal stress	Increase	
Incremental stress amplitude	Constant to increase	Differences in literature, large effect if shear failure
Load duration	Constant	0.1 to 0.25 s
Load rate or frequency	Increase	small increase
Drainage	Constant	
Mixture		
Void ratio	Decrease	At low moisture content
	Increase	At high moisture content
Angularity and surface roughness	Increase	
Fines	Decrease	Minor effect
Compaction water content	Decrease	
Environmental		
Degree of saturation	Constant	Effective stresses must be used

7.2.3 Introduction to Construction

A complete pavement management system must follow through from design of new pavements or rehabilitation to the implementation phases of construction and maintenance, plus data feedback. The transition from design to construction is one of the most important and difficult organizational boundaries in the pavement management system. [Haas 94]

Construction converts a design recommendation into physical reality. Successful construction meets the planning and design objectives, within budget and time constraints. Contract tendering, contract award, construction

schedule, materials supply and processing, and actual construction and quality control are usually conducted in a relatively routine manner. It is not sufficient, however, that design should function with little thought to construction or maintenance. Yet this is the case in many agencies, especially larger ones, where the traditional organization involves separate divisions for design and construction.

If conditions in the field are found to differ from the design inputs and assumptions, changes must be made. Such changes, at least major ones, should normally be reviewed and approved by the design group. Proper pavement management will ensure that sufficient communication occurs, that new or innovative design or construction solutions are not inhibited, and that as-constructed documentation is complete and understandable. This latter point is very important. Because of the usual hurry and day-to-day problems of construction, it is easy to delay or avoid such documentation of the pavement as it was actually built. There have been many experiences where search through the files at some time after construction yield incomplete or erroneous information. Proper feedback and plans for rehabilitation depend on reliable and complete as-constructed documentation.

This does not attempt to provide details on construction practices, construction control, or construction management per se. These are comprehensive subjects that are treated in a variety of construction manuals, operating procedures, and guides published by various agencies. Attention in this module is focused on: 1) the interrelationships of construction with other phases of pavement management and 2), the documentation data that construction should produce for both construction and other uses. If these functions are carried out in a systematic manner, then the normal and expected variations in construction methods, equipment, and materials, can be properly taken into account.

This module also summarizes the various levels of construction management and their relationship to pavement management. Finally, this module considers information acquired in construction and construction control as it relates to the pavement management system.

Construction Documents

In order to move the selected design to the construction phase, a set of definitive documents expressing the details of the selected design, or of the alternative offered for contract tenders, is needed. These documents not only convey details to construction but also serve as legal documents in procuring the services of a contractor or construction agency.

The traditional documents of design and construction are plans and specifications. These usually consist of the following:

1. A set of drawings that give detailed dimensions and other design aspects
2. A set of specifications that describe in detail the materials to be provided, their arrangement, required characteristics, etc.
3. A set of "standards and specifications" that have been previously approved by the agency and are in general use

The first purpose of these implementation documents is to describe the proposed pavement to the construction group. Because the pavement is usually built in conjunction with bridges, drainage facilities, and other items, the interrelationships involved are also described. In some agencies the construction group is equipped to construct the pavement as designed, and in such cases construction can immediately begin. Usually, however, the construction group has a supervisory or control function and the actual construction are independently done under a legal contract. In these cases, the documents become highly important because they serve a second purpose as the basis for bidding, pricing, and agreements regarding payment for the work.

Construction Management

Construction management involves the use of physical, financial, and personnel resources to convert designs to physical realities. This general concept intersects pavement management in the actual building of the pavement

structure. The processes are successful when the stated planning and design objectives are met and the pavement is put into service.

The process of construction management is comprehensive and complex. It involves many considerations such as estimation, designation and scheduling of activities, organizational and personnel aspects, legal aspects, finances and cost control, records, and quality control. There are many published books and manuals available on the subject. Although it is beyond the scope of this course to treat this important subject in depth, the levels of pavement construction management as they apply to public agencies are subsequently discussed.

A major problem in construction over the years has been payment for the final product. Historically, the approach has been one of full payment for a pavement constructed according to (procedurally based) specifications and judged to be acceptable. Some agencies have now moved to what they term end product specifications, which cover as-built density, gradation, voids, thickness and others. Also, some agencies reduce full payment through penalty clauses if the specifications are not met, and a few have bonus clauses if the specifications are exceeded.

It should be possible however to eventually develop true end product specifications based on long term pavement performance [Kuzy 91, Vlatas 89]. The measures used could include as-built roughness, deflection and surface friction and the specifications would place limits on these measures (i.e., the rate of deterioration) plus surface distress over the life cycle of the pavement. Such an approach would transfer most of the risk to the contractor, but also provide for a more rational payment scheme, and a greater incentive for innovation.

7.2.4 Construction as Related to Other Phases of Pavement Management

Effective pavement management depends on communication and coordination among all phases. Figure 7.1 illustrates the general information flows to construction from the other phases of a pavement management system. Details of these phases have been discussed previously. The diagram shows that construction receives vital information from all the phases.

On the other hand, construction also provides information to the other pavement management phases. The general nature of this set of information flows is given in Figure 7.2. The diagram again shows that information from construction is vital to efficient management in the other phases.

7.2.5 Documentation of Construction Data

It must be emphasized that the function of construction does not end with providing the actual physical facility but that it should also document the as-built properties or condition of the pavement. This represents important management information, and it should be acquired and processed in a manner that is easily understandable and usable.

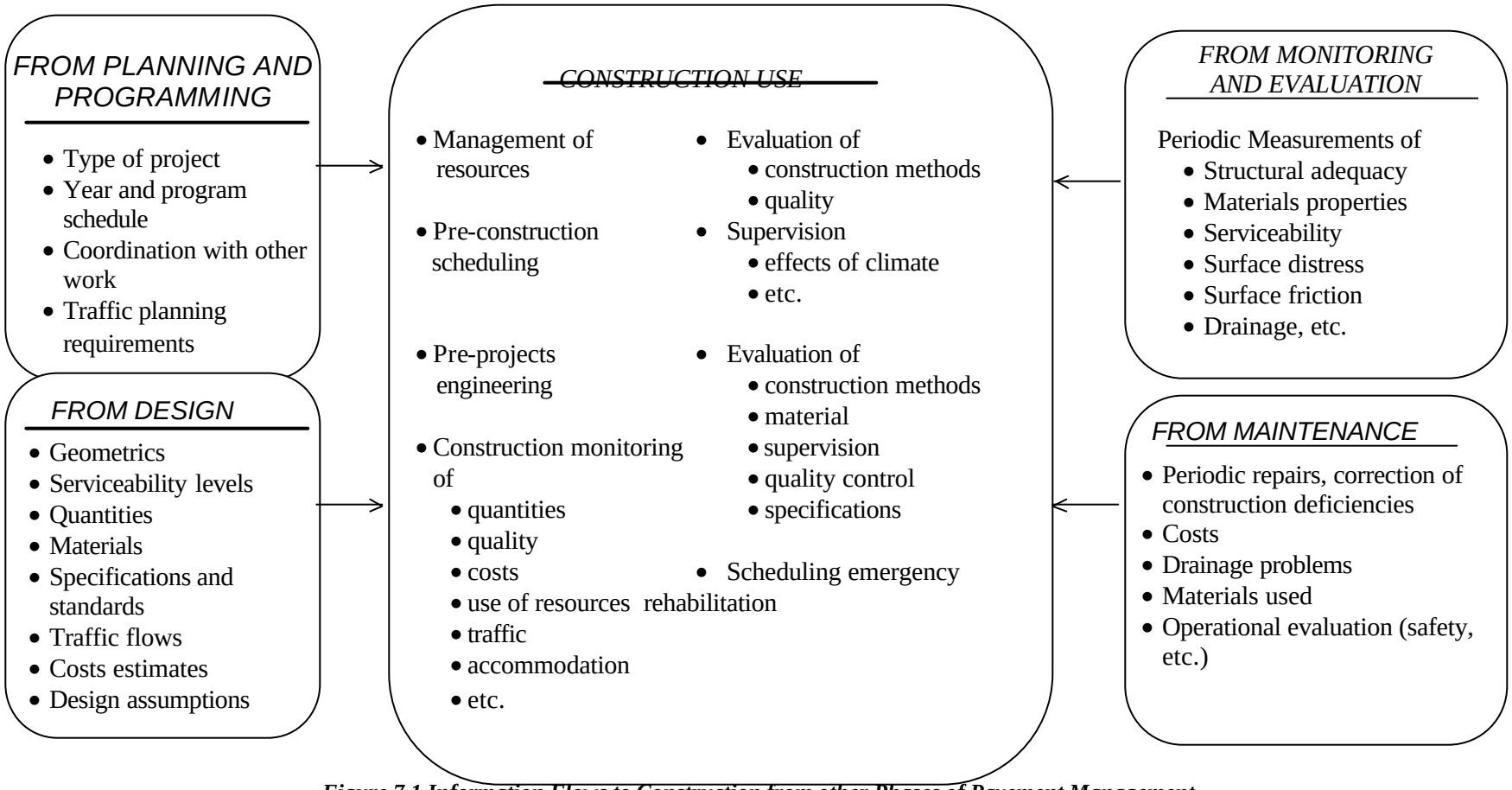


Figure 7.1 Information Flows to Construction from other Phases of Pavement Management

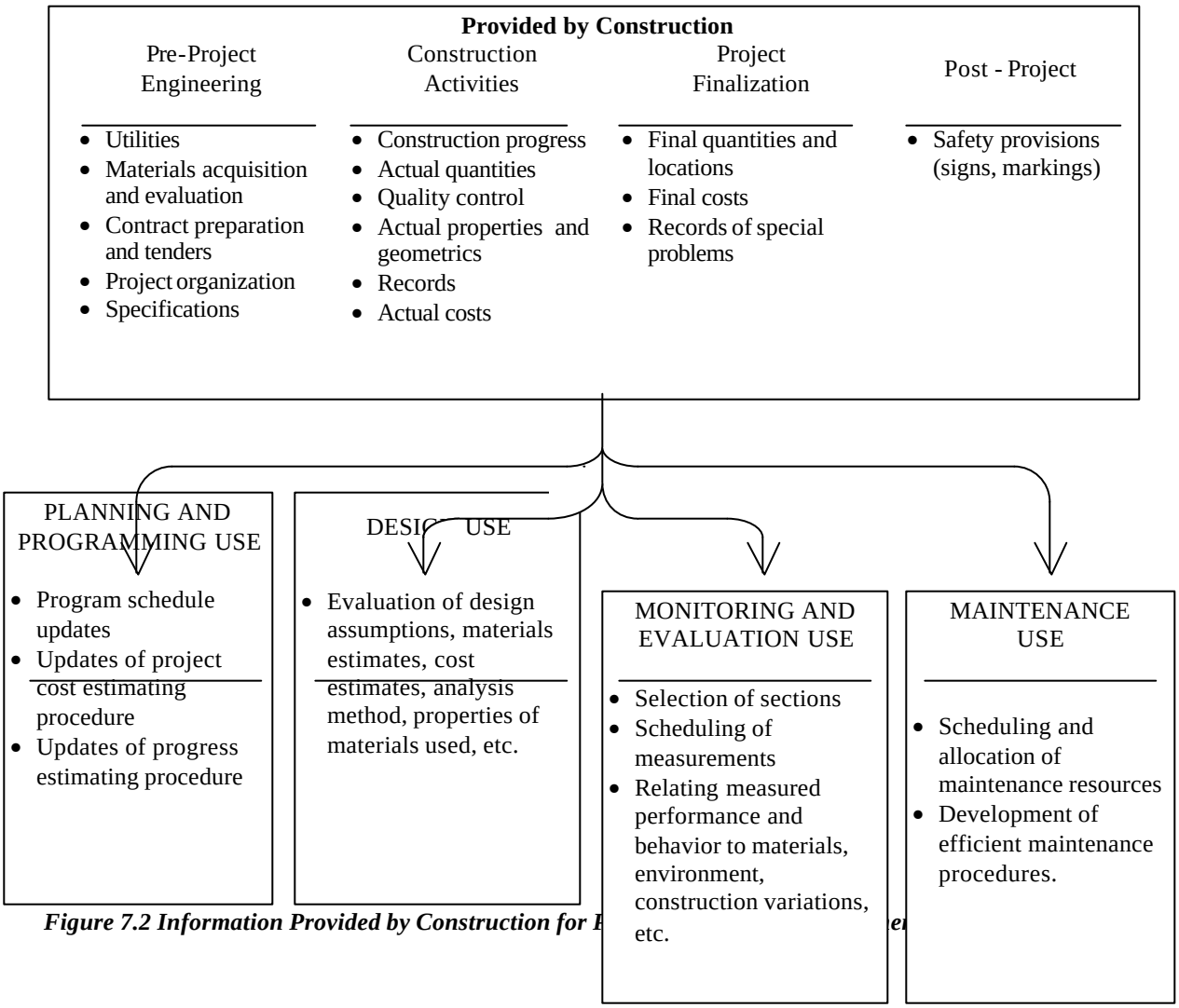


Figure 7.2 Information Provided by Construction for P[illegible]ent

A considerable amount of data can be generated in carrying out a pavement construction project. Records of these data have the following basic functions:

- Documented evidence that the project was constructed according to the contract specifications, for construction management purposes (QA/QC) [payment, administration, etc.].
- A database to assess the adequacy of standards and practices used effectiveness of the quality control methods, costs of the various operations, and the effects of construction practices on pavement performance.
- Quality of construction test results (QA/QC) and ratings for use in evaluating, analyzing, and understanding future pavement performance.
- Data for other phases of pavement management.

Each highway agency has forms for documenting construction project data. It is beyond the scope of this course to present these forms, but a complete example set for pavement projects is available [RTAC 77]. The major functional classes of pavement construction project data that should be included are shown in Figure 7.3. This is not an absolute classification in that there is considerable overlap, but it follows the basic functions previously listed.

Some of the important types of as-built construction information can be classified as follows:

- Type and thicknesses of pavement structure actually built, and over what lengths

- Actual material quantities used, their tested properties, and the variations in properties
- Material types and sources
- Actual costs (total and unit)
- Actual construction dates or times
- Pertinent records of weather, rainfall, drainage problems, traffic during construction, and so on
- Initial structural adequacy of the pavement
- Initial roughness of the pavement

The latter two types of information may be handled as a part of the pavement evaluation function. Nevertheless, they do represent important pieces of as-built construction information and should not be neglected.

The extent and quality of a pavement management system is dependent, to some degree, on the extent of the available data. For construction management, two basic levels of data collection can be used:

1. A minimum level aimed at providing minimal job history information, and
2. A higher level, depending on the capabilities and requirements of the particular agency, to serve as a more detailed basis for a pavement management system.

When the level of data to be collected has been established, the information has to be coded and organized for use in a computerized processing system. The coding methods adopted should be consistent with those employed in other phases of the pavement management system.

The following sections consider the subject of construction documentation and information in somewhat more detail.

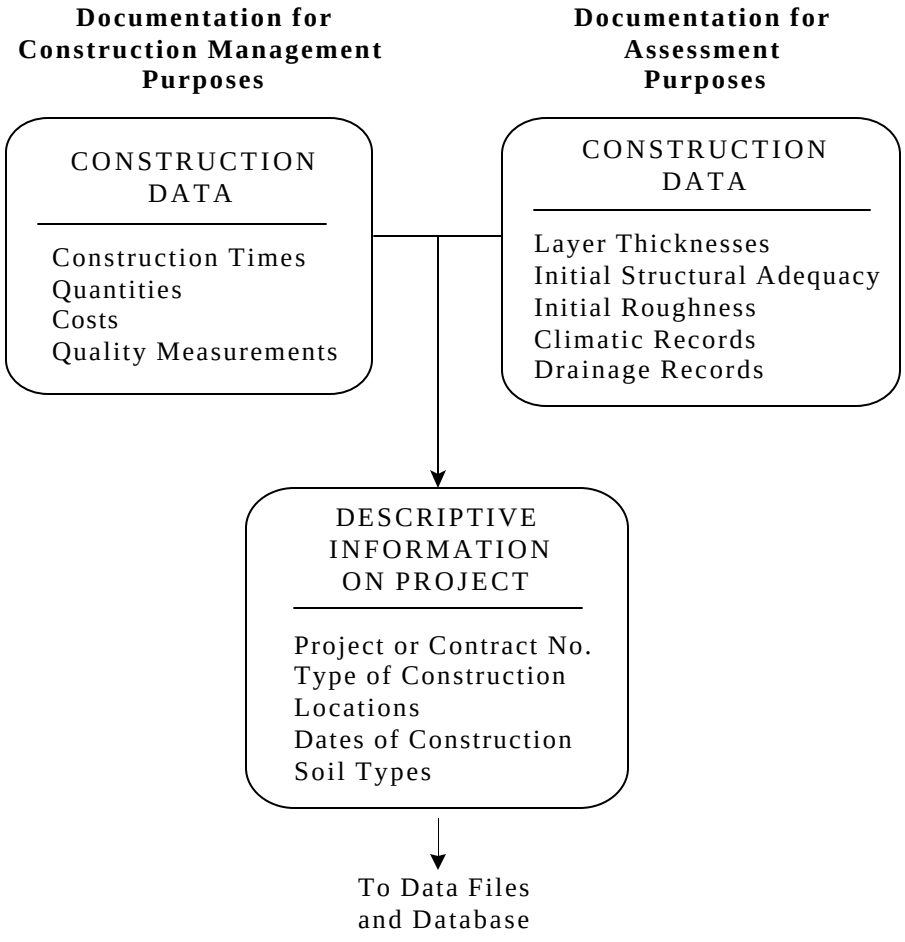


Figure 7.3 Major Functional Classes of Construction Information Documentation

Descriptive Information on Project

General descriptive information on pavement projects should include the project or contract number, the dates of construction, types of construction and specific locations (i.e., beginning and end points), subgrade soil types and locations, etc.

In addition, information on such items as contractors, subcontractors, various external factors (project shutdowns, materials supply problems, etc.) may be acquired, depending on the agency and the nature of the project.

Data for Construction Management Purposes

Figure 7.3 identified one class of data related to construction management purposes. Although the component data types are part of the overall project data that can be used for subsequent assessment and other pavement management purposes, they have been separately identified to indicate their initial role for purposes contract administration and payments.

The major types of data within the classification are construction times, actual quantities of subbase, base, surface and other materials processed or used, actual costs incurred, and quality measurement data (aggregate gradation, compaction, percent of binder, voids, etc.).

Quality control, as previously discussed, is one of the most important aspects of construction management on a pavement project. Quality control not only provides a basis for preventive or corrective action but also a means for observation or documentation for subsequent analysis. Although construction quality control should contribute to uniformity and better performance, it is not a substitute, for example, for good design and specifications. Rather, it should be consistent and compatible with other aspects of pavement management.

Construction Data for Assessment Purposes

The third major class of construction data documentation of Figure 7.3 relates to its use in assessment. It includes information on geometry (i.e., layer thicknesses), structural adequacy and roughness immediately after construction and on such climatic and drainage factors as rainfall, temperature, water table, and scour problems.

Some of this data may be acquired during construction and some are obviously acquired on the new, as-built pavement. Again, there is considerable overlap of this class of data with the other two classes shown in Figure 7.3. However, they have been separately identified to emphasize the importance of construction data for not only assessing construction practices, quality control procedures, standards and specifications, but also for assessing the effects of construction on pavement performance.

Types of Data to be Collected

Figure 7.3 illustrated the major classes of construction information that should be acquired. Figure 7.4 [RTAC 77], gives a summary set of guidelines related to typical construction data, data items and their collection, use, and frequency. The data in this figure is related to three levels of construction management: central, regional, and field. The data are also divided into a pre-construction or preparation stage, a during-construction stage, and an after-construction or final stage.

Management Level	Data Items	
	a. Pre-construction stage	
Central	1. Credit allowance 2. Bid publication 3. Bid opening 4. Bid analysis 5. Choice of Contractor	6. Contract signing 7. Schedule of contract 8. Number of working days 9. Number of contracts in active status by the low bidder 10. Pavement design data
Regional	1. Expropriation 2. General project description 3. Quantities and unit price 4. Interference of other projects, municipalities, companies, etc.	5. Mix designs 6. Order to start work 7. Contractor's list of labor & equipment 8. List of personnel for surveillance and quality control
Field	1. Survey 2. Preparation of "right-of-way" and "land access" plans 3. Determination of quantities 4. Climatic zone	5. Terrain classification 6. Public utilities: telephone, underground ducts, and electricity 7. Material sources and construction products
Management Level	**Daily**	**Monthly**
	b. Construction stage	
Central	1. Approval of work overruns 2. Notice of claims	1. Pay quantities for each type of work and percent complete 2. Payment
Regional	1. Work overruns checked against design and credit allowance 2. Actions against claims 3. Minutes of field meetings	3. Verification of paid quantities and estimation 4. Analysis of delays construction control test results summary
Field	1. Work overruns 2. Contract daily statistics 3. Quantities and costs 4. Working and downtime days 5. Material quality control test results; mix design test results; construction control test results 6. Diary of operations	1. Resume of decisions and actions taken at field meetings
Management Level	**Information Items**	
	c. Post-construction stage	
Central	1. Construction control summary 2. Resume of statistics on costs, working days, downtime days on account of strikes, expropriation, public utilities, etc.	3. Owner's surveillance costs 4. Quality variations from design 5. Quality of finished product
Regional	1. Construction data 2. Claims 3. Constructed pavement test results	4. Resume of quantities and cost 5. Last field inspection for final payment
Field	1. Engineer's report 2. Subgrade soils	3. Drainage

Figure 7.4 Some General Guidelines Related to Construction Data Collection [RTAC 77]

Certain types of climatic and drainage data during the construction period should also be acquired, because these two factors can affect pavement life and performance. There are other environmental factors reflected mainly in design considerations that also influence performance, data for these are usually gathered in the design or pavement evaluation phases. These other factors include the general geographical-climatic environment, the climatic environment expected and actually encountered subsequent to the construction, and so on. Climatic conditions during the construction period can have a highly significant effect on the performance of a pavement and, therefore, provide an important input into a pavement management system.

Climatic data acquired during the construction period can be used for the following purposes:

1. Provision of vital background information on actual job conditions for any subsequent analysis, negotiation, or adjudication of claims with a contractor.
2. Adjustment of design and construction standards in a particular region or environment to minimize the influence of climatic conditions in subsequent construction.
3. Explanation of any unusual variations in pavement performance as a result of climatic variables. The information could also be used to include the effects of climate in serviceability-age (or traffic) models used in design.
4. Assessment of the effects of climate on materials and maintenance practices.
5. Development of realistic construction schedules and completion dates.

The minimum climatic data that should be acquired during the construction of a pavement includes general written summaries and diaries of climatic conditions and time lost due to climatic conditions. For a more complete evaluation, the climatic factors of temperature, rainfall, snowfall, and wind can be recorded in terms of daily high, low, and average temperatures, daily rainfall, daily snowfall, and daily maximum wind velocity. Such climatic data can be summarized in tabular or graphical format as required.

The drainage characteristics of a pavement and its surrounding area can also have a significant effect on performance.

Information on drainage can be used for the following purposes:

1. To provide advance warning of likely maintenance problems resulting from drainage deficiencies such as high water table, ponded water, and scour potential
2. To improve subsequent construction cost estimates where drainage is likely to be a problem
3. To evaluate design methods and materials employed to control drainage

The minimum drainage data that should be acquired includes a general description of the drainage characteristics of the project, departures from design requirements or standards, and reports of deficiencies in subbase, base, and surface courses as a result of drainage factors during construction.

7.3 USING PMS MODELS TO COMPARE EFFECTIVENESS OF OVERLAYS AND SURFACE TREATMENTS FOR ADOT

7.3.1 Background

Over the years ADOT has used a variety of materials and techniques for overlays and surface treatments, depending upon the requirements of the pavement sections involved. Most of these treatments and materials were selected because of past performance history in Arizona. The arrival of several new and innovative materials, techniques and specifications, encouraged increasing use of several new treatments over time. A project carried out by TRDI for ADOT [ADOT 99] provided an opportunity to make an unbiased comparison of treatments on the basis of up to 15 years performance data from the PMS database. It should be stressed that the evaluation was based on observed PMS data rather than a controlled design experiment. Consequently, valuable information can be gained by “trend” review, regardless of statistical significance of differences in performance.

7.3.2 Approach

From all available layer combinations in the PMS project file, the appropriate "overlay" projects have been identified, categorized as different "overlay" techniques, and further analyzed.

Table 7.4 lists the various materials and techniques used by ADOT, together with their abbreviations as listed in the PMS database. Note: For "mill and fill" the abbreviation R&R has been used if material was replaced to the original level, otherwise RE (+thickness) and AC (+thickness) was used.

A. Functional Treatments

1. Seal coat (SC)

2. Open graded asphalt concrete friction course (FC)

3. Open graded asphalt concrete friction course with asphalt rubber (FR)

4. Mill and fill with asphalt concrete friction course (RE+FC)

5. Mill and fill with asphalt concrete friction course with asphalt rubber (RE+FR)

B. Structural Treatments

1. Asphalt concrete overlay (AC)

2. Asphalt concrete overlay with seal coat (AC+SC)

3. Asphalt concrete overlay with asphalt concrete friction course (AC+FC)

4. Rubberized asphalt concrete overlay (AR)

5. Mill and fill with AC overlay with FC surface (R&R+FC, or RE+AC+FC)

6. Mill and fill with AC overlay with FR surface (R&R+FR, or RE+AC+FR)

7. Mill and fill with AR overlay (RE+AR)

8. Mill and fill with AR overlay and FR (RE+AR+FR)

9. Recycling (RC)

Table 7.4 Materials and Techniques used by ADOT and their Abbreviations

CODE	DESCRIPTION	CODE	DESCRIPTION
AB	AGGREGATE BASE	LB	LIME TREATED BASE
AC	ASPHALTIC CONCRETE	LC	LEVELING COARSE - AC,AZMO
AR	AC WITH ASPHALTIC RUBBER	LS	LIME SUBGRADE
AS	ACSC – ASPH.CONCR. SURF. COARSE	MC	MIX & COMPACT EXISTING MATERIALS
BB	BITUMINOUS TREATED BASE	OA	OPEN GRADED BASE MATERIAL
BM	BASE MATERIAL - AB, SM	OB	OPEN GRADED BITUMINOUS TREATED BASE
BS	BITUMINOUS TREATED SURFACE	OC	OPEN GRADED ASPHALTIC CONCRETE

CB	CEMENT TREATED BASE	PC	PORTLAND CEMENT CONCRETE (PCC)
CF	CONSTRUCTION FABRIC	PD	PCC, DOWELLED
CL	LEAN CONCRETE BASE	PP	PCC, PRE-STRESSED
CS	CEMENT TREATED SUBGRADE	PR	PCC, CONTINUOUSLY RE-INFORCED
DC	DOUBLE CHIP SEAL (2 EMULSIFIED ASPHALT APPLICATIONS)	PS	PLANT MIX SEAL COAT
FB	FLY ASH BASE	RC	RECYCLED AC - ASPHALT REMOVED, REJUVENATED, REPLACED
FC	ACFC, ASPH. CONC. FRICT. COURSE	RE	REMOVE EXISTING MATERIAL
FF	FILTER FABRIC	RF	ROCK FILL
FL	FLUSH COAT - FOG SEAL	RM	RUBBER. MEMBR. (INTERLAYER or SEAL CT)
FR	ACFC WITH ASPHALTIC RUBBER	RO	RECYCLED AC OVERLAY
FS	FLY ASH SUBGRADE	SB	AGGR. SUBBASE (SIMILAR TO SELECT MATL)
GR	GRIND	SC	SEAL COAT - COVER MATL with EMULS.ASPH.
GV	GROOVE	SM	SELECT MATERIAL
HS	HEATER SCARIFICATION	SR	SLURRY SEAL
KS	CRACK & SEAT PCCP	SS	SUBGRADE SEAL

For the structural treatments sufficient data is available for those identified above as B1, B2, and B3. Little or no data exist for B4 (AR overlay). For the mill and fill treatments this analysis did not differentiate between "mill and fill" and "mill and fill with AC overlay" because the "fill" is
also considered an AC overlay. In the evaluation the treatments B5, and also B6, were therefore combined in the treatments. B7 has sufficient data, but no records were found for B8.

For B9 (recycling) the database does not differentiate between hot and cold recycling. However, data is available for the following four treatments:

1. Recycling only (RC),
2. Recycling + asphalt overlay + asphalt friction course (RC-AC-FC),
3. Remove material + recycle + asphalt overlay + asphalt friction course (RE-RC-AC-FC), and
4. Remove material + recycle + asphalt friction course (RE-RC-FC). The first two are presumably recycled in-situ; the last two are plant mixed.

In summary, Table 7.5 shows the treatments selected and extracted from the project database.

Table 7.5 Overlay and Material Treatments Tested

Nr	Treatment	Description	Code	Records
	Functional			

1		Asphalt Concrete Friction Course (ACFC)	FC	209
2		Asphalt Rubber Friction Course	FR	47
3		Mill + Asphalt Concrete Friction Course	RE-FC	42
4		Mill + Asphalt Rubber Friction Course	RE-FR	27
5		Seal Coat– cover material with asphalt	SC	679
6	Structural	Asphalt Concrete	AC	115
7		Asphalt Concrete + ACFC	AC-FC	402
8		Asphalt Concrete + Seal Coat	AC-SC	189
9		Asphalt Concrete + Flush Coat	AC-FL	756
10		Recycle in situ	RC	10
11		Recycle + asphalt overlay + ACFC	RC-AC-FC	83
12		Mill + AC overlay	RE-AC	98
13		Mill + AC overlay + ACFC	RE-AC-FC	226
14		Mill + AC overlay + rubber ACFC	RE-AC-FR	119
15		Mill + AC with asphalt rubber	RE-AR	11
16		Remove + (plant)recycle + AC overlay + ACFC	RE-RC-AC-FC	46
17		Remove + (plant)recycle + ACFC	RE-RC-FC	15

7.3.3 Runs with Visual Modeler

As a first step, a dataset was assembled for the 22 treatments listed in Table 7.5 for all projects since the 1920s. The number of records found for each treatment is also given in this table. Each record represents a pavement section within a project, often with a length of several miles. The performance indexes are collected at each mile point and the dataset for these treatments cover many years, so the number of data points for performance measurements is five to 15 times higher than the number of records in the project's database. For example the number of records for AC is 115, but the number of data points for roughness of AC is 1254.

The condition (performance) dataset for the years 1981-1994, whose age of last rehabilitation was 15 years or less, was processed for analysis and imported into Visual Modeler [Perrone 98], [TRDI 98] for detailed analysis. The reader is referred to Chapter 4 for a description of this modeling tool.

Early work with Visual Modeler on ADOT's PMS database made it clear that roughness was by far the most useful performance indicator to distinguish between different materials and circumstances. Consequently it was decided to concentrate on roughness for these overall comparisons. Data for friction were evaluated for most of the functional overlay materials, and for structural overlay materials as far as they were exposed to traffic. In a similar way data for cracking and rutting were evaluated for several groups of treatments.

It should be stressed that a complete comparison is only possible when other variables such as the cost of the treatment, the construction time and traffic delay, sensitivity to adverse weather during construction, environmental aspects, etc, are considered also.

Apart from these general considerations it should be realized that these comparisons are also influenced by the fact that in the database with overlay techniques/materials no information is available about the structure underneath the overlay (other than the type of wearing course). It is obvious that such structure can have a major influence, particularly on roughness, but for thinner layers also on cracking and rutting. There are currently no simple ways to include that information in the database. Another point is that the roughness immediately prior to overlay was not used. It might be that certain pavement structures were smoother prior to overlay than others. In short: this was not a controlled design experiment.

Typical Analysis Results

The analyses run in this section resulted in generally four types of data fits. We have labeled these types A, B, C, and D for easy reference. These are illustrated in the next four figures.

Type A data fit is the most typical. There are an adequate number, generally a large number of datapoints, over a 14-year period, which results in a good fit (See Figure 7.5).
Type B data fit is typical of newer type treatments. This provides a good fit over fewer datapoints and a shorter observed timeframe. (See Figure 7.6)

Typical Type C data fit defines a problem area. In these cases the data are heavily biased to an early age such as one or two years with lots of variability. Not only the distribution but the data trend is biased. The results produce an abnormal slope, which is counter intuitive and not useful. (See Figure 7.7).

Typical Type D data fit is defined by a reasonable number of datapoints over a short timeframe with a few outlier points, which have little effect. These data sets produce useful results for the comparisons in this chapter (see Figure 7.8).

The following sections present data for roughness, friction, cracking and rutting in a standardized format. For each performance indicator a ranking is made of the effectiveness of the treatments, and at the end an assessment is made about the overall ranking.

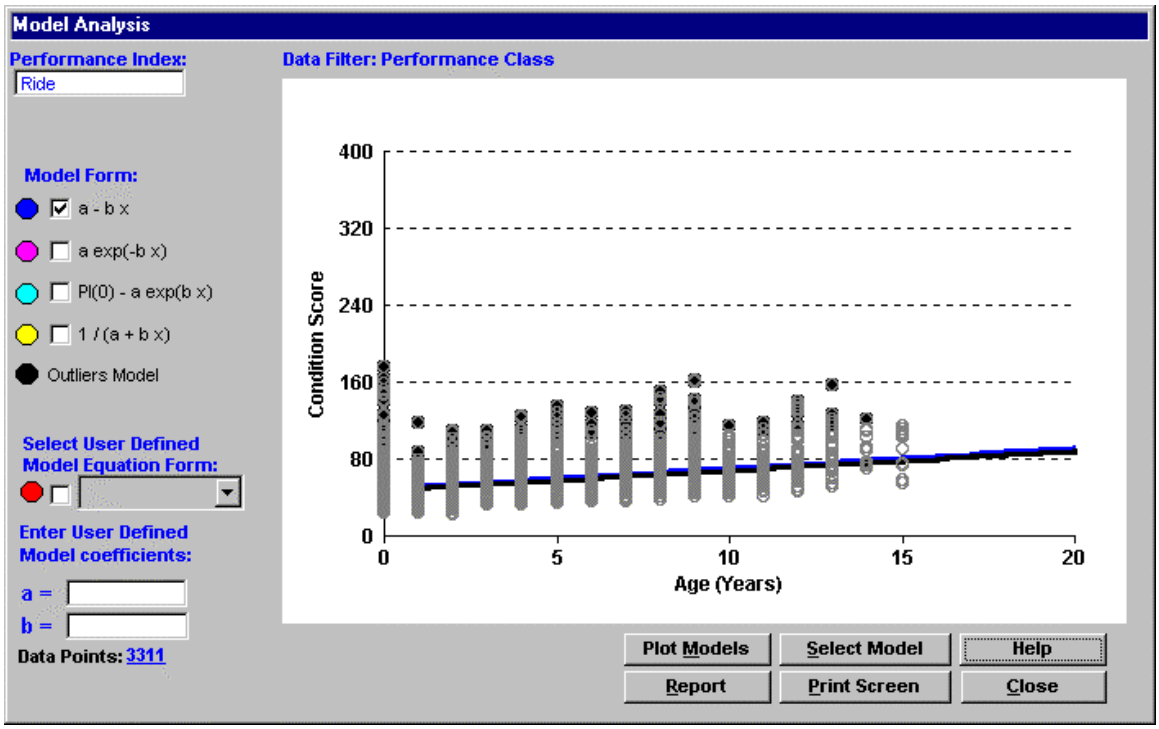


Figure 7.5 Typical Type A Data Analysis Fit - Many Data Points - Generally Good Fit

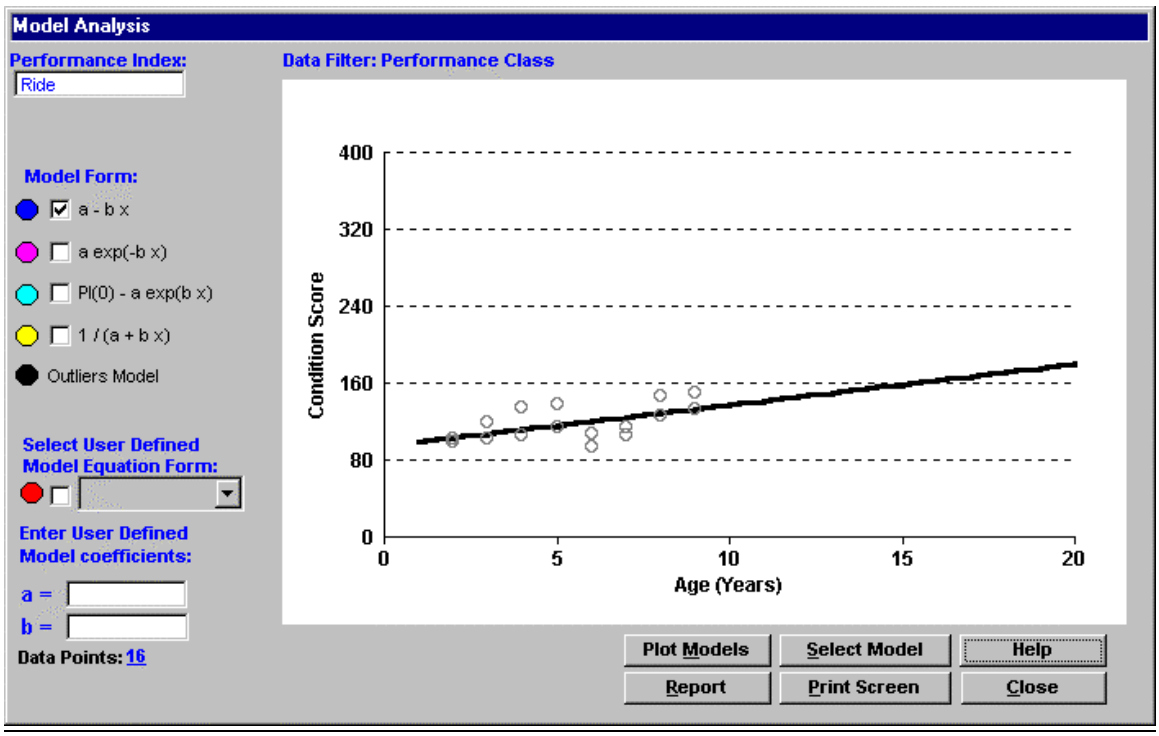


Figure 7.6 Typical Type B Analysis - Generally Good Fit for Datapoints

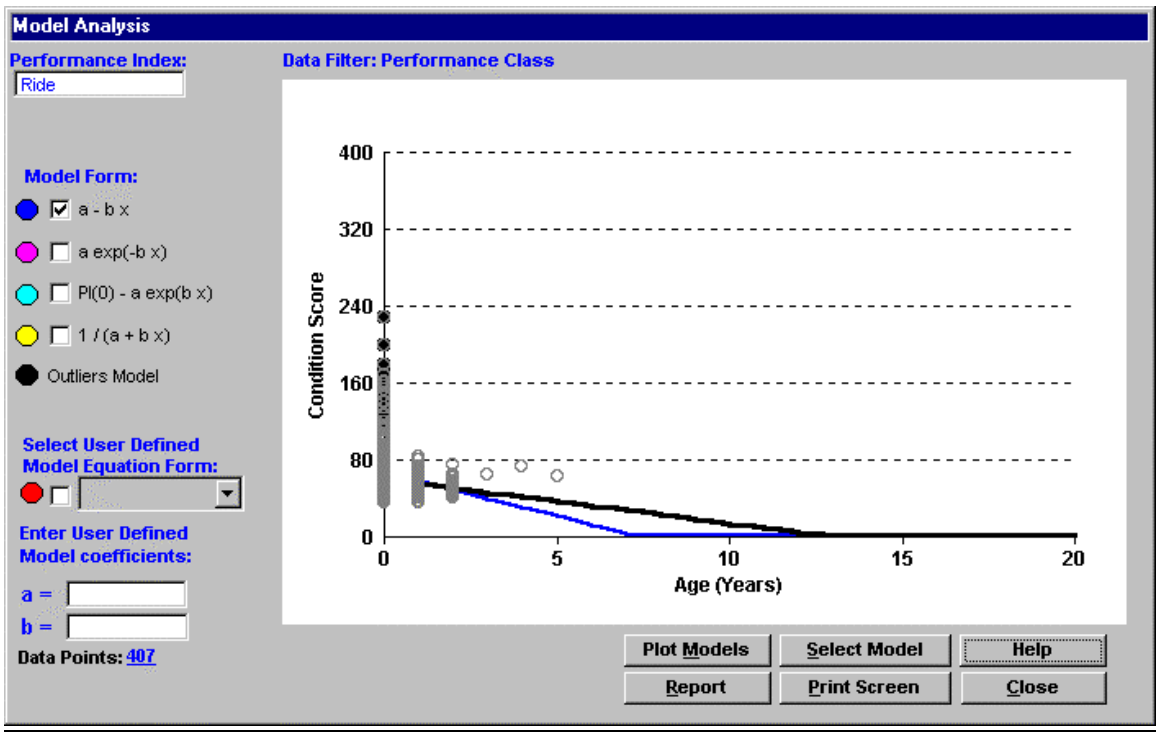


Figure 7.7 Typical Type C Analysis - Poor Fit, Biased Distribution, Wrong Slope

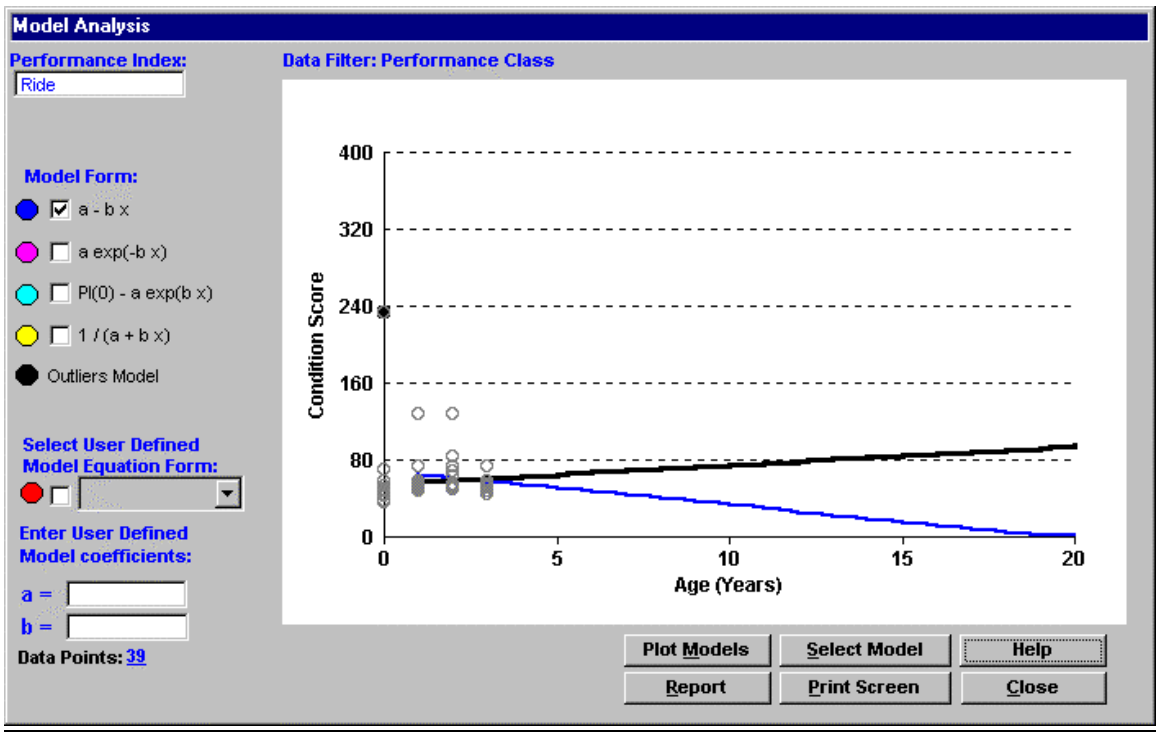


Figure 7.8 Typical Type D Data Analysis - Good Fit for Short Timeframe after Removal of Obvious Outliers

7.3.4 Roughness Data

In order to be able to make mutually meaningful comparisons between various treatments, they were divided into groups, as shown in Table 7.6. These five groups cover roughness data from 1981 to 1994 for structural and functional overlays and surface treatments for all pavements.

Table 7.6 Groups of Overlays and Treatments for Roughness Evaluation

Structural Overlays + Surface Treatments		**Structural Overlays + Friction Courses**	
AC	Asphalt Concrete	ACFC	Asph. Concr.+ AC Frict. Course
REAC	Layer removal + AC	REACFC	Layer rem.+AC+ACFrict.Course
REAR	Layer removal + Rubber AC	REACFR	L.removal +AC+ RubberACFC
ACSC	AC + Seal Coat		
ASFL	AC + Flush Coat		
Functional Overlays		**Recycling Treatments**	
FC	Asph Concr Frict Cours (ACFC)	RC	In-place Recycling
FR	Rubberized ACFC	RCACFC	Recycling+ACFC
REFC	Layer removal + ACFC	RERCFC	Remove, (plant) Recycle+ACFC
REFR	Layer removal + Rubber ACFC	RERCACFC	Remove, (plant) Recycle+AC overlay+ ACFC
SC	Seal Coat		

For each group a summary of the results from Visual Modeler is presented in three parts:

1. A graph for each treatment in the group based on its model, see Figures 7.9 – 7.12. The original graphs from Visual Modeler with data points can be found in Appendix B, which is published separately. For the models on roughness and friction the intersections with the vertical axis relate to their initial values after construction. For cracking and rutting these initial values should be zero, but this was not always the case, because the linear regression

model was not forced through condition=zero at age zero. The slope of each line is a measure of the rate of deterioration with time for that particular treatment.

2. A table containing information about the treatments concerned is presented as part of each figure. The headings show the number of data points, the length of time (approx. five, ten, or 15 years) during which performance was measured, and the significance of the model. Further information about the division of data points over Interstates and other roads, about the model and its statistical data and significance is given in Appendix B, which, together with Appendix A, is printed as a separate volume.

3. A description of the major trends inside each group is then presented.

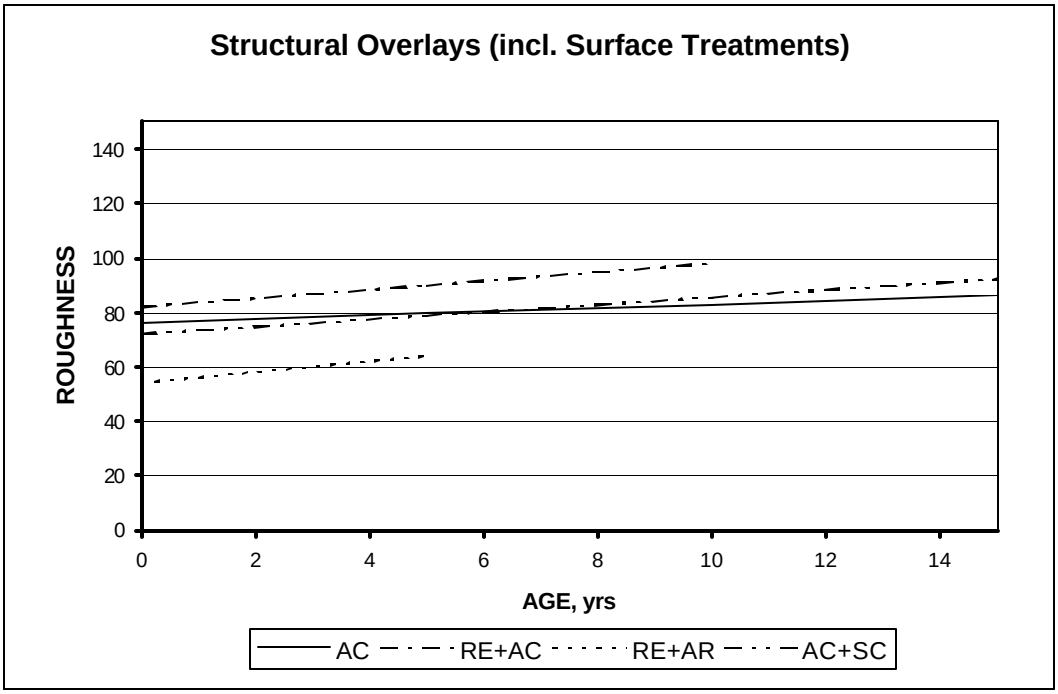
In reviewing these results it is important to note that the ADOT criteria for roughness (In accumulated inches per mile) are:

Satisfactory	<93
Tolerable	93-143
Objectionable	>143

Roughness of Structural Overlays (including Surface Treatments)

The following trends can be observed for this group of structural overlays as shown in Figure 7.9.

- RE+AR shows the lowest initial roughness, but the significance of this curve is very low, and the age of all datapoints is below five years.
- AC, AC+SC, and RE+AC have similar slopes and have a high significance for the analysis and a similar low rate of deterioration.



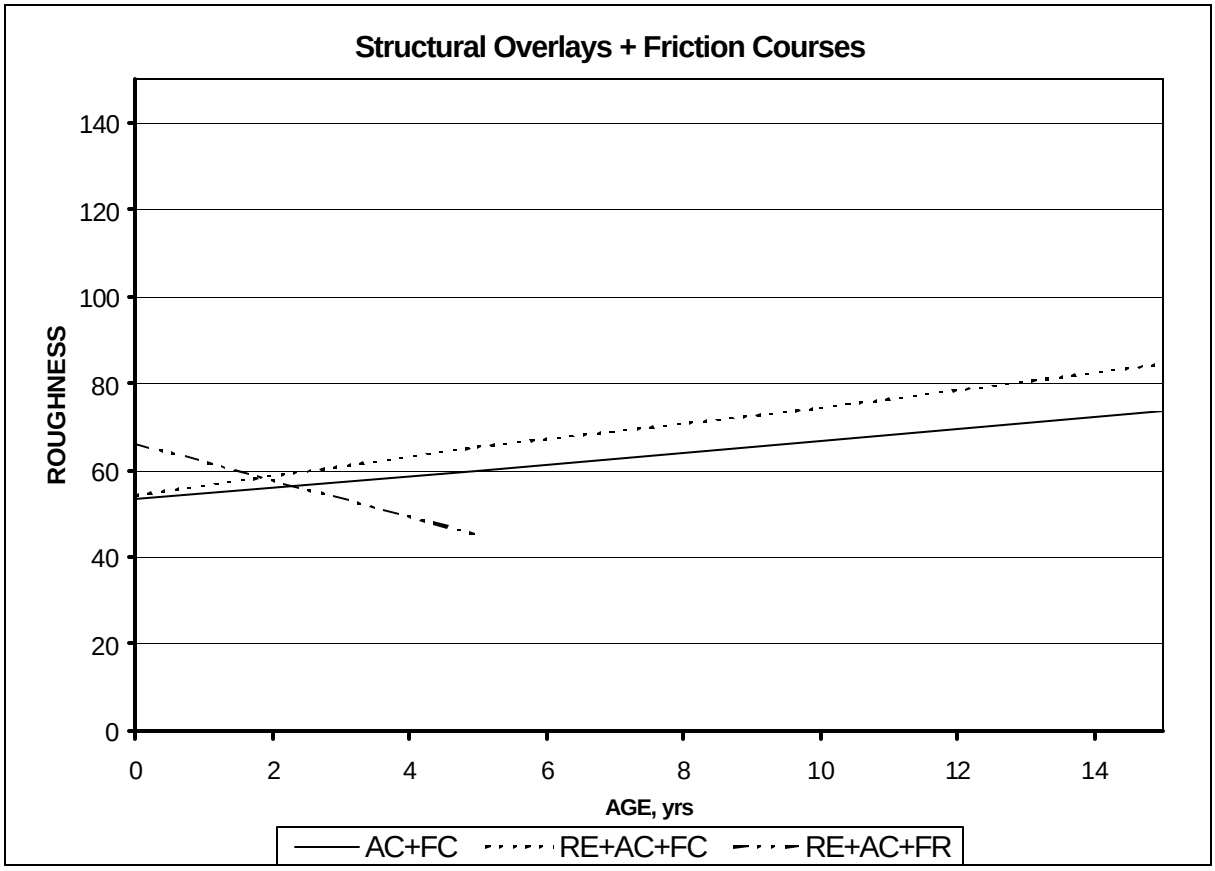
Roughness of Structural Overlays, incl. Surface Treatments				
Code	**Description**	**Nr. of Records**	**Max Age**	**Significance**
AC	Asphalt Concrete	1254	15	High
REAC	Layer removal + AC	539	10	High
REAR	Layer removal + Rubber AC	39	5	Very Low
ACSC	AC + Seal Coat	7205	15	High

Figure 7.9 Structural Overlays including Surface Treatments

Roughness of Structural Overlays + Friction Courses

The following trends can be observed for this group of structural overlays plus friction courses in Figure 7.10.

- AC+FC and RE+AC+FC all have a comparable low initial roughness of about 50-54 inches per mile and low deterioration rate of between 1.4 and 2.1 in/mi/yr. AC+FC shown as a solid line in Figure 7.10 have a slightly slower rate of roughness increase.
- RE+AC+FR has an abnormal slope (Type C data fit), which could have been caused by the low age range of data. The original Visual Modeler graphs are shown in Appendix B, which, together with Appendix A, is printed as a separate volume.



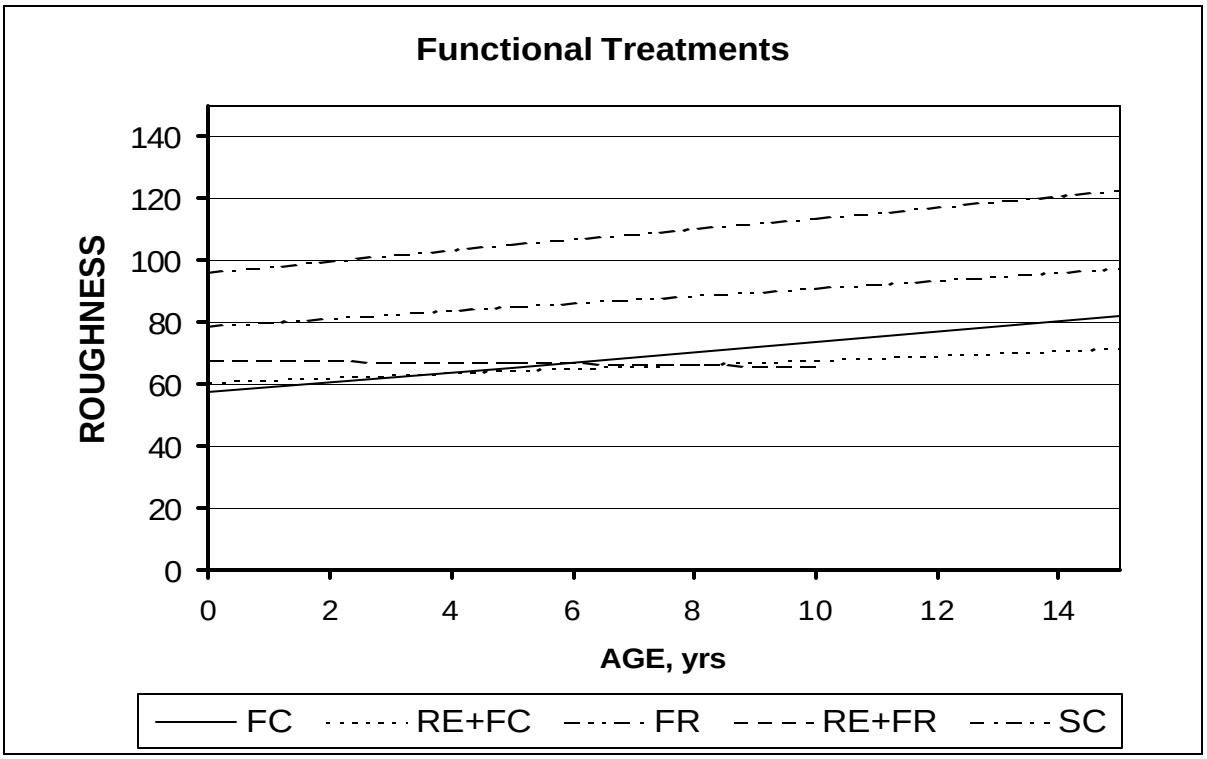
Roughness of Structural Overlays + Friction Courses				
Code	**Description**	**Nr. of Records**	**Max Age**	**Significance**
ACFC	Asph. Concr.+ AC Frict. Course	8352	15	High
REACFC	Layer rem.+AC+ACFrict.Course	3213	15	High
REACFR	L.removal +AC+ RubberACFC	407	5	Abnormal Slope

Figure 7.10 Structural Overlays + Friction Courses

Roughness of Functional Treatments

The following trends can be observed for this group of treatments in Figure 7.11

- FC and RE+FC have a comparable low initial roughness of about 60 in/mi and a low deterioration rate.
- Both analyses have a high significance and the RE+FC has the best overall demonstrated performance of this set of options.
- FR and SC have a somewhat higher initial roughness, but a similar rate of deterioration and also a high significance.
- RE+FR has a lower initial roughness than FR, but it has a fitted slope of almost zero as a rate of deterioration. The observed data limitation is 10 years.



Roughness of Functional Treatments				
Code	**Description**	**Nr. of Records**	**Max Age**	**Significance**
FC	Asph Concr Frict Cours (ACFC)	7284	15	High
FR	Rubberized ACFC	296	15	High
REFC	Layer removal + ACFC	692	15	High
REFR	Layer removal + Rubber ACFC	195	10	Zero Slope
SC	Seal Coat	17187	15	High

Figure 7.11 Functional Treatments

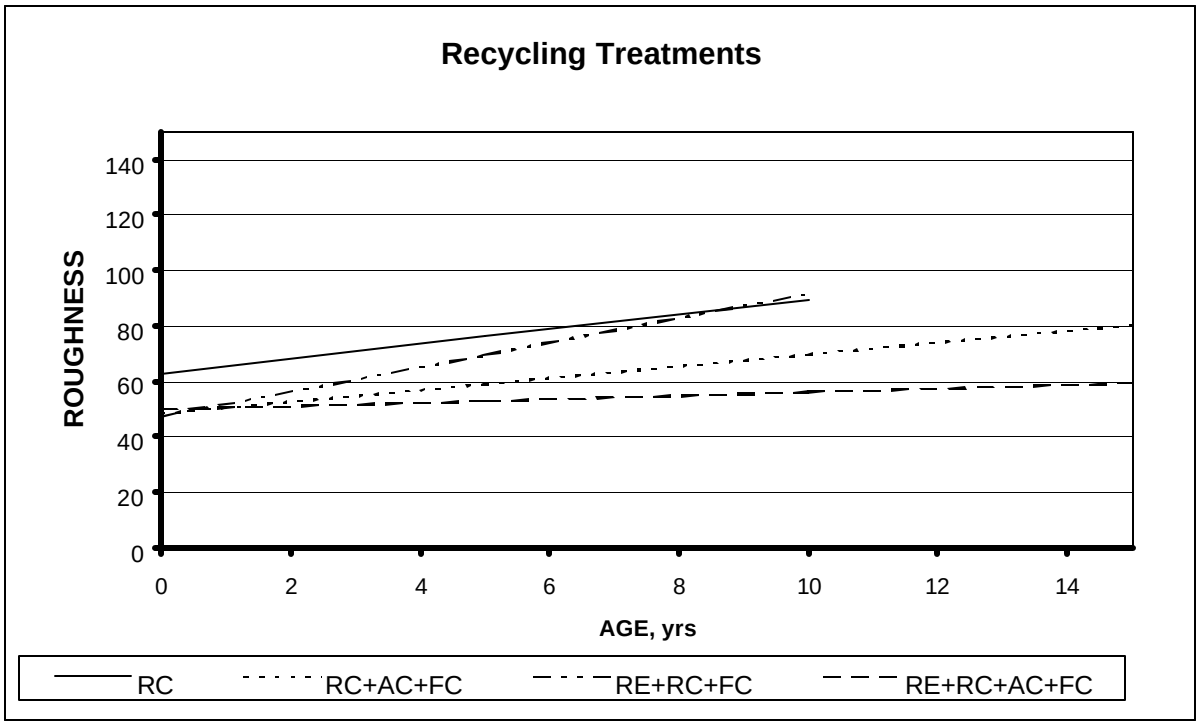
Roughness Data for Recycling Treatments

The following trends can be observed for these treatments as shown in Figure 7.12.

- RC+AC+FC, RE+RC+FC, and RE+RC+AC+FC all show the same initial roughness between 47 and 50 in/mi, smaller than RC at 63 in/mi.

- RE+RC+AC demonstrates the lowest rate of deterioration with a roughness slope of 0.6 in/mi/yr. RE+RC+FC has the highest rate of deterioration or slope of 4.4 in/mi/yr.
- The significance of all models is high except for RC, which has medium significance.

Clearly for this set of treatments RE+RC+AC shows the best overall performance



Roughness data for Recycling Treatments				
Code	**Description**	**Nr. of Records**	**Max Age**	**Significance**
RC	In-place Recycling	70	10	Medium
RCACFC	Recycling + ACFC	3311	15	High
RERCFC	Remove, (plant) Recycle+ACFC	363	10	High
RERCACFC	Remove, (plant) Recycle + AC overlay+ ACFC	1037	15	High

Figure 7.12 Recycling Treatments

Comparison of Treatments for Roughness

An overall comparison between the various materials and treatments has been carried out by determining the time it takes for each model prediction for that treatment to reach a tolerable roughness level of 93 in/mi (Table 7.7). This was done by extending each model plot to a level of 93 in/mi and determining the corresponding age. This is an "artificial" age score which is often well beyond the expected life of a treatment, but it can be used as a yardstick in the ranking. Those models with an abnormal slope were omitted from this comparison because the models are not valid and their "age" at a roughness of 93 in/mi could not be determined (Table 7.8). They will be discussed separately on the basis of their initial roughness levels. The treatments for "PCC only" have also been omitted because they are not comparable.

Table 7.7 Score for Models with Significant Slope (higher number means higher score)

Code	Description	Nr. of Records	Max Age	Significance	Score

RERCACFC	Remove, (plant) Recycle + AC overlay + ACFC	1037	15	High	69
REFC	Layer removal + ACFC	692	15	High	43
ACFC	Asph. Concr.+ AC Frict. Course	8352	15	High	29
AC	Asphalt Concrete	1254	15	High	25
FC	Asph Concr Frict Cours (ACFC)	7284	15	High	22
RCACFC	Recycling + ACFC	3311	15	High	21
REAR	Layer removal + Rubber AC	39	5	Very Low	20
REACFC	Layer rem. + AC+ ACFrict.Course	3213	15	High	19
ACSC	AC + Seal Coat	7205	15	High	16
FR	Rubberized ACFC	296	15	High	11
RC	In-place Recycling	70	10	Medium	11
RERCFC	Remove, (plant) Recycle + ACFC	363	10	High	10
REAC	Layer removal + AC	539	10	High	7
SC	Seal Coat	17187	15	High	0

Table 7.8 Score for Models with Abnormal Slope (lower number means higher score)

Code	*Description*	**Nr. of Records**	**Max Age**	**Significance**	**Score coeff a**
REACFR	L.removal +AC+ RubberACFC	407	5	Abnormal Slope	66
REFR	Layer removal + Rubber ACFC	195	10	Abnormal Slope	68

7.3.5 Cracking Data

Although roughness is by far the best performance indicator we also evaluated cracking for information. For the evaluation and comparison of cracking the treatments were divided into the following six groups shown in Table 7.9.

These six groups covered cracking data for various structural overlay materials, for functional overlays and surface treatments for all pavement records from ‘81 to ‘94.
The format shown is similar to that used for roughness data.

ADOT’s criteria for Cracking are:

Low <10%
Medium 10-30%
High >30%

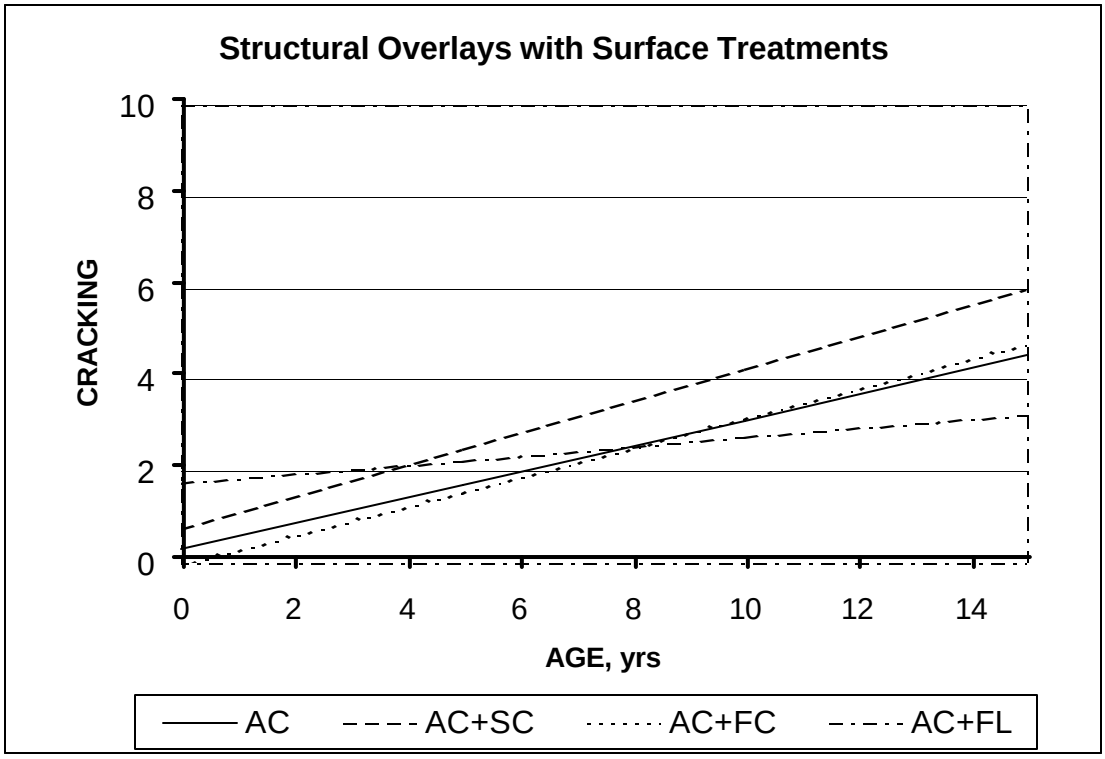
Table 7.9 Six Comparable Groups of Treatment Types

Structural Overlays with Surface Treatments		**Layer Removal + Structural Overlays**	
AC	Asphalt Concrete	REAC	Layer removal + AC
ACSC	AC + Seal Coat	REAR	Layer removal + Rubber AC
ACFL	AC + Flush Coat	REACFR	Layer removal+AC+Rubber ACFC
ACFC	AC + AC Friction Course	REACSC	Layer removal + AC + Seal Coat
Functional Treatments		**Recycling Treatments**	
FC	AC Friction Course (ACFC)	RC	In-place Recycling
REFC	Layer removal + ACFC	RCACFC	Recycling+AC+ACFC
FR	Rubberized ACFC	RERCFC	Remove,(plant) Recycle + ACFC
REFR	Layer removal + Rubber ACFC	RERCACFC	Remove, (plant) Recycle + AC overlay + ACFC
SC	Seal Coat		

Cracking of Structural Overlays with Surface Treatments

The following trend can be observed for the group of treatments shown in Figure 7.13.

- AC and AC+SC show a faster rate of deterioration than AC+FL



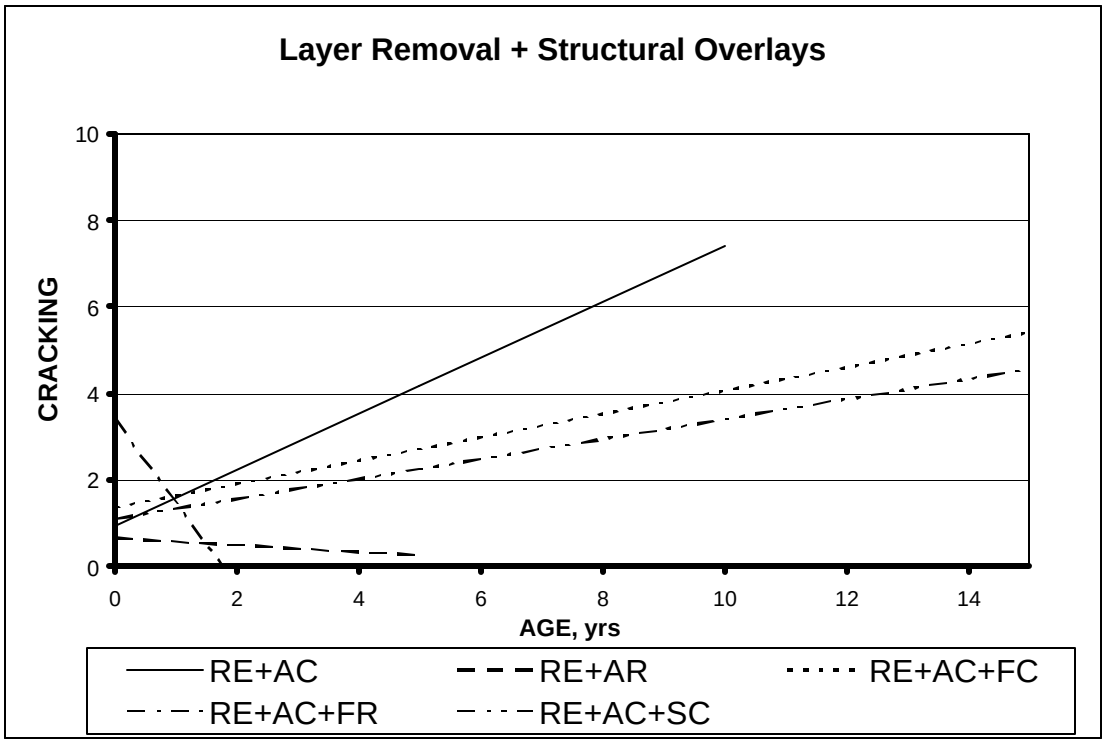
Cracking of Structural Overlay Materials with Surface Treatments				
Code	**Description**	**Nr of Records**	**Max Age**	**Significance**
AC	Asphalt Concrete	1237	15	Yes
ACSC	AC + Seal Coat	7210	15	Yes
ACFL	AC + Flush Coat	756	15	Yes
ACFC	Asph. Concr.+ AC Frict. Course	8364	15	Yes

Figure 7.13 Structural Overlays with Surface Treatments

Cracking after Layer Removal + Structural Overlays

The following trends can be observed for this group of treatments shown in Figure 7.14.

- RE+AC shows a much higher rate of deterioration than both RE+AC+FC and RE+AC+SC. These relationships are all significant.
- RE+AR and RE+AC+FR only have data for a few years, and these show an abnormal slope. The data suggest that the rubber additions have a beneficial influence.



Cracking after Layer Removal + Structural Overlays				
Code	**Description**	**Nr of Records**	**Max Age**	**Significance**
REAC	Layer removal + AC	554	10	Yes
REAR	Layer removal + Rubber AC	39	5	Abnormal slope
REACFC	Layer removal +AC+ACFrict.Course	3223	15	Yes
REACFR	Layer removal +AC+ RubberACFC	407	5	Abnormal slope
REACSC	Layer removal + AC + Seal Coat	306	15	Yes

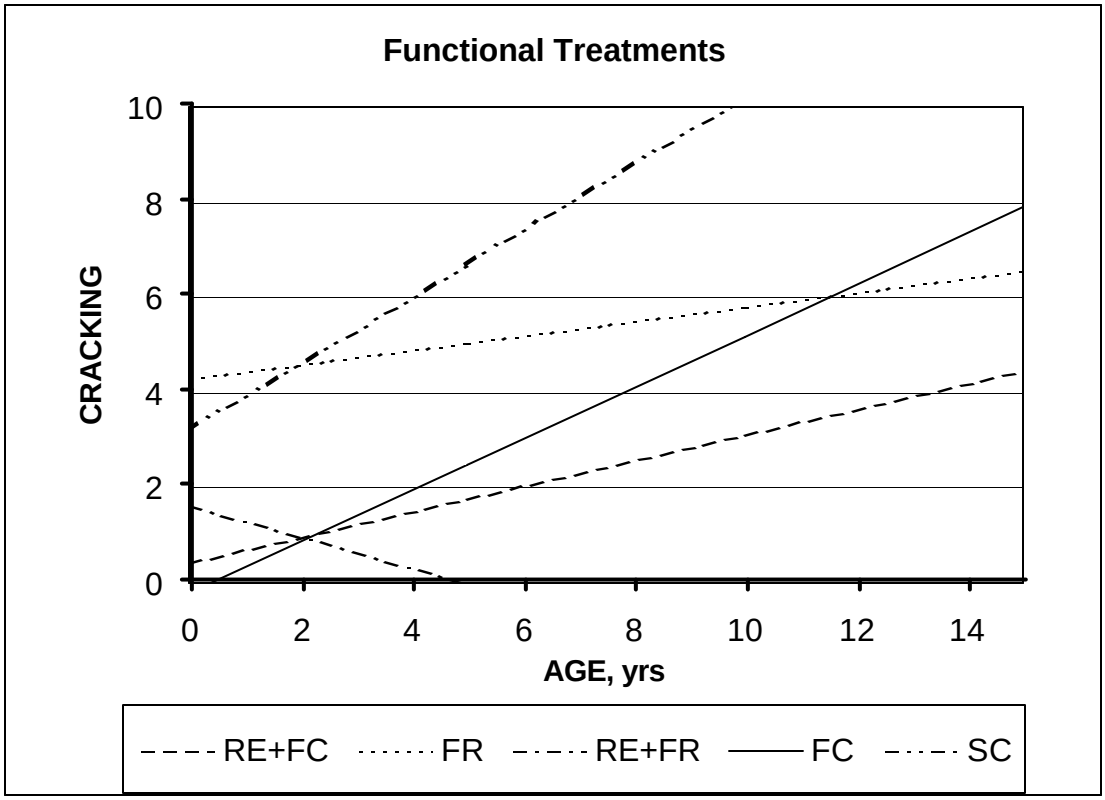
Figure 7.14 Layer Removal + Structural Overlays

Cracking of Functional Treatments

The following trends can be observed for this group of treatments shown in Figure 7.15.

- FC has a higher rate of deterioration than RE+FC, so prior milling seems beneficial.

- The model for FR is not significant, and RE+FR shows an abnormal slope, but also here prior milling suggests a benefit. The addition of rubber to the friction course also seems beneficial for the rate of deterioration.



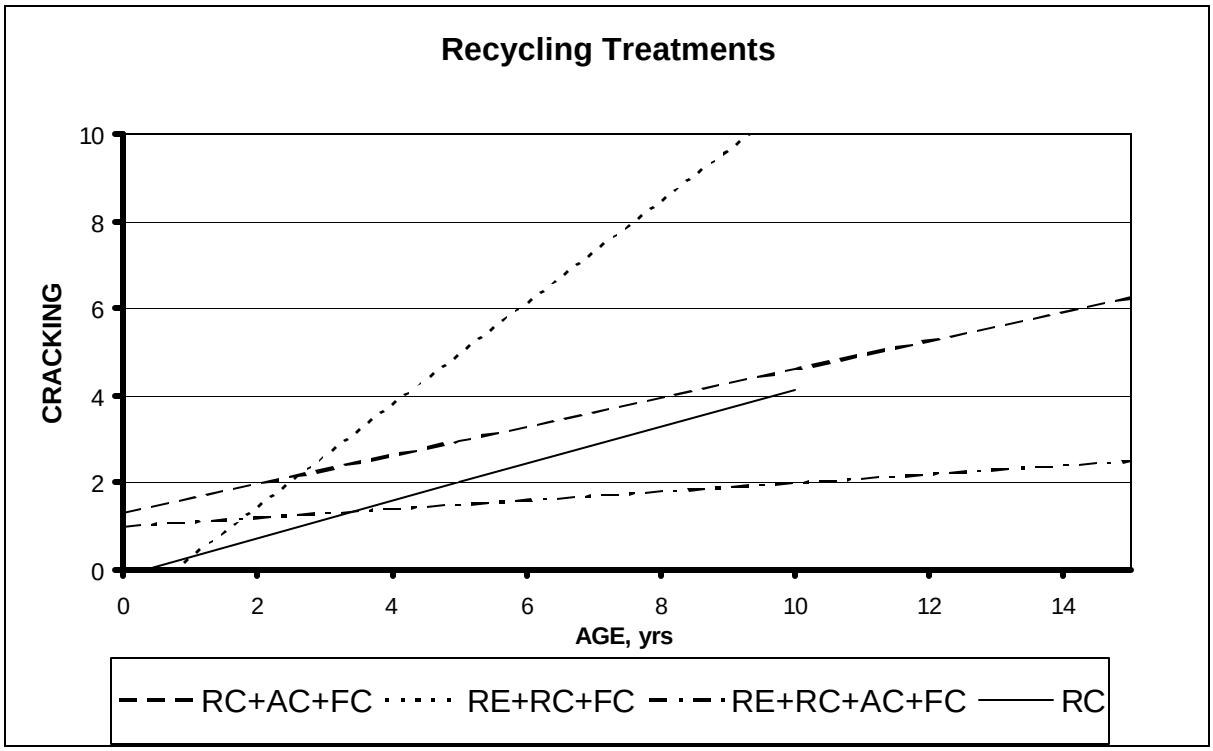
Cracking of Functional Treatments				
Code	**Description**	**Nr of Records**	**Max Age**	**Significance**
FC	Asph Concr Frict Cours (ACFC)	7291	15	Yes
REFC	Layer removal + ACFC	693	15	Yes
FR	Rubberized ACFC	298	15	No
REFR	Layer removal + Rubber ACFC	195	10	Abnormal slope
SC	Seal Coat	17145	15	Yes

Figure 7.15 Cracking of Functional Treatments

Cracking of Recycled Treatments

The following trends can be observed for this group of treatments shown in Figure 7.16.

- RC and RC+AC+FC are fairly similar, RE+RC+AC+FC has a lower rate of deterioration, and RE+RC+FC has a much higher rate of deterioration than the others.



Cracking of Recycling Treatments				
Code	**Description**	**Nr of Records**	**Max Age**	**Significance**
RC	In-place Recycling	70	10	Yes
RCACFC	Recycling+ACFC	3307	15	Yes
RERCFC	Remove, (plant) Recycle+ACFC	363	10	Yes
RERCACFC	Remove, (plant) Recycle+AC overlay+ ACFC	1039	15	Yes

Figure 7.16 Recycling Treatments

Comparison of Treatments for Cracking

The comparison between the various materials and treatments has been carried out by determining the time it takes for each model to reach a medium cracking level of 10% (Table 7.10). Those models with an abnormal slope had to be left out in this comparison; they will be ranked separately on the basis of their initial cracking levels (Table 7.11).

Table 7.10 Scores for Models with Significant Slope (higher number means higher score)

Code	Description	Nr. of Records	Max Age	Significance	Score
RERCACFC	Remove, (plant) Recycle + AC overlay + ACFC	1039	15	Yes	90
ACFL	AC + Flush Coat	756	15	Yes	82
REACSC	Layer removal+AC + Seal Coat	306	15	Yes	39
FR	Rubberized ACFC	298	15	No	38
REFC	Layer removal + ACFC	693	15	Yes	35
AC	Asphalt Concrete	1237	15	Yes	35
REACFC	Layer removal + AC + ACFC	3223	15	Yes	32
ACFC	Asphalt Concrete + ACFC	8364	15	Yes	31
RCACFC	Recycling + ACFC	3307	15	Yes	26
ACSC	AC + Seal Coat	7210	15	Yes	26
RC	In-place Recycling	70	10	Yes	24
REAC	Layer removal + AC	554	10	Yes	14
SC	Seal Coat	17145	15	Yes	10
RERCFC	Remove, (plant) Recycle+ACFC	363	10	Yes	8

Table 7.11 Score for Models with Abnormal Slope (higher number means lower score)

Code	Description	Nr. of Records	Max Age	Significance	Score
REAR	Layer removal + Rubber AC	39	5	Abnormal Slope	0.7
REFR	Layer removal + Rubber ACFC	195	10	Abnormal Slope	1.6
REACFR	Layer removal+AC+Rubber ACFC	407	5	Abnormal Slope	3.4

7.3.6 Rutting Data

For the evaluation and comparison of rutting the treatments were divided into three groups as shown in Table 7.12. These three groups cover rutting data for various structural overlay materials and techniques and recycling treatments for all pavements between 1981 and 1994.

A similar format has been used as for roughness data.

Table 7.12 Groups of Overlays and Treatments to Evaluate Rutting

STRUCTURAL OVERLAYS INCL. FRICTION COURSES AND RUBBER		**Recycling Treatments**	
AC	Asphalt Concrete	RC	In-place Recycling
ACFC	Asph. Concr.+ AC Frict. Course	RCACFC	Recycling + ACFC
REAC	Layer removal + AC	RERCACFC	Remove, (plant) Recycle + AC overlay + ACFC
REAR	Layer removal + Rubber AC		
LAYER REMOVAL + FRICTION COURSES WITH/WITHOUT STRUCTURAL OVERLAYS & RUBBER			
REFC	Layer removal + ACFC		
REFR	Layer removal + Rubber ACFC		
REACFC	Layer rem.+AC+ACFrict.Course		
REACFR	L.removal +AC+ RubberACFC		

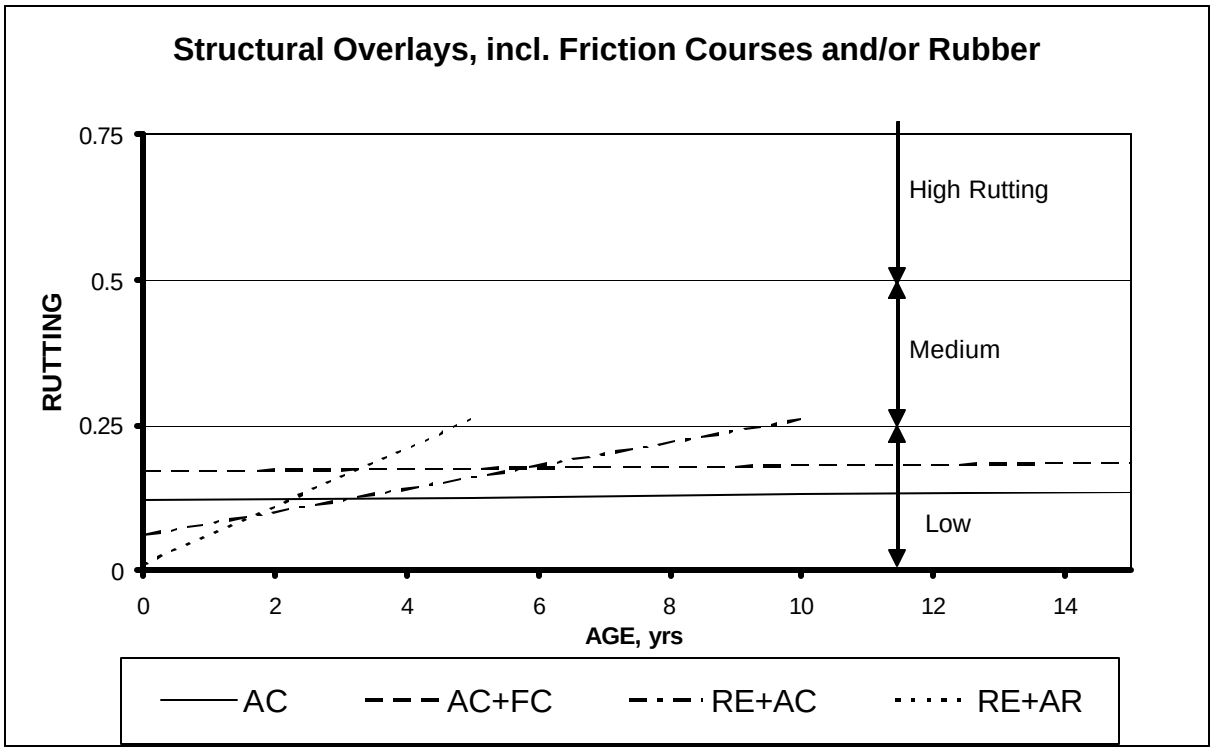
ADOT's criteria for rutting are:

Low	< 0.25 inch
Medium	0.25 - 0.5 inch
High	> 0.5 inch

Rutting Data For Overlays, Include Friction Courses and/or Rubber

The following trends can be observed for this group of treatments as shown in Figure 7.17.

- AC and AC+FC show little rutting and a very low rate of deterioration.
- RE+AC and RE+AR show much higher rates of deterioration, but the number of data points and the age for RE+AR are both low.



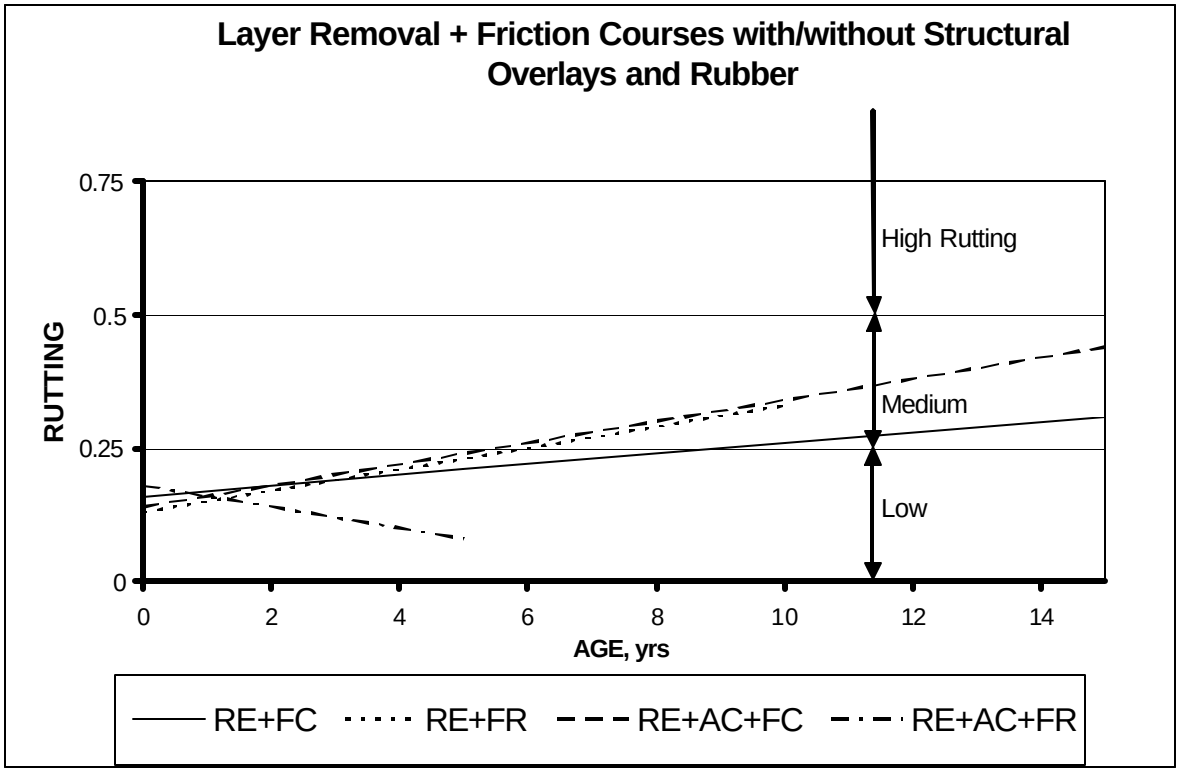
Rutting Data for Structural Overlays incl. Frict. Courses and/or Rubber				
Code	**Description**	**Nr of Records**	**Max Age**	**Significance**
AC	Asphalt Concrete	541	15	Yes
ACFC	Asph. Concr.+ AC Frict. Course	3842	15	Yes
REAC	Layer removal + AC	385	10	Yes
REAR	Layer removal + Rubber AC	39	5	Yes

Figure 7.17 Structural Overlays including Friction Courses and/or Rubber

Rutting for Layer Removal + Friction Courses with/without Structural Overlays and Rubber

The following trends can be observed for this group of treatments as shown in Figure 7.18.

- RE+FR gives a slightly higher rate of deterioration than RE+FC.
- Compared with RE+FC, RE+AC+FC gives a lower rutting resistance.
- RE+AC+FR has an abnormal slope, and an age less than five years.



Rutting Data for Layer Removal + Friction Courses with/without Structural Overlays & Rubber				
Code	**Description**	**Nr of Records**	**Max Age**	**Significance**
REFC	Layer removal + ACFC	558	15	Yes
REFR	Layer removal + Rubber ACFC	171	10	Yes
REACFC	Layer removal + AC + ACFrict.Course	2742	15	Yes
REACFR	Layerremoval + AC+ RubberACFC	407	5	Abnormal Slope

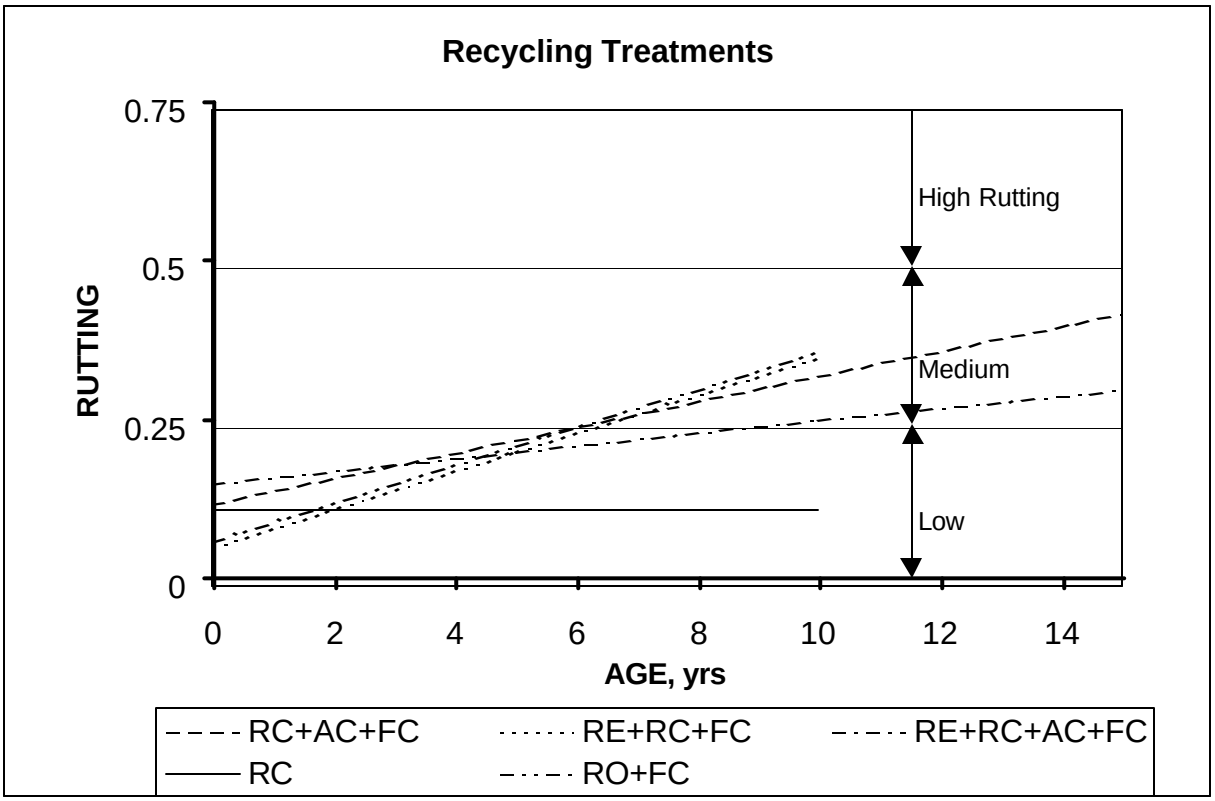
Figure 7.18 Layer Removal + Friction Courses with/without Structural Overlays and Rubber

Rutting Data for Recycling Treatments

The following trends can be observed for this group of treatments as shown in Figure 7.19.

- RC has a very low rate of deterioration, but the model is not significant.
- RC+AC+FC has a slightly higher rate of deterioration than RE+RC+AC+FC, which shows the influence of the method of recycling.
- RE+RC+FC has a higher rate of deterioration than the other recycling methods.

Rutting Data for Recycling Treatments



Code	Description	Nr of Records	Max Age	Significance
RC	In-place Recycling	39	10	No
RCACFC	Recycling + ACFC	2203	15	Yes
RERCFC	Remove, (plant) Recycle + ACFC	323	10	Yes
RERCACFC	Remove, (plant) Recycle + AC overlay + ACFC	968	15	Yes

Figure 7.19 Recycling Treatments

Comparison of Treatments for Rutting

The comparison between the various materials and treatments has been carried out by determining the time it takes for each model to reach a medium rutting level of 0.25 (Table 7.13). Those models with an abnormal slope had to be left out in this comparison; they will be ranked separately on the basis of their initial rutting levels (Table 7.14).

Table 7. 13 Scores for Models with Significant Slope (higher number means higher score)

Code	Description	Nr. of Records	Max Age	Significance	Score
AC	Asphalt Concrete	541	15	Yes	13
REAC	Layer removal + AC	385	10	Yes	10
RERCACFC	Remove, (plant) Recycle + AC overlay + ACFC	968	15	Yes	9
REFC	Layer removal + ACFC	558	15	Yes	9
ACFC	AC + AC Friction Course	3842	15	Yes	8
REACFC	Layer removal + AC + ACFC	2742	15	Yes	6
RCACFC	Recycling + AC + ACFC	2203	15	Yes	6
RERCFC	Remove, (plant) Recycle + ACFC	323	10	Yes	6
REFR	Layer removal + Rubber ACFC	171	10	Yes	6
REAR	Layer removal + Rubber AC	39	5	Yes	5

Table 7.14 Score for Models with Abnormal Slope (higher number means lower score)

Code	Description	Nr. of Records	Max Age	Significance	Score coeff a
RC	In-place Recycling	39	10	Abnormal slope	0.12
REACFR	Layer removal + AC + Rubber ACFC	407	5	Abnormal slope	0.18

7.3.7 Conclusions and Overall Comparisons

The effectiveness of various surface treatments, overlay materials, and other rehabilitation techniques have all been ranked with respect to roughness, cracking, and rutting. For a large group of models this was done on the basis of the time it would take for each rehabilitation treatment to reach a certain performance level, such as "tolerable roughness" or "medium cracking".

For some models this approach did not work, because the slope of the performance curve was abnormal, or almost zero, so that the above mentioned performance level would not be reached. In those cases another criterion was used. This different methodology makes it difficult to compare treatments in this group with those in the first one. However, having an abnormal slope does not mean that the treatment does not perform well. A steep abnormal slope is difficult to explain, but a zero or near zero slope can be interpreted as a very low level of deterioration, which is obviously very beneficial.

In order to get some overview of the treatments that performed best, the following methodology has been followed:

- For each performance indicator the total number of treatments considered was divided by four, this resulted in seven treatments for roughness and for cracking, four for rutting and three for friction. For these treatments the best performers were selected from the first group (the group with a criterion to reach a certain performance level). Only significant models were selected.

- For each performance indicator a selection was made from the second group of treatments with abnormal or zero slope on the basis of:

    - Initial performance (coeff. a): below critical level used for first group.
    - The slope: below 0.7 in/mi/yr
    - The number of records: above 30
    - The significance of the relationship and the solidity of the plot of data points in Visual Modeler

The selected treatments were added to those in the first group.

This methodology led to the following lists of top performers:

Good performers to prevent roughness (no ranking):

- AC — Asphalt concrete (AC)
- ACFC — Asphalt concrete + AC friction course
- FC — Asphalt concrete friction course (ACFC)
- REFC — Layer removal + ACFC
- REFR — Layer removal + rubber ACFC
- RCACFC — Recycling + AC overlay + ACFC
- RERCACFC — Remove, (plant) recycle + AC overlay+ ACFC

Good performers to minimize cracking progression (no ranking):

- AC — Asphalt concrete (AC)
- ACFL — Asphalt concrete + flush coat
- FR — Rubberized asphalt concrete friction course
- REAR — Layer removal + rubber asphalt concrete
- REFC — Layer removal + ACFC
- REFR — Layer removal + rubber ACFC
- REACSC — Layer removal + asphalt concrete + seal coat
- RERCACFC — Remove, (plant) recycle + AC overlay + ACFC

Good performers to minimize rutting progression (no ranking):

- AC Asphalt concrete (AC)
- REAC Layer removal + asphalt concrete
- REFC Layer removal + ACFC
- REACFR Layer removal + AC + rubberized ACFC
- RERCACFC Remove, (plant) recycle + AC overlay + ACFC

Please note that no ranking has been given for these top-performers for the following two reasons:

- Each list is composed of populations from two different groups, one where a selection was made on the basis of "life", and one where totally different criteria were applied.
- The individual linear regressions have in general a high level of significance, but the difference among the lines has not been tested.

Consequently it was not possible to rank the various treatments in quantifiable terms, but the trends that were found are certainly valid.

It is not surprising to see that in general, different treatments perform better for different performance indicators. This is most obvious for friction. In this respect one is reminded of the fact that not all treatments were considered for all performance indicators. For example, there seemed no point in testing the rutting performance of a flush coat or any other thin surface treatment. An overall comparison is therefore difficult to make. Still some observations are worth noting:

Good performers for all four performance indicators roughness, friction, cracking and rutting:

- Asphalt concrete (AC), and/or layer removal + asphalt concrete (REAC)
- Rubberized asphalt concrete friction course (FR) and/or layer removal + FR (REFR)

Good performer for roughness, friction and cracking:

- Asphalt concrete + Flush coat (ACFL). See comments at end of this section

Good performers for roughness, cracking, and rutting:

- Asphalt concrete friction course (FC) and/or layer removal + FC (REFC)
- Layer removal + recycle + asphalt concrete + FC (RERCACFC)

This overview should give guidance for practical application, but it is recommended that the final selection be made on a case-by-case basis, including particular requirements or economic constraints in the decision process.

It should also be stressed that the findings are based on the performance of a number of layer combinations as they occurred. Take flush coats as an example. This material, that is considered to be a relatively low-cost maintenance treatment to extend pavement surface life, came out surprisingly well in the comparisons. That does not mean to say that this material should be recommended to reduce roughness or cracking, or to enhance friction for badly deteriorated roads. The reason that flush coats showed such good performance was probably because the underlying pavement structure was sound (see also comments made towards end of section "Approach"). At the time that the flush coat was selected the pavement must have been healthy enough to receive a relatively cheap treatment in order to extend its useful life. Thus, flush coats, applied at an appropriate time in a pavement's life cycle can be a cost effective treatment to preserve a sound pavement.

The trends shown should give guidance for practical application, but it is recommended that the final selection be made on a case-by-case basis, including particular requirements or economic constraints in the decision process.

As explained earlier these findings are not based on the outcome of a controlled design experiment, but the data do cover many miles of Arizona roadway, and the reported trends are valuable.

7.4 USE OF PMIS TO SOLVE MATERIALS PROBLEMS FOR PCC PAVEMENTS IN TEXAS

7.4.1 Introduction

Since 1974 visual condition survey data has been collected on portland cement concrete pavements in the state of Texas for the TxDOT PMIS database and for the Rigid Pavement Database. Beginning in September 1992 it was observed, while processing the data from this database, that some of the pavements seemed to suffer considerably more distress than other seemingly similar pavements. Basically there appeared to be two general groupings of pavements, some that performed better than average and another group that performed poorer than average. There was at first no clear indication as to why this was true but subsequent research based on the findings of the PMS analysis solved the problem. [McCullough 95, Dossey 94, McCullough 93]

7.4.2 Aggregates for Continuously Reinforced Concrete Pavements

Included in Texas' 73,000 centerline miles (117,482 km) of highways are both rigid pavements (portland cement concrete, or PCC) and flexible pavements (asphalt concrete). Most of these pavements were constructed using crushed limestone (LS) and/or siliceous river gravel (SRG) as coarse aggregates. It is commonly known that the type of coarse aggregate used in construction significantly affects the performance of pavement structures (since 60 to 80 percent of the pavement volume is some type of coarse aggregate). Thus, it may be necessary in evaluating the performance of both rigid and flexible pavements to include the coarse aggregate type as a variable. By determining the different physical and chemical characteristics of course aggregates in addition to other characteristics that affect pavement performance it might be possible to make pavement design adjustments.

Prior to 1993 the design and construction of asphalt concrete (AC) and portland cement concrete (PCC) pavement in Texas did not take into account the variation in coarse aggregate material properties. Generally, during the bidding process, a contractor selected the aggregate type based on competitive prices from various aggregate suppliers. The contractor then constructed the required pavement with the coarse aggregate of his own choosing, even though prior field tests may have indicated that pavement performance will vary significantly depending on the physical properties of the aggregate.

For portland cement concrete pavements, the Texas design process was based on the premise that concrete volume affects the occurrence of transverse cracks or joints that develop along continuously reinforced concrete pavements (CRCP) and that are formed or sawed at prescribed spacings for jointed concrete pavements (JCP). The spacing and width of cracks in CRCP are related to the longitudinal steel placed in the concrete slab. Thus, the crack pattern of CRCP is an important physical performance characteristic of the pavement.

Although not reflected in 1990, Texas concrete specifications, the physical properties of the coarse aggregate have a significant effect on the crack pattern developed in CRCP. The chemical properties serve as indicators of physical performance. The principal physical properties of concrete that vary with coarse aggregate type include modulus of elasticity, the coefficient of construction and expansion, strength, and shrinkage. All of these have a direct effect on crack spacing, crack width, and corresponding pavement performance.

During the late 1970s and early 1980s, the CTR conducted a project entitled "Evaluation of the Performance of Texas Pavements Made with Different Coarse Aggregates" for the State Department of Highways and Public Transportation (presently the Texas Department of Transportation). One phase of the study dealt with pavement management condition surveys for the Department that were used to establish criteria for selecting projects for rehabilitation. In addition, condition survey data were used to develop numerous improvements in construction

procedures and in the design process. One of the major observations that came out of the study was that, generally, CRCPs placed in Texas with limestone coarse aggregates had 20% to 50% longer service lives than CRCPs placed with siliceous river gravel coarse aggregates [Hankins 91]. As a result of these studies, several districts began to add a note to construction plans requiring that crushed limestone be used as the coarse aggregate. Contractors without much question during Texas' economic boom of the 1980s accepted this requirement. At that time the suppliers were marketing aggregates of all types as rapidly as they could produce them.

A second major contribution of the study was the development of the CRCP-1 computer program that provided designers with an analysis method for using concrete properties, climatic conditions, and subbase friction conditions to predict crack spacing, steel stress, and crack width as a function of time. This program permits the user to analyze the performance over time of any given design.

As building slowed down in Texas, especially in the Houston area, concern developed regarding the requirement of limestone aggregate. Consequently, the department requested that CTR use the CRCP program to develop an acceptable CRCP design that incorporated siliceous river gravel. This information was developed and subsequently, a proposed standard design - CRCP (B-89B) - was executed by the Highway Design Division of TxDOT.

The purpose of the analysis was to develop designs that produced similar crack spacings for both aggregate types. Unfortunately, because SRG produces more cracks than limestone because of SRG's higher thermal coefficients and modulus of elasticity, the resulting design standard required more steel for limestone than for SRG. Of course, this presented a problem to contractors, since those who chose to use the limestone aggregate that was providing better performance (from the standpoint of cracking) were in effect being penalized by the requirement that also used more steel. TxDOT initiated Project 1233, "Evaluation of the Performance of Texas Pavements Made with Different Coarse Aggregates," as a CTR and Texas Transportation Institute (TTI) joint study of this problem, which is summarized below. [Dossey 94, McCullough 95]

7.4.3 Study of Early Age Behavior

An early report from this study documents the short-term performance of several continuously reinforced concrete test pavements [Suh 92]. The Texas PMIS highlighted the need for this study. The report focuses on: 1) the importance of the heat of hydration in connection with early-age behavior of CRCP; 2) the effect on early-age cracking of the construction season and the time of placement during the day; 3) the detrimental characteristics of the early-age cracks in terms of their shapes and widths; 4) the effect of coarse aggregate type on cracking; 5) the factors affecting crack width; 6) the determination of setting temperature, which is used as a reference temperature in the calculation of temperature-induced stresses; and 7) the correlation between the shrinkage of concrete pavement in the field and that of lab-cured cylinders. Based on these observations and findings, the study recommends a method of simulating the early-age observations in the CRCP analysis computer program.

Using distress curves produced in the study and shown in Figure 7.20, researchers calibrated the failure prediction model in a new CRCP-7 computer program. The report offers the following conclusions and recommendations, drawn from the short-term monitoring:

1. For all the paired test sections, there were significant differences in both crack spacing and crack width between the siliceous river gravel (SRG) sections and the limestone (LS) sections. The LS sections, without exception, experienced fewer cracks and larger crack spacings than the SRG sections. The LS sections also experienced smaller crack widths than the SRG sections, even though the crack spacings were larger.

2. Significant differences were found between the paired (equivalent) limestone and SRG sections, with the limestone sections experiencing both larger crack spacings and smaller crack widths. The SRG test sections experienced early minor punchouts. The crack widths measured in the test sections constructed in the summer, though wider than those measured in the equivalent winter test sections, are still well below the AASHTO limits.

3. Generally, with the winter projects, crack spacing and crack width decreased as the percent steel increased. A difference in bond area, achieved by the use of different bar sizes (No. 6 and No. 7 bars) was tested, but at the age of one month had not shown a significant difference in cracking.

4. The location of new cracks formed between two adjacent old cracks in CRCP seemed to depend on the distance between the two old cracks (longitudinal length of the slab segment). When the slab segment was three to four feet long, a new crack occurred near the middle of the slab segment. Longer slab segments showed greater scattering of the new cracks near the middle of the slab segment. For slab segments longer than eight feet, the locations of new cracks were randomly spaced.

5. Factors that significantly affect crack width were construction season, coarse aggregate type, amount of steel, and time of crack occurrence. Hot weather placement showed much wider cracks at a given slab surface temperature condition than cool weather placement. The use of SRG aggregates resulted in wider cracks than the use of LS aggregates, and the difference was larger at lower temperatures. The greater the amount of longitudinal steel, the narrower the crack width. Cracks occurring during the first three days of construction were significantly wider than those that occurred later.

6. Shrinkage of a concrete cylinder cured in the laboratory at 75ºF and 40% relative humidity, without removing the cylinder mold, sufficiently represented the field shrinkage of the concrete slab.

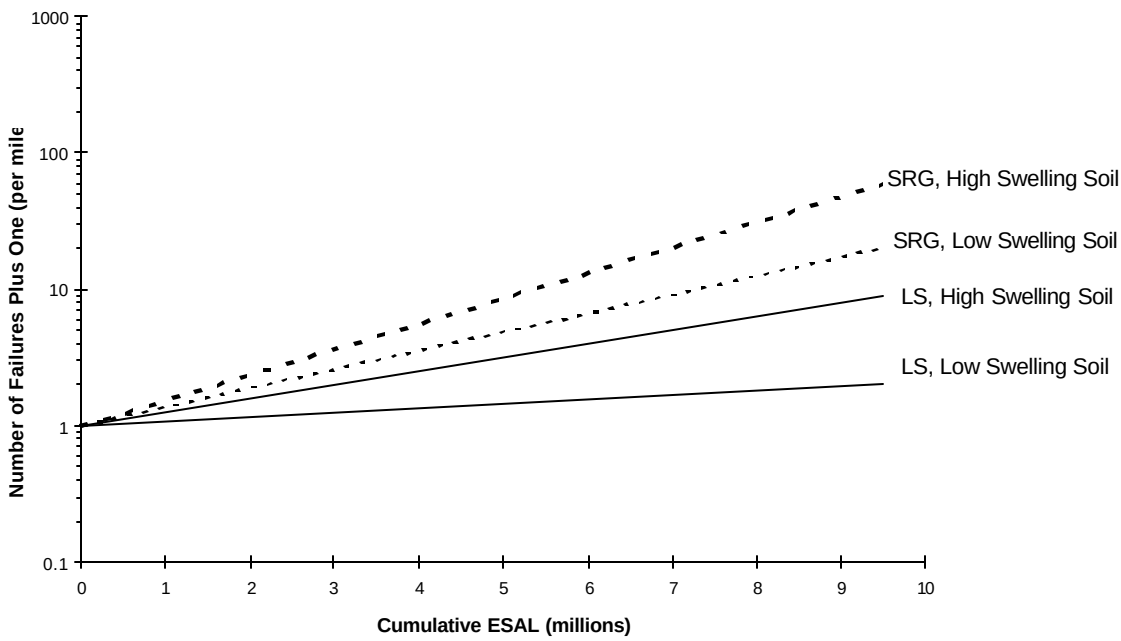


Figure 7. 20 Mean Failure Curves for Different Coarse Aggregate Types and Subbase Swelling Conditions

7.4.4 Summary of Field Test Section Results

The performance of CRCP constructed with siliceous river gravel (SRG) coarse aggregate differs significantly from that of CRCP constructed with limestone (LS) coarse aggregate. Specifically, for the same steel design, pavements containing siliceous river gravel experienced substantially more distress (e.g., cracks, spalling, and punchouts) than those containing limestone.

In creating this new design standard, researchers at the CTR at the University of Texas at Austin first analyzed the properties of concrete samples constructed with both SRG and LS coarse aggregates and then entered the results in the CRCP-4 computer program as input values to predict the performance of pavements containing these materials. For various slab thicknesses and steel designs, the program predicted the performance of each pavement in terms of crack spacing, crack width, and steel stress. Using the CRCP design criteria, an optimum longitudinal steel design can be selected for each combination of slab thickness and coarse aggregate type.

7.4.5 Engineering Application

Based on the results of this early analysis, the TxDOT developed new design standards for using SRG or LS aggregates. These standards include different coarse aggregate types, for a given slab thickness, different bar spacings were used with same sized bars. The major feature of the new design standard was the use of different amounts of longitudinal steel for different coarse aggregate types in a manner such that similar crack spacings for the two aggregate types would be expected within the limiting design criteria. Since the major distresses of CRCP (i.e., punchouts and spalling) have a strong correlation with crack spacing, the new design standard sought to minimize the distress by controlling the crack spacing and thus the new design for SRG concrete specified less longitudinal steel than specified for LS concrete. This is due to the fact that higher percent steel results in lower crack width and lower crack spacing. Table 7.15 summarizes these relationships.

Table 7.15 Summary of Aggregate - Steel - Crack Relationships

Limestone	Siliceous River Gravel
Fewer Cracks	More Cracks
Larger Crack Spacing	Smaller Crack Spacing
Smaller Crack Width	Larger Crack Width
Design Calls for Higher % Steel	Design Calls for Lower % Steel

7.4.6 Aggregate Blending

Given the variety of aggregates available for concrete pavement construction, TxDOT required a quick method for selecting aggregate to meet the needs of a particular project. By using a categorization technique for coarse aggregates, a pavement designer could select an aggregate quickly by looking at aggregate properties and characteristics that are most important to the project.

Concrete behavior is predictable for concrete made with a single type of aggregate. However, a single aggregate type does not always provide the desired properties for a project. An alternative is a blending of two aggregate types to yield more desirable properties. In 1992, there was no method for predicting pavement properties resulting from controlled blending of two different aggregate types. The relationship between pure aggregate concrete and blended aggregate concrete was thought to be linear, as shown in Figure 7.21, for a number of concrete properties. This study verified that concept through laboratory research and field analysis of test pavements.

7.4.7 Summary of Aggregate Blending Results

Early TxDOT efforts to study the effects of aggregates on portland cement concrete were limited to unblended, or single aggregate, concrete. The results showed that aggregate type had a strong effect on the performance of PCC pavements. Of the pavements tested, those constructed of siliceous river gravel performed worse than those pavements constructed with crushed limestone (especially in situations where high temperature differentials existed during paving). The results were attributed to the gravel's high coefficient of thermal expansion. However, Study

1244 showed that portland cement concrete made with a blend of siliceous river gravel and crushed limestone can mitigate some of the gravel's adverse properties.

For each of the four properties studied in this project (i.e., compressive strength, elastic modulus, splitting tensile strength, and coefficient of thermal exp ansion), a linear mathematical model was compared with a regression model. For each, the linear model used follows:

$$f(blend) = f(SRG) + [\{f(LS)\} \times (\%LS/100)]$$

where

f(blend)	=	property value of a mix with blended aggregates,
f(SRG)	=	property value of a mix with 100% siliceous river gravel,
f(LS)	=	property value of a mix with 100% crushed limestone, and
%LS	=	percent crushed limestone

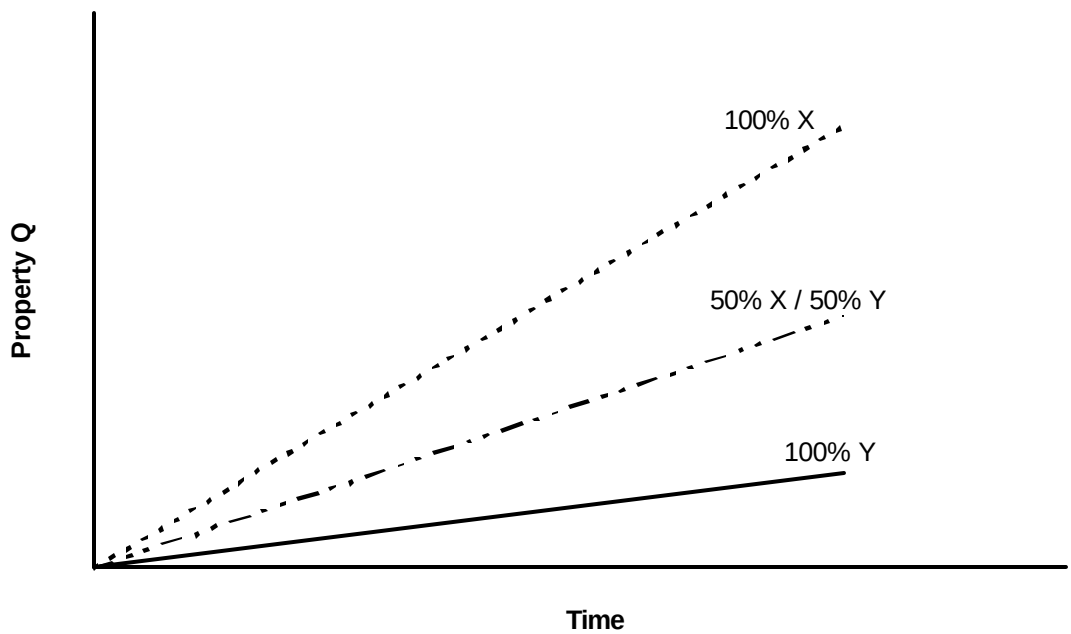


Figure 7.21 Conceptual Relationship of Pure and Blended Aggregate Concretes

Although there was significant experimental scatter, the means of all four regression equations followed the linear hypothesis equations. The r^2 (coefficient of determination) values varied from 92 percent for the coefficient of thermal expansion, to 99 percent for compressive strength.

Though the experiment was limited to four aggregate samples and one curing method, the experimental results demonstrate that properties of blended limestone and gravel aggregate vary approximately linearly in proportion to the blending ration of the aggregate types used.

7.4.8 Recommendations from the Study

The study recommended that aggregate blending be used to migrate undesirable concrete properties, such as high thermal coefficient of expansion. In this way, pavement engineers can use coarse aggregates that would otherwise be unsuitable under certain paving conditions (e.g., high daily temperature variation). Also, equal performance can

be obtained from pavements constructed with very different coarse aggregates. From the research conducted in this study, the following recommendations can be made:

- Blending of coarse aggregates should be performed during concrete mixing to ensure even blending;
- The linear weighted averaging technique presented here should be developed as a computer tool to assist designers in previewing the performance of a blended aggregate mix design prior to laboratory concrete testing;
- Additional sources for both crushed limestone and siliceous river gravel should be tested to further expand the database for the CRCP and CHEM programs, ensuring accurate predications of properties for unknown aggregate and aggregate blends; and
- Further testing of aggregates other then limestone and siliceous river gravel should be conducted to expand and refine the aggregate property prediction equations and to confirm the linearity of other aggregate types.

With the refinement of the blend models from further research, properties can be accurately estimated for any ration of crushed limestone and siliceous river gravel where properties of the pure aggregates are known. Using estimates from the newly developed models, designers will be able to go one step further by using currently established design tool programs to determine how the predicted blend will affect pavement properties, thus producing equal pavement performance.

7.4.9 Engineering Benefits

This study illustrates very graphically the engineering benefits that can derive from a pavement management database. It also illustrates that these benefits often require additional work to develop, after the database shows the aspects of the problem. Only through proper analysis and evaluation does the data produce useful engineering results.

To summarize this case study, when the pavement engineering staff examined the PMS database, it clearly indicated a diversity of performance among the existing rigid pavements. This diversity pointed out missing data elements in the database because at that time the database had no element for aggregate type. However, a further search of the database showed that the county or local area in which the pavement was located seemed to correlate highly with the observed damage. This then led to the understanding that generally a particular aggregate type is common within a particular county or area of the state. This was such an important finding that a major research project followed (which has been discussed herein), to determine the cause of this problem and to derive possible solutions.

The cause was found to be the type of aggregate used in the portland cement concrete mixture. The solution was found to be the use of limestone type aggregates or a blending of aggregates to provide improved properties that approximate the limestone aggregate more closely than the rounded river gravel aggregate.

This case study also points out clearly the fact that from time to time additional data elements will need to be added to a pavement management database. In this case it shows the need to go back and add coarse aggregate type as a factor in the inventory data and materials type records of the Texas PMIS database.

7.5 SMOOTHNESS SPECIFICATIONS AS PART OF CONSTRUCTION QUALITY CONTROL IN KANSAS

7.5.1 Overview of the Kansas DOT'S PMIS

The Kansas pavement management system is based on Markovian probability matrices. It is divided into two parts, a NOS or network optimization system and POS or project optimization system. The network optimization system functions using three pavement distress variables to derive an optimum list based on linear programming. The POS system is not totally developed at this time. Its prediction models are presently being revised under contract with Dr. Ray Moore, from Kansas University and Professor Najjar from Kansas State University.

The KDOT PMS uses a database called the PMIS or Pavement Management Information System, which basically contains pavement type, location identification, and historical distress information. The PMIS has over 10,000 miles of the highway system that is maintained by state forces. The distribution of this mileage by pavement type is as follows:

Portland cement concrete	700 miles
Composite	1,100 miles
Full design bituminous	2,800 miles
Partial design bituminous	5,400 miles

The reference system is county/route/milepost. For distress surveying the highway system is divided into approximately one-mile segments to facilitate modeling using the Markov technique. The system also generally obtains and uses information from the inventory database which is termed (CANSYS), including inventory information, structural data, traffic data, etc. by section location. Mr. Walrafen, the pavement management engineer, mentioned that he has little problem connecting the two separate databases but did indicate on several small examples that there are the normal problems of location identification continuity.

In Kansas, the pavement management activity functions as part of the Geotechnical Unit. There is a Pavement Management Section, a Pavement Design and Soils Section and other subgroups.

Generally, the KDOT Pavement program statewide is divided into two parts. Part A covers large projects which are often selected on a "worst first" basis, including a formula for traffic, age, funding, etc. Part B covers the rehabilitation of about 1,200 miles per year using state funding. This averages a rehabilitation of about 1½ inch thickness over the selected 1,200 miles. This is the portion that is dealt with by NOS.

Approximately 1,000 miles of pavement is tested per year with FWD, related to design for various construction programs. Approximately 100-200 miles per year is reviewed by a peer group as part of the large project selection function. This peer group includes five engineers; one from the district involved (there are six districts), one each from design, construction, and materials, and a fifth one from the central administration.

There was the original intention to upgrade the Project Optimization System (POS) using the pavement management data; however that has not yet happened. The current pavement condition survey methods are carried out with three randomly selected 100-foot long subsections. This means that on a year-to-year basis the 100-foot subsections are randomly selected again and that there is no continuity in the locations of the sections from year-to-year. This is of course, a good way to insure unbiased estimates for pavement management; however, it makes it very difficult to get continuity in data for model building. There is almost no repetitive data from which to select a database for use in upgrading the POS. This effectively points out one of the problems that is sometimes associated with trying to carry out multiple activities with the same database. In order to ensure a completely random data set each year for the network level optimization system, section condition surveys are re-randomized each year. This however, does not create a database that can be used to upgrade the performance models in the POS. It would be superior for POS to select a database, which has observations of the same condition subsections annually. This points out the need, once

again, to have a research element in the pavement management information system or the pavement management database.

Once a project is selected for funding it is reviewed by the pavement designer at the state level. The Pavement Manager and his team consider distress from the PMS and other factors, but no formal decision tree is used and they do not have a formal project level decision program. Each district is given an annual report from the PMS, which they use for various purposes. The team feels that the district personnel are very supportive and positive about the Kansas PMS.

Recently, Kansas updated their pavement management system by changing one of the three factors in the NOS optimization for flexible pavements. The original optimization included fatigue cracking and block cracking, transverse cracking and roughness for flexible pavements. However, over the past ten years they have noted that fatigue cracking had very little impact on the pavement decisions resulting from their pavement management system. On the other hand, their database showed that rutting had major impact on their pavements and was a major damage factor. Currently the pavement management system uses rutting, transverse cracking and roughness in their overall optimization scheme for flexible pavements. Each of these is broken into three severity levels. For example Table 7.16 shows these levels for roughness and rutting. The pavement management team feels that this has greatly improved the functionality of their PMS. Table 7.17 shows the distribution of rutting mileage by district for 1989.

Table7. 16 Severity Levels

Levels	Roughness	Rut depth
1	< 1.66 m/km	< 13 mm (½")
2	1.66 to 2.59 m/km	13 to 25 mm (½ to 1")
3	> 2.59 m/km	> 25 mm (1")

Table 7.17 Distribution of Miles with Rutting by District

District	Miles with >1/2" Rutting	% of Rutted Mileage/District
1	296	24
2	65	5
3	184	15
4	403	32
5	234	19
6	56	5

Each year the pavement management section produces a NOS Condition Survey Report, which is a primary output of the pavement management process. A sample report is reproduced as Attachment 7A. [KDOT 96]

7.5.2 As-Constructed Smoothness Specifications

Introduction

A great deal of background on the importance of constructing pavements in an initially smooth condition has been studied over the last 20 years. Hudson and Haas have illustrated that with parallel pavement performance curves, a pavement with higher initial roughness (lower serviceability) will become unserviceable before a pavement that was initially constructed smoother [Haas 94]. The rate of roughness increase is related to both pavement structure and initial roughness level. Kansas studied this problem using its PMS and showed the smoother a pavement is in the beginning, the longer it will remain at an acceptable roughness level. As a result of this evaluation, Kansas changed their roughness specification to remove the so-called "blanking band" for the profilograph roughness specification. This change in specification has resulted in smoother, as-constructed pavements and according to an evaluation of the PMIS database is producing pavements with longer lives within the Kansas highway system. Hossain and Parcells have published the paper "Smoothness Control in Asphalt Pavement Construction Development of Specifications, Implementation and Results" [Hossain 95]. It is reproduced below.

Surface smoothness on newly constructed pavement is a major concern of the highway industry. This smoothness, or riding comfort, is an indication of the quality of the newly constructed pavements since it affects road users directly. Smoothness specifications for asphalt concrete (AC) pavements now in effect in Kansas have evolved over the past few years through a number of revisions. Pavement profiles with short wavelengths and smaller amplitudes

than the industry-accepted 5.1-mm (0.2 in.) can harm the ride quality of pavements. This experience has led the Kansas Department of Transportation to diminate the blanking band width in the profilograph trace reduction process. The implementation of this zero, or null, blanking band was successful and has resulted in smoother asphalt pavements in Kansas. The currently used specifications for AC pavements have been based on the consideration of a number of factors related to the construction of and the measurement of smoothness on AC pavement. The incentive payment amounts have been calculated to make these compatible with the incentive payments for concrete pavement. The results show that contractors can achieve these smoothness specifications and the number of sections in the bonus range indicates that the incentive payments encouraged better-quality paving. These results should have a positive impact on AC paving in Kansas. Overall, an increasing number if miles of pavements with lower profile index values are being constructed since the implementation of smoothness specifications for AC pavements.

Pavement smoothness and roughness can be described by the magnitude of profile irregularities and their distribution over the measurement interval. The road surface smoothness on newly constructed pavement is a major concern of the highway industry. This smoothness, or riding comfort, is a measure of the quality of the newly constructed pavements since it affects the road users directly. According to Hudson [Hudson 81], the primary purpose for smoothness measurement is to maintain construction quality control.

It is accepted that there is a growing interest in the highway industry for attaining smoother and smoother pavement surfaces. Results for a 1992 NCHRP study show that of the 22 states reporting, 91 percent used smoothness criteria on new pavement construction [Scofield 92]. In 1990 NCHRP reported that of 36 states reviewed, 80 percent utilized smoothness criteria on new pavement construction [Woodstrom 90]. The increasing trend in the use of ride quality specifications is also evidenced by the 1992 study, in which 21 states out of 25 queried believe that there will also be a future increase in ride quality requirements. A 1987 AASHTO survey showed that 53 percent of the states using profilographs for acceptance of concrete pavements used incentive and disincentive specifications [AASHTO 88]. The incentive and disincentive values in smoothness specifications typically ranged from 1 to 5 percent of the bid item price, with 31 percent of the states reporting allowable incentive up to 5 percent. The relatively high incentives now possible with many of the profilograph specifications place an ever-increasing burden on the measurement process and data reduction process. Variability in test results can substantially affect contractor payments [Scofield 92].

Development Of AC Pavement Smoothness Specifications Factors Considered

In 1985 the Kansas Department of Transportation (KDOT) selected a 7.63 m (25 ft) California-type profilograph and a 5.1 mm (0.2 in.) blanking band for evaluation of the profilogram for determining the smoothness of portland cement concrete (PCC) pavement construction [Parcells 92]. In 1985 the first three PCC pavement projects with smoothness requirements were constructed. However, the incentive clauses were not exercised. Profilograph measurements were taken on each wheel path. The profilograph results in terms of profile roughness index (PRI) on 0.16 km (0.1-mi) intervals on these projects were analyzed. The first two projects had a high percentage of sections in the bonus range indicating that smoothness of 0 to 63 mm/km (0 to 4 in./mi.) was practical and achievable. In 1990 the specifications given in Table 7.18 were adopted as standards for controlling concrete pavement smoothness in Kansas.

Table 7.18 Schedule for Adjusted Payment for PCC Pavements, 1990 Specification 502.06 with 5.1 mm or 0.2 in Blanking Band

Profile Index millimeter per kilometer per 0.16 kilometer segment	**Price Adjustment Percent of Contract unit bid price**
48 or less	106
48.1 to 64	103
64.1 to 159	100
159.1 to 191	96
191.1 to 222	92
222.1 to 238	90
238.1 or more	88 (Corrective Work required or replace)

Although smoothness specifications with profilograph measurements were implemented on PCC pavements in 1985, new bituminous pavements had surface tolerance requirements as measured by a 3.05 m (10 ft) straight edge or a 7.62 m (25 ft) stringline at selected locations. The maximum variation of the surface in 3.05 m (10 ft) straight edge or a 7.62 m (25 ft) stringline at selected locations. The maximum variation of the surface in 3.05 m (10 ft) was not allowed to exceed 4.76 mm (3/16 in.); the maximum for 7.62 (25 ft) was 7.94 mm (5/16 in.) [KDOT 90]. Evidently these requirements were not sufficient for constructing smooth-riding bituminous pavements, and public complaints about the quality of rides on newly paved asphalt concrete (AC) pavements were rampant. By 1990 KDOT was very successful in controlling concrete pavement smoothness. This success led to the development of profilograph-based specifications for AC pavements in 1990. The major elements of the smoothness specifications for asphalt pavements evolved through consideration of the following:

- The roadway elements that normally would be included in the smoothness specifications for bituminous pavements are finished surfaces of the mainline pavement, side roads, auxiliary lanes, and ramps. Each of these elements should have a minimum paving depth of 102 mm (4 in.). This minimum thickness was selected because of economics. All KDOT's substantial maintenance projects have actions that are less than 102 mm (4 in.) thick. Because of budget restraints on the substantial maintenance program money, it was believed that this money should be spent on incentives.
- Unlike concrete pavements, there are no hand-poured sections on any of the elements described previously. Thus, a single set of specifications would be developed irrespective of the posted speed limit on the roadway.
- Specifications would be developed for statewide application regardless of route type of functional classification. This should encourage the contractors to pave uniformly throughout the state.
- The following would be excluded from pay adjustments under terms of the smoothness specifications:
    - Bridge decks unless to be overlaid.
    - Acceleration and deceleration lanes for at-grade intersections.
    - Shoulders.
    - Pavement on horizontal curves that have a 304.8 m (1,000 ft) or less centerline radius of curvature and pavement within the superelevation transition of such curves.
    - Pavements consisting of new or recycled bituminous concrete surfacing 102 mm (4 in.) or less in plan thickness
    - County secondary and federal aid urban projects unless specified otherwise on the plans, and
    - Projects less than 0.5 mi. in length (excluding bridge lengths).
- The California-type profilograph would be used for as-constructed smoothness measurements and the schedule for adjusted payment would be fashioned after that for concrete pavements. Doing this will bring some kind of parity between the specifications for these competing types of pavements. It was accepted that during paving of bituminous pavements, contractors had a better opportunity to meet smoothness requirements than during paving of concrete pavements so the disincentive payments would be much higher for bituminous pavements. The specifications for the "bumps" would be similar to that for concrete pavements (deviations in excess of 10.2 mm in a length of no more than 7.6 m, or 0.4 in. in 25 ft).
- The pavement smoothness would be established as a separate pay item with a zero-bid item amount. The pay schedule would then include incentives or disincentives in accordance with the pay schedule that will be added to or subtracted from the total contract amount through this pay item.
- The incentive or disincentive amounts would be determined on a 0.16 km (0.1 mi.) basis, which would be summed to an aggregate amount on a lane-mile basis. There should be a threshold target that when

exceeded would require. Specific remedial action by the contractor such as that in the concrete pavement specification (i.e. grinding).

- The test method and trace reduction procedures would be similar to those used for concrete pavements.
- The contractor would be responsible for determining the smoothness of pavement by operating a profilograph. KDOT may perform profilograph testing on the surface for monitoring and comparison purposes and during disputes over test results.

On the basis of these considerations, profilograph results for ensuring smoothness on bituminous pavements with greater than 102 mm (4 in.) paving depth were implemented through Special Provision 90P-39 as given in Table 7.19, where the roughness limits established were somewhat similar to PCC pavement schedule.

Table 7.19 Schedule for Adjusted Payment for AC Pavements Special Provision 90P-39 with 5.1 mm or 0.2 in Blanking Band

Profile Index (mm per km per 0.16 lane km)	**Contract Price Agreement per 0.16 lane km (Dollars)**
32 or less	+152.00
32.1 to 47	+76.00
47.1 to 142	0.00
142.1 to 174	-102.00
174.1 to 205	-203.00
205.1 to 237	-254.00
237.1 or greater	-305.00

Calculation of Incentive Payments

The highest incentive payment of $152/0.16 km (0.1 mi.) section as presented in Table 7.19 for the profile index of 32 mm/km (2.0 in/mi.) or less was based on the average cost of an AC overlay 89 mm (3.5 in.) thick, 161 m (528 ft) long, and 3.66 m (12 ft) wide. Many states pay for AC paving by the square yard paved; KDOT pays for AC paving by the tons of mix and again for the asphalt binder. For the Kansas condition, there was no direct correlation between pay items for AC and PCC pavements. Therefore, a direct conversion of PCC incentive payments for smoothness to AC condition was not possible. The 89 mm (3.5 in.) AC pavement thickness was arbitrarily selected because it was thought that the incentive payment should be compared with that for PCC pavement at this thickness level. Details of this incentive payment calculations are as follows:

- The amount of AC in an overlay section 89 mm (3.5 in.) thick, 161 m (528 ft.) long, and 3.66 m (12 ft) wide (unit weight = 145 pcf):
  0.2917 x 145 x 12 x 528/2,000 = 134 tons
- The cost of this section of overlay (based on the price of AC, BM-2 for KDPOT):
  134 tons x $18.90/ton = $2,532
- The price of this section including incentive payment (maximum 106 percent, based on then-current PCC pavement payment adjustment schedule):
  $2,532 x 1.06 = $2,684
- The maximum amount of incentive for a 0.16 km (0.1 mi.) section =
  $2,684 - $2,532 = $152.00

The payment schedule for the profile/index 33.1 to 47 mm/km (2.1 to 3.0 in.) was established to be half the amount for 0 to 32 mm/km (0 to 2.0 in./mi.) (i.e., $76/0.16 km or 0.1 mi. section). The disincentive amounts were made progressively higher (up to $305/0.16 km or 0.1 mi. section) to discourage contractor negligence.

Implementation of AC Pavement Smoothness Specifications

As mentioned earlier, the profilograph results for ensuring smoothness on AC pavements with greater than 102 mm (4 in.) paving depth were implemented through Special Provisions 90P-39 (as given in Table 7.19) during 1990. During this year, the incentive and disincentive clauses were not enforced. The profilograph results were collected and analyzed using the 5.1 mm (0.2 in.) blanking band. Table 7.20 gives the specification compliance for the 5.1 mm (0.2 in.) blanking band. Out of 851 sections (0.16 km or 0.1 mi.) constructed in 1990, there were 547 sections (64%) in the bonus range, 226 sections (27 percent) in the full-pay range, and 78 sections (9%) in the penalty zone. Figure 7.22 illustrates the results: no specific statistical distribution is obvious. Most of the sections were lumped in the bonus range. However, the data contained some sections on which profilograph specifications were not required but were considered rough and measurements were made.

Table 7.20 Profilograph Results on AC Pavement using 5.1 mm Blanking Band for 1990 Special Provision 90P-39

Roadway	No of 0.16 kilometer sections	Compliance with specified PRI (mm/km)					
		PRI (0-47) Bonus	(%)	PRI (47.1 - 142) Full-pay	(%)	PRI (>142) Penalty	(%)
All 1990	851	547	64	226	27	78	9

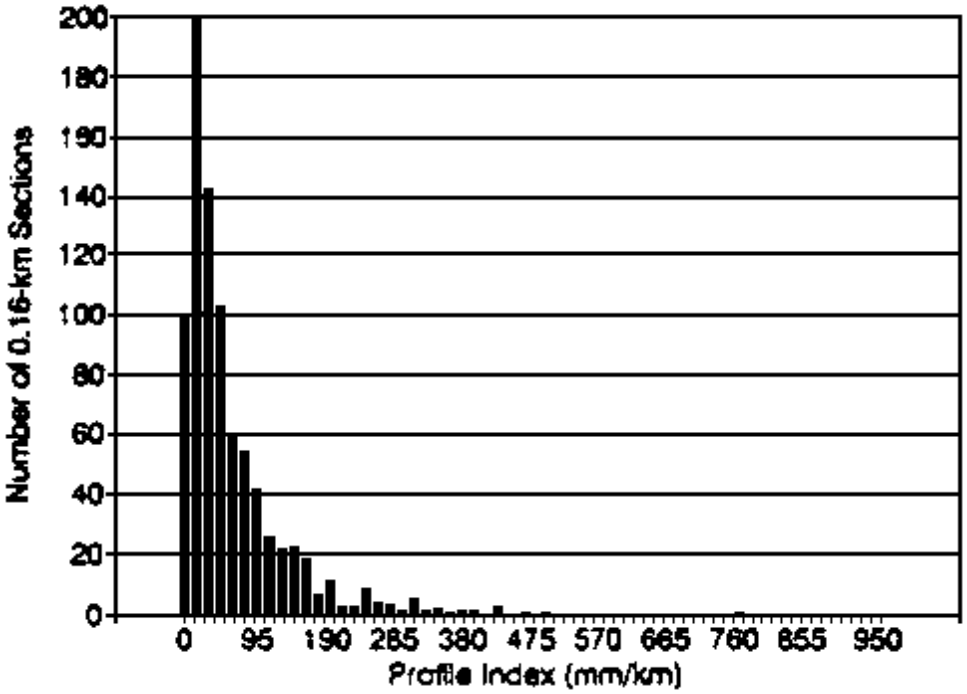


Figure7. 22 Specification Compliance of AC Pavement Sections with Special Provision 90P-39 (0.2 in. or 5.1 mm. Blanking Band)

Revised Profilograph Trace Reduction Procedure

In 1990 there was a noticeable high-frequency vibration on a concrete pavement reconstruction project on I70. This vibration was not noticed for another concurrent new PCC pavement project on I470, however. A closer review of the profilograph traces on these projects showed that on the I-70 project there was a consistent sine-wave cyclic oscillation of about 2.44m (8 ft) spacing and with 5.1 mm (0.2 in.) amplitudes. Most of these surface deviations were covered up by the 5.1 mm (0.2 in.) blanking band during trace reduction. On the I-470 project, the oscillation waves were of about 9.14 m (30 ft) spacing and about 5.1 mm (0.2 in.) amplitude, which were, again, covered up by the 5.1 mm (0.2 in.) blanking band during trace reduction [Parcells 94]. This issue of the effects of short wavelengths on PRI was tied to the question about the proper blanking band width.

The I-70 and I-470 projects of 1990 prompted KDOT to experiment with the blanking band width in order to qualify the apparent visual difference of profilograph traces on these projects. It was decided to use a zero blanking band width, or null blanking band. The null blanking band is nothing but a reference line usually placed approximately at the center of the trace having the line equally dividing the scallops above or below the centerline. The null blanking band was also extended to cover profilograms from bituminous pavements.

Reanalysis of the profilograms from the AC pavement projects of 1990 was done using the null blanking band. Table 7.21 presents the specification compliance for the null blanking band. Out of 842 sections analyzed, 71 sections (8%) were in the bonus range, 753 sections (90%) in the full-pay range, and 18 (2%) in the penalty range. Figure 7.23 illustrates the results. The distribution of the measurements is somewhat normal, which should be expected for a set of engineering measurements. It appears that the null blanking band has enhanced the ability of the profilograph to measure the smoothness of newly constructed AC pavements. However, these results made it obvious that the specifications of Special Provision 90P-39 needed to be changed in order to interpret null blanking band results [Parcells 92].

Table 7.21 Profilograph Results on AC Pavements using Null (0.254 mm) Blanking Band for 1990 Special Provision 90P-39

Roadway	No of 0.16 kilometer sections	Compliance with specified PRI (mm/km)					
		PRI (0-158) Bonus	(%)	PRI (158.1-631) Full-pay	(%)	PRI (>631) Penalty	(%)
All 1990	842	71	8	753	90	18	2

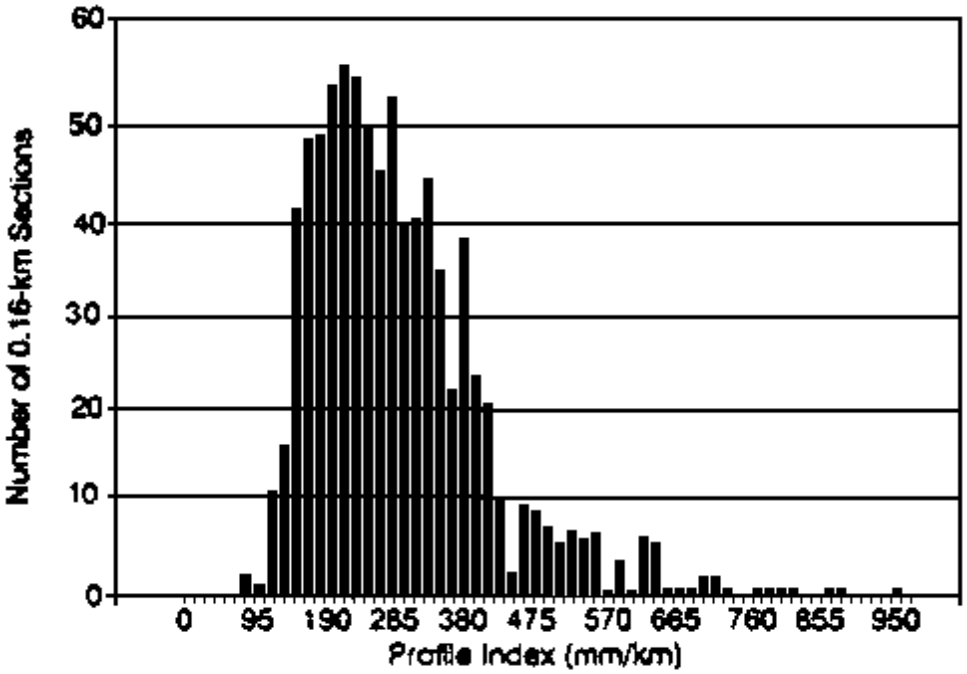


Figure7. 23 Specification Compliance of AV Pavement Sections with Special Provision 90P-39 (null blanking band)

Revision of Pavement Smoothness Specifications

In 1991 Special Provision 90P-39-R1 was incorporated for AC pavement projects that also required the use of the null blanking band for mechanical profilographs or 0.25 mm (0.01 in.) blanking band for computerized profilographs. The corrective action for a rough section was modified slightly, eliminating the requirement to reseal the diamond-ground pavement, and incorporated in Special Provision 90P-39-R2 in 1992. The schedule for adjusted payments in this special provision at various levels of smoothness achieved in construction is given in Table 7.22. This requirement was applicable to all projects with multiple paver passes including cold milling with overlay or cold recycle with an overlay. The working depth in those cases might be less than 102 mm (4 in.). However, pay adjustment did not apply if the plan thickness is less than 102 mm (4 in.) on the existing surfaces [KDOT 90].

In 1993 the results of profilograph testing on 5.866 0.16 km (0.1 mi.) sections from 30 paving projects in 1992 were analyzed using the null blanking band and compared with the results from the sections of 1990 and 1991. Table 7.23 gives the results. The 1992 results showed an increased percentage of sections in the bonus range with a similar reduction in the full-pay group. It is apparent that the smoothness limits in Special Provision 90P-39-R2 were achievable [Parcells 94b].

During the implementation of Special Provision 90P-39-R2, some contractors complained that requiring all pavement sections to be profiled on the same day they were placed was causing the contractors to stop paving earlier during afternoon hours in order to have time to finish rolling and profiling before reopening the highway to traffic.

Table 7.22 Schedule for Adjusted Payment for AC Pavements, Special Provision 90P-39-R2

Profile Index (mm per km per 0.16 lane km)	**Contract Price Agreement per 0.16 lane km (Dollars)**
110 or less	+152.00
110.1 to 158	+76.00
158.1 to 473	0.00
473.1 to 631	0.00 (correct back to 394 mm/km or less)
631.1 to more	-203.00 (correct back to 394 mm/km or less)

Table 7.23 Profilograph Results on AC Pavements Using Zero (0.254 mm) Blanking Band for 1993 Special Provision 90P-39-R2

Roadway	No of 0.16 kilometer sections	Compliance with specified PRI (mm/km)					
		PRI (0-158) Bonus	(%)	PRI (158.1-631) Full-pay	(%)	PRI (>631) Penalty	(%)
1990 (reanalysis)	842	71	8	753	90	18	2
1992 (reanalysis)	1890	57	3	1796	95	37	2
1992	5866	1467	25	4341	74	58	1
1993	4168	625	15	3499	84	42	1

Special Provision 90P-39-R3 contains an option allowing the contractor to delay profiling the final portion of a day's paving (not to exceed five 0.16 km or 0.1 mi. sections) until the first working day that production is continued on the same lane. When deciding whether to exercise this option, the contractor should be aware that the profile index of the pavement will probably be higher after it has been opened to traffic than it would have been if profiled as soon as rolling was completed.

As more and more AC pavements projects were being built with these smoothness specifications, the clauses of grind-back provisions to profile index of 394 mm/km (25 in./mi.) or less in Special Provision 90P-39-R3 were disputed by the contractors. They argued that if they had achieved a profile index of 473 mm/km (30.0 in./mi.) then no grinding would have been necessary but that a profile index of 473.1 mm/km (30.1 in./mi.) would require grinding back to 394 mm/km (25 in./mi.). Special Provision 90P-39-R4 and subsequent revision 90P-39-R5 now require grind-back to 473 mm/km (30.0 in./mi.) or less in case of a measured profile index greater than 473 mm/km (30.0 in./mi.) along with the penalty payment, if any.

Current Situation

In 1994 the results of profilograph measurements on 4,166 sections of AC pavement were collected from 24 paving projects completed in 1993. Table 7.23 presents the trace reduction results. Figure 7.24 illustrates the results graphically, and a normal distribution of the results is apparent. The traces were reduced using a null blanking band and the results were compared with those of 1990, 1991, and 1992. There is a decreased percentage of sections in the bonus range with a similar increase in the full-page group. The results show that although the incentive payments have decreased, the currently used specifications for AC pavements are achievable by the contractors. The results should establish that under current specifications bonus can be achieved through better quality paving rather than by chance.

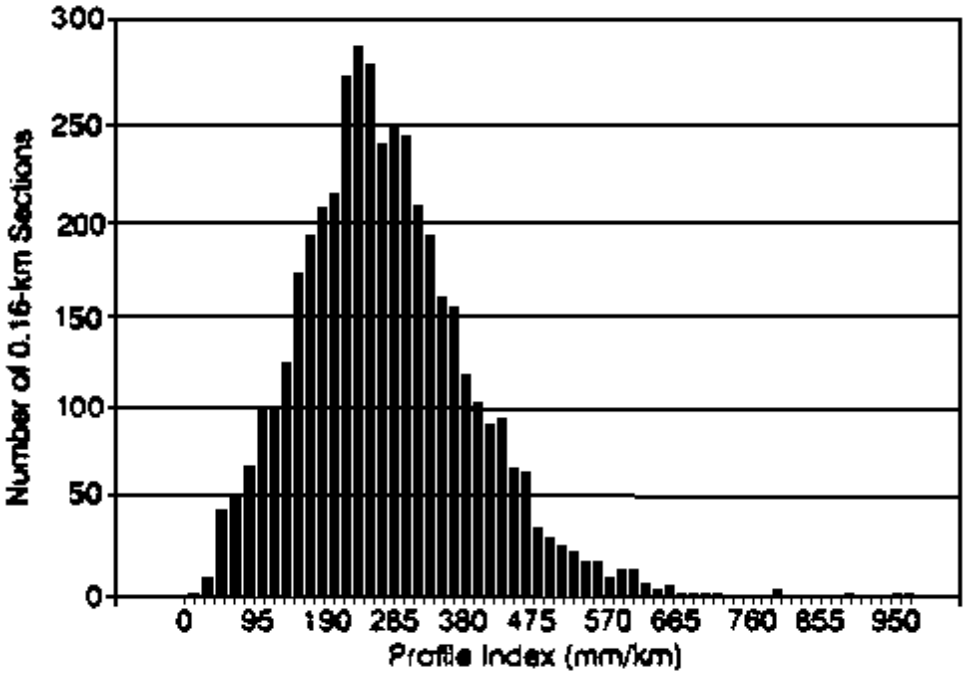


Figure 7.24 Specification Compliance of AC Pavement Sections with Special Provision 90P-39-R2 (null blanking band)

Cost Analysis Of AC Pavement Smoothness Specifications

The incentive and disincentive payments made to the contractors in 1991, 1992, and 1993 were analyzed to determine a trend in such payments. Table 7.24 provides the results of this analysis and Figure 7.25 illustrates the results graphically. The incentive payments were much higher during the second year of the implementation of smoothness specifications. The incentive to lane0limometer-paved ratios was 12.07, 203.59, and 129.25 (22.5, 325.7, and 206.8 for lane miles) for 1991, 1992, and 1993 respectively. As illustrated in Figure 7.25, the incentive payments were showing trends of stabilization after a sharp increase in 1992. The disincentive payments were very minimal compared with the incentive payments in 1992 and 1993; they were somewhat stable then, also. However, the opposite was true in 1991. This indicates that the new specifications have made a positive impact on the overall quality of AC paving over the past three years.

Table 7.24 Results of Cost Analysis of AC Pavement Smoothness Specifications

Year	No. Of 0.16 km 0.17 Sections	Bonus ($)	Bonus/ Lane km Paved ($/km)	Penalty ($)	Penalty Lane km Paved ($/km)
1991	1890	4256	14.07	7919	26.19
1992	5866	191084	203.59	4060	13.43
1993	4568	94488	129.25	3857	12.75

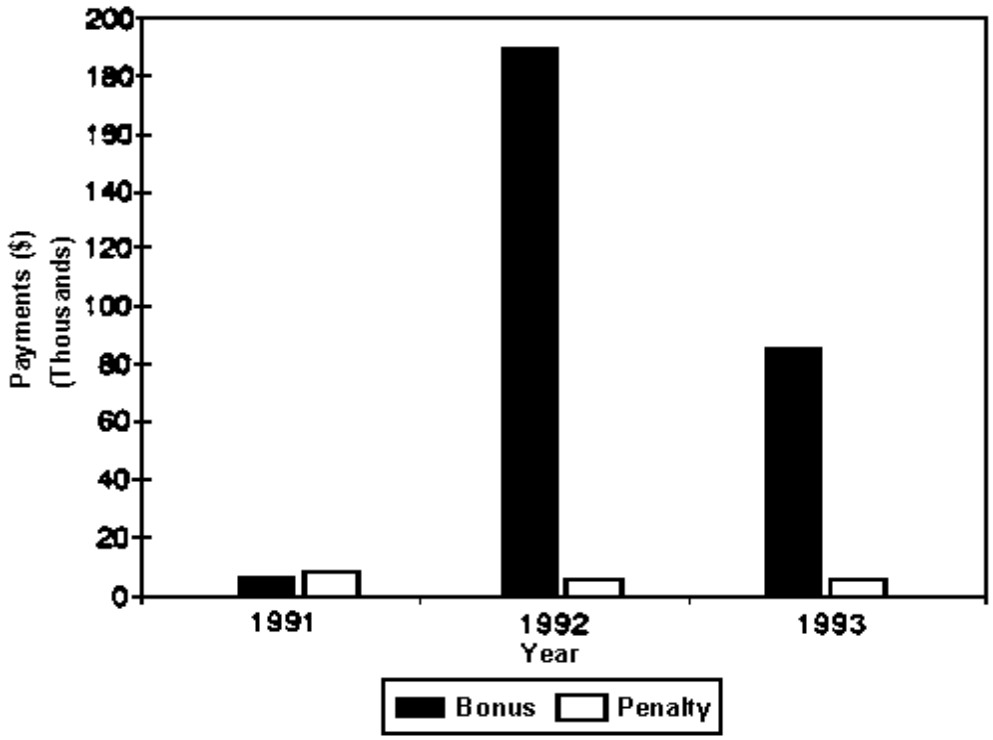


Figure 7.25 Incentive and Disincentive Payments for AC Paving in 1991, 1992, and 1993

Conclusions

Smoothness specifications for AC pavements now in effect in Kansas have evolved over the past few years through a number of revisions. Pavement profiles with short wavelengths and smaller amplitudes than the industry-accepted 5.1 mm (0.2 in.) can harm the ride quality of pavements. This experience has led KDOT to eliminate the blanking band width in the profilograph trace reduction process first for concrete pavements, then for bituminous pavements. The implementation of this zero, or null, blanking band was successful and has resulted in better quality pavements in Kansas. The currently used specifications for AC pavements can be achieved by contractors, and the number of sections in the bonus range indicates that incentive payments have encouraged better paving than in the past. This should have a positive impact on asphalt pavement paving in Kansas. In general an increasing number of miles of pavement with low profile index are now being constructed since smoothness specifications for bituminous pavements were implemented.

7.5.3 Other Examples

Cold In-place Recycling

Cold in place recycling has been a standard rehabilitation treatment in Kansas for a number of years however the mix type previously used in Kansas was very sensitive to asphalt content. A slight increase in asphalt tended to cause instability, a slight decrease in asphalt has caused brittleness and raveling. This problem has been identified in the PMS for various pavements and a revised mix design and specification incorporating fly ash has been developed. Since they have changed to this new mix design, evidence in the PMS indicates that the rehabilitation with in-place recycling is performing at an improved level.

The pavement management system and the PMS database provided clear information for this engineering application. Without the PMS, it would not have been possible to easily evaluate the problems such as transverse cracking and rutting, which may be associated with cold in-place recycling. Asphalt mix types are identified in the database, which allowed the CIR projects to be readily identified. Thus, the PMS was a fundamental tool used to initiate and support this engineering application.

Evaluation of Joint Distress Related to Changing Aggregates Specifications

Kansas pavements have always suffered from difficulty with aggregate related deterioration termed "D-cracking." In the early 1980's state DOT personnel evaluated data from the pavement management database to quantify the difficulty they were facing with their aggregate specifications. As a result they developed a new aggregate specification resulting in a more durable type of aggregate being used for the last 16 years to relieve the D-cracking problem. The PMS database is used to track field performance of the new aggregates. In the PMS the D-cracking associated with durable aggregate is primarily reflected by the level of observed joint distress, which is recorded in the condition surveys.

For the past 16 years the new aggregate specification has been in force. The Kansas DOT pavement management staff has reviewed the pavement management data for the past several years and has shown that the ratio of joint distress for pavements using the new aggregate specification versus the pavements using the old aggregate specification is much lower. That is the new aggregate specification is functioning very well and the better aggregate is tending to eliminate D-cracking.

Without the PMS it would have been very difficult to separate the existing pavements by aggregate type. It would have been very difficult to evaluate the distress history and to evaluate the progression of distress on these pavement sections because the data would not have been available for the study. This case study is a clear indication of the engineering applications of the pavement management system in KDOT and of the value of a pavement management database for engineering applications.

Maintenance Awareness

The pavement management group feels strongly that providing reports to the districts on projects selected by the pavement management group and providing the ratings of all pavement within each district has heightened the awareness of the need for maintenance in the districts. In this case, those pavements that are in poor condition (but are not budgeted within the annual budget) often receive extra maintenance to keep them functioning properly. Whereas those pavements that are scheduled for early rehabilitation or reconstruction are not maintained because it is considered to be a waste of money. In this case the pavement management system is clearly improving the maintenance function within the districts.

The ability to enhance the maintenance value of the pavement management system is partially disrupted by the lack of a connection with the maintenance management system database that operates within the state. As in many other states the maintenance management system is operated in a separate section and there is no connection between the two databases, nor is one possible at the electronic level. Only paper feedback on maintenance can flow into the pavement management process. This is a serious problem.

Evaluation of Environment

In 1993, Kansas experienced a major reduction in the average pavement serviceability level between two consecutive years. This coincided with an extremely wet rainy season and excessive flooding in Kansas. The pavement management system provided adequate data for a study to be carried out comparing rainfall data from the NOAA database against the PMS data. In this particular case, the study did not provide a good positive correlation, however this does not detract from the value of the PMS data. It suggests that additional data items or a different approach to the study was needed. The addition of environmental information to the PMS can help in engineering analyses of PMS data.

Asphalt Mix Design

The Kansas DOT uses a wide variety of asphalt concrete mixes but there is no valid summary of which mixes are performing best. This could be evaluated if mix design information was made a part of the PMIS. This is not currently done and therefore it is not possible to define, from the materials point of view, which mixes are best. The pavement management system group would seriously like to add this additional data to its database and this is a demonstration of how the PMS could be changed for engineering purposes.

Evaluation of New Design Concepts

New design concepts are often recommended for trial by the Federal Highway Administration or other sources. One such example is the installation of positive drainage in new pavement sections as well as the retrofit of subsurface drainage in existing pavements. During the past several years Kansas has tried several sections of pavement with positive drainage. The pavement management staff feels that this positive drainage has not improved pavement performance in Kansas. They have not yet tested this but if a good set of data associated with new design concepts was maintained as a part of the PMS database it would easily be possible to test this concept.

State DOT's should give significant thought to including detailed data on pavement characteristics in their PMS for any pavement which is testing new or modified concepts of design, construction or materials.

7.5.4 Ideas for Future Development

Rehabilitation

The PMS team is concerned that the current rehabilitation approach in Kansas is merely putting a series of band-aids on their pavements and that thin overlays are continuing to maintain a good "performance level" for functional use. They feel however, that the basic pavement structure is itself slowly disintegrating and wearing out to the degree that at some point it will not be possible to get adequate life out of thin overlay rehabilitation. Consequently it is feared that the underlying value of the pavement structure statewide is deteriorating and the PMS team would like to be able to make this evaluation. Currently the pavement management system will not allow this.

Design in Lieu of POS

At the current time, since POS is not functioning, a three-man team goes out to decide on actions to be done on individual sections of pavement in lieu of POS. This group chooses from among five or six alternatives. They select those candidates for actual design that are the most potentially valuable. This is in effect an expert system using experts themselves rather than capturing that data for a computer program.

Laboratory Information Management System

The PMS staff is looking for the development of an automated laboratory information management system that would tie into the pavement management database. It is considered that this would be a significant improvement in the pavement management system activity.

Research Program

KDOT has a research program patterned along the TTI/Texas program. Research is done at both the state and university level at both major universities. One of the major developments from this program has been PROSCAN at Kansas State University by Dr. Devore and Dr. Hossain who developed an automatic program for reading profilograph traces. This PROSCAN is being used by other states also. A program also is being funded currently to improve POS as previously discussed.

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8 PMS FOR TRACKING PREVENTIVE MAINTENANCE ACTIONS

8.1 INTRODUCTION

An up-to-date PMS database is an essential tool to determine the need and the timing of preventive maintenance, but also to monitor the performance of the maintenance treatment. In this module several examples from State DOTs will be given, and both aspects will be covered.

There is often confusion about the meaning and the scope of preventive maintenance, and about the treatments that can or should be used under this heading. Therefore, attention will be paid first to the definition(s) of the term Preventive Maintenance and to the treatments that would normally qualify to be called "preventive." For this item use is made of information obtained from the FHWA/NHI course "Pavement Preservation: The Preventive Maintenance Concept" [FHWA 00].

The first examples of the role of Pavement Management Systems in tracking the performance of preventive maintenance actions come from a study by Dr. Gilbert Baladi carried out in 2000/2001 for the FHWA [Baladi 01]. Dr. Baladi visited six DOT organizations, and a summary of his findings will be given in Section 8.3. Following that an example will be given in Section 8.4 of the use of a District Level Index to select projects for Preventive Maintenance in Texas DOT, and the experience of Wisconsin DOT with Preventive Maintenance Strategies for Continuously Reinforced Concrete Pavements will be covered in Section 8.5.

8.2 WHAT IS MEANT BY PREVENTIVE MAINTENANCE

The following text has been taken from the FHWA/NHI course "Pavement Preservation: The Preventive Maintenance Concept." [FHWA 00]

Maintenance, routine maintenance, rehabilitation, and reconstruction: these terms are used frequently but do not mean the same thing to everyone. The confusion on this topic is understandable, as almost every individual agency, as well as the Federal government and other industry organizations, have weighed in with a definition of their own.

In AASHTO's *Maintenance Manual* [AASHTO 87] the following definition of maintenance is found: "A program to preserve and repair a system of roadways with its elements to its designed or accepted configuration." Another widely referenced document, AASHTO's *Guide for Design of Pavement Structures* [AASHTO 93] offers minimal guidance on maintenance treatments or practices, but does offer this definition of maintenance: "the preservation of the pavement including shoulders and related drainage." These concepts of maintenance are certainly broad enough to cover almost any reactive effort applied to a pavement.

Routine maintenance, for which a need was first identified shortly after the first paved surfaces were constructed, is reactive in nature, fixing a defect once it occurs. Unfortunately, routine maintenance activities–such as the filling of a pothole or sealing cracks on a failing pavement–are what most people think of when they hear the term "maintenance." Routine maintenance is frequently performed on pavements that are failing. It is a stopgap approach that keeps traffic moving, but is rarely thought of as contributing to long-term pavement performance. Routine maintenance is often performed under harsh conditions; the weather is often bad or the condition of the pavement mitigates against a long-lived repair. These types of repairs historically perform the worst and draw the greatest ire from the traveling public.

By replacing "routine" with the term "preventive," a completely different approach to pavement preservation is suggested. While the terms "preventive" and "maintenance" have been paired before, it is really only in the past decade that highway agencies have started to actually implement programs that address the intent of this concept and have begun to see the resultant benefits. In addressing the newfound emphasis in this area, AASHTO's Standing Committee on Highways has provided the following explanation of preventive maintenance [FHWA 99]:

> *… the planned strategy of cost effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system [without increasing structural capacity].*

Geoffroy, in NCHRP 223 [Geoffroy 96], describes a preventive maintenance strategy as "an organized, systematic process for applying a series of preventive maintenance treatments over the life of the pavement to minimize life-cycle costs."

These descriptions suggest that planning is essential, and that a treatment's costs, results (effectiveness), and performance must be considered. These ideas are best summed up by the definition that has been used by AASHTO's Lead State Team on maintenance [FHWA 99]):

Preventive maintenance is applying the right treatment to the right pavement at the right time.

This definition readily differentiates preventive maintenance from reactive or routine maintenance. Preventive maintenance anticipates pavement performance, and suggests the use of treatments that can be applied in a timely manner to improve service to the customer.

Formal definitions can help to create a common basis for understanding the concepts associated with preventive maintenance. Based on the above discussion, the following definitions are suggested.

- **Preventive Maintenance** [AASHTO 98]: Preventive maintenance is the planned strategy of cost effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system [without increasing the structural capacity].

- **Pavement Preservation**: The sum of all activities undertaken to provide and maintain serviceable roadways; this includes corrective maintenance and preventive maintenance, as well as minor rehabilitation projects [FHWA 99].

- **Pavement Preventive Maintenance**: The application of one or more treatments (such as joint repair, seal coats, shoulder repair, and restoration of drainage systems), generally to the surface of a structurally sound pavement, which are cost-effective methods of preserving the pavement structure [FHWA 99].

- **Pavement Rehabilitation** [AASHTO 87]: Resurfacing, restoration, and rehabilitation work undertaken to restore serviceability and to extend the service life of an existing facility. This may include partial [surface] recycling of the existing pavement, placement of additional surface materials or other work necessary to return an existing pavement, including shoulders, to a condition of structural or functional adequacy. [FedReg 89]

- **Pavement Reconstruction**: Construction of the equivalent of a new pavement structure, which usually involves complete removal and replacement of the existing pavement section including new and/or recycled materials.

While the terms may be used different ways within different agencies, especially as far as funding is concerned, these definitions address the broad distinctions between the different types of pavement activities.

So far the information from the FHWA/NHI course "Pavement Preservation: The Preventive Maintenance Concept" about the definition of "Preventive Maintenance". The same course also presents a list of preventive maintenance techniques for asphalt and PCC pavements; this list is reproduced in Table 8.1.

Table 8.1 List of Preventive Maintenance Techniques

Preventive Maintenance Techniques for AC Surfaced Pavements	Preventive Maintenance Techniques for PCC Surfaced Pavements
• Crack Filling/Crack Sealing • Fog Seals • Slurry Seals • Microsurfacing • Chip Seals • Cold In-Place Recycling (CIR) • Hot In-Place Recycling (HIR) • Milling • Thin Hot-Mix Asphalt (HMA) Overlays • Maintenance of Drainage Features • Other Maintenance Treatments	• Joint Resealing • Crack Sealing • Diamond Grinding/Diamond Grooving • Undersealing • Full-Depth Repair • Joint Spall Repair • Load Transfer Restoration • Maintenance of Drainage Features

8.3 STUDY FOR SIX STATE DOT'S PREVENTIVE MAINTENANCE (BALADI STUDY)

8.3.1 Scope of the Baladi Study

The scope of this study is to synthesize and evaluate the state-of-the-practice of six SHAS who have developed and implemented preventive maintenance (not reactive or routine maintenance) program based on pavement needs. The evaluation will be based on the analysis of the historical distress data of the particular SHA. During the course of the study, the types of analyses and the forensic investigation used by the four SHAS to select the project boundaries, time for construction, and preventive maintenance actions were also obtained and are summarized in the report.

The selection of the six SHAS was accomplished through consultation between the principal investigator, special expert group and FHWA personnel. First, a pool of candidate SHAs was identified based on their preventive maintenance programs and the availability of the pavement condition data that can be sued for the evaluation of the various preventive maintenance actions. Fromthis pool six SHAs were selected and visited. [Baladi 01].

8.3.2 Executive Summary – Baladi Preventive Maintenance (PM) Study

One of the first lessons that can be learned from asset/pavement management is that it is generally insufficient to rely solely on one's intuition. Stated differently, it is not sufficient to drives over a pavement section, observes and/or feels its current condition, and relies on this intuition alone to decide whether or not the section is a candidate project for preventive maintenance and/or rehabilitation and what type of action that needs to be taken. Although the model of driving over a pavement section has its merit and may yield information relative to our perception of the ride quality and the current pavement surface condition, in most situations, the information is not adequate to make good, balanced and cost-effective decisions. Some pavement sections may have good ride quality and pavement surface condition and yet their rates of deterioration are high enough to warrant action immediately or within one or two year time.

Some may argue that the model of driving a pavement section and making decisions has worked for many years. Although, this point cannot be disputed, it does not mean that the model is efficient or cost-effective. Any model, however old, must be tested before its accuracy and efficiency can be determined. The true test of making decisions based solely on driving over a pavement section is "the retrospect test." The test involves using historical pavement distress and preventive maintenance data to reconstruct the past and to determine how well the model has worked in selecting projects and prescribing actions. The test can be performed on many projects if one is willing to modify the existing practice/model and to download and examine the historical pavement management distress data. The information obtained from such an examination plus those obtained from driving the pavement section would make people better informed and hence, would enhance the quality and accuracy of their decisions.

Well-engineered and managed pavement preservation processes include a preventive maintenance program. For each potential project, the program must address the selection of space (project boundaries), time of construction, and feasible preventive maintenance actions. Such selection of space, time of construction and PM action is the essence of engineering management of the pavement network and has substantial impact on the longevity of the pavement. Recently, few State Highway Agencies (SHAs) have developed and implemented or in the process of implementing preventive maintenance (PM) program as a part of their pavement management system. During the implementation process few have raised the question of whether or not the PMS distress data can be used to manage the PM program. Hence, the objectives of this study are to examine and discuss the role that the PMS data have played in the development and implementation of the PM programs and in tracking the performance of PM projects.

The scope of the Baladi study is to synthesize and evaluate the state-of-the-practice of six SHAs (Arizona, California, Georgia, Michigan, Montana and Pennsylvania) who have developed and implemented PM (not reactive or routine maintenance) program based on pavement needs. The evaluation was based on the analysis of the pavement historical distress data. During the course of the study, the types of analyses and the forensic investigation used by the six SHAs to select the PM project boundaries, time for construction, and preventive maintenance actions were also obtained and are included in the report. Based on the analysis and on a site visit of each project, the following conclusions were drawn:

The Role of the PMS Distress Data

1. In all six agencies, the pavement distress data in the PMS databank are relatively accurate and reflect the true pavement surface conditions observed in the field.
2. Although the PM practice varies from one SHA to another, all six agencies use the PMS distress data and the pavement rate of deterioration to select PM project boundaries and the type and time of action.
3. In each agency, it was stated by various staff members that the PMS distress data played a crucial role in the development of cost-effective PM programs.
4. All six agencies use the pavement distress data to track the performance and determine the cost and benefits of PM projects.
5. There is no need to create a separate pavement distress databank for the PM program. The PMS distress data can be used for the entire pavement preservation program.

Other Issues

1. In all agencies, the pavement distress and cost data showed that the old policy of constructing pavement and letting it go to rehabilitation or reconstruction is not a cost-effective policy and it has created substantial user dissatisfaction.
2. In all six agencies, the preemptive action of the PM program has resulted in a substantial decrease in the amount of dollars allocated to reactive maintenance.
3. Based on the PMS distress data and the pavement rate of deterioration, some agencies have developed PM guidelines regarding the selection of the optimum time window for PM action and the expected extension in the pavement service life due to that action.

Recommendations

Although most SHAs today collect pavement distress data on a specified frequency, few have yet to realize the full potential and benefits of such data. Hence, such agencies have not realized the full return on their investment in the PMS distress data. Based on the results of this study and on various conversations with engineers and staff of numerous SHAs, and on site visits to numerous pavement projects, the following recommendation was drawn: Current activities in the asset management area, in general, and the pavement management, in particular, should be redesigned and the resources be streamlined opt provide full support to SHAs to realize the full potential of their PMS distress data.

8.3.3 Arizona Department of Transportation Visit

General

The operational structure in the pavement areas within the Arizona Department of Transportation can be divided into two categories; exclusively main office functions and main office with district office consultation. For example, pavement design, planning, cost estimates, pavement management, and pavement distress data collection are exclusively main office functions. On the other hand, the main office with district consultation accomplishes the selection of pavement projects and type of fixes based on the PMS distress data and pavement needs. Such use of the PMS data has, over time decreased if not eliminated arbitrary and inefficient project selection.

One important point should be noted here is that, the number of complaints regarding the discrepancies between the actual pavement conditions and the PMS pavement condition data have substantially decreased over time. Hence, the confidence level in the accuracy of the PMS data has increased.

The PMS database in Arizona contains 30 years of historical distress data. The data are typically used to track project performance and estimate the conditions of the pavement network. Based on such data, it is estimated that the service lives of rigid and flexible pavements in the State of Arizona are about 40 and 18 years, respectively. It should be noted that the pavement management system in the state is mainly driven by pavement roughness.

Pavement Preservation

The pavement preservation program in the State of Arizona consists of three components:

- Reconstruction
- Rehabilitation
- Preventive maintenance

In addition, the Arizona DOT conducts routine maintenance activities that include mainly flush seal and scrub seal and single and double chip seal. The flush and scrub seal activities are typically conducted by the DOT, while contractors conduct the others.

State Road 88 (SR 88)

State Road 88 (SR 88) in Arizona is a 2-lane road that was maintained in 1996. The preventive maintenance project on both lanes is nine mile long (between mileposts 201 and 209). The project consisted of 2-inch dense graded asphalt concrete overlay capped by 5/8-inch asphalt rubber/asphalt concrete friction course. The project was visited in September 2000.

Figures 8.1 through 8.4 depict the historical distress data of SR 87 between mileposts 201 and 209. Figure 8.1 shows the percent cracking of each mile and the period (1986 to 1999). Figure 8.2 shows the average percent cracking of SR 87 for the same period. As can be seen, the

percent cracking increased from about 3% in 1985 to about 17% in 1995 and it decreased to 0 after the preventive maintenance action was completed in 1996. Four years after construction, the percent cracking remained at 0% level. Similarly, the improvement in the pavement roughness is shown in Figures 8.3 through 8.4.

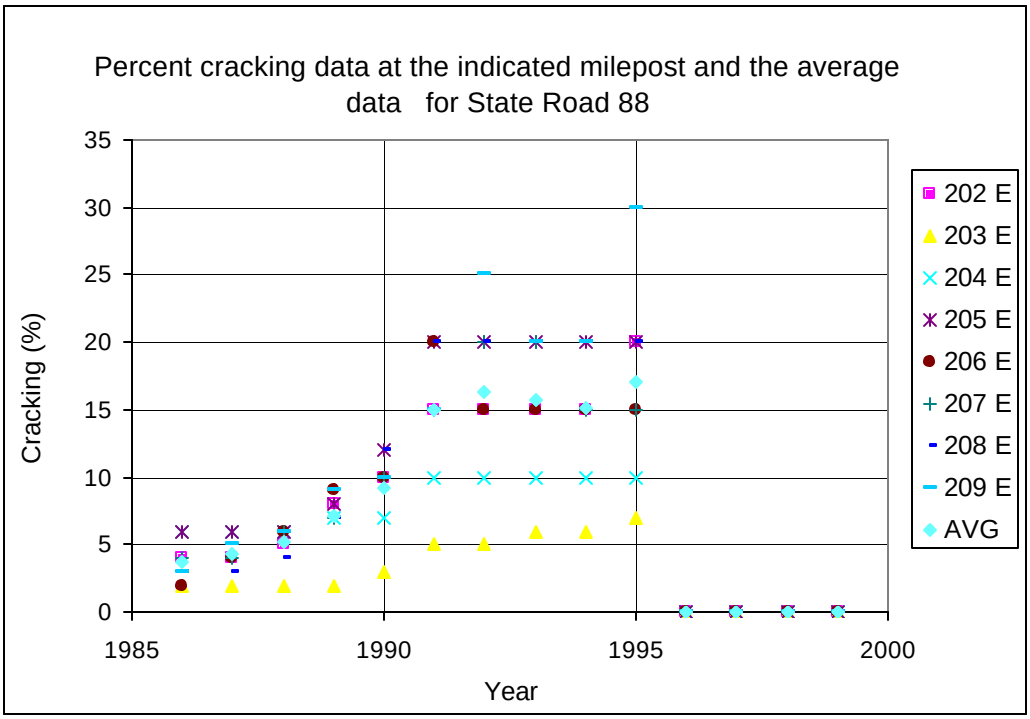

Figure 8.1 Percent Cracking Data

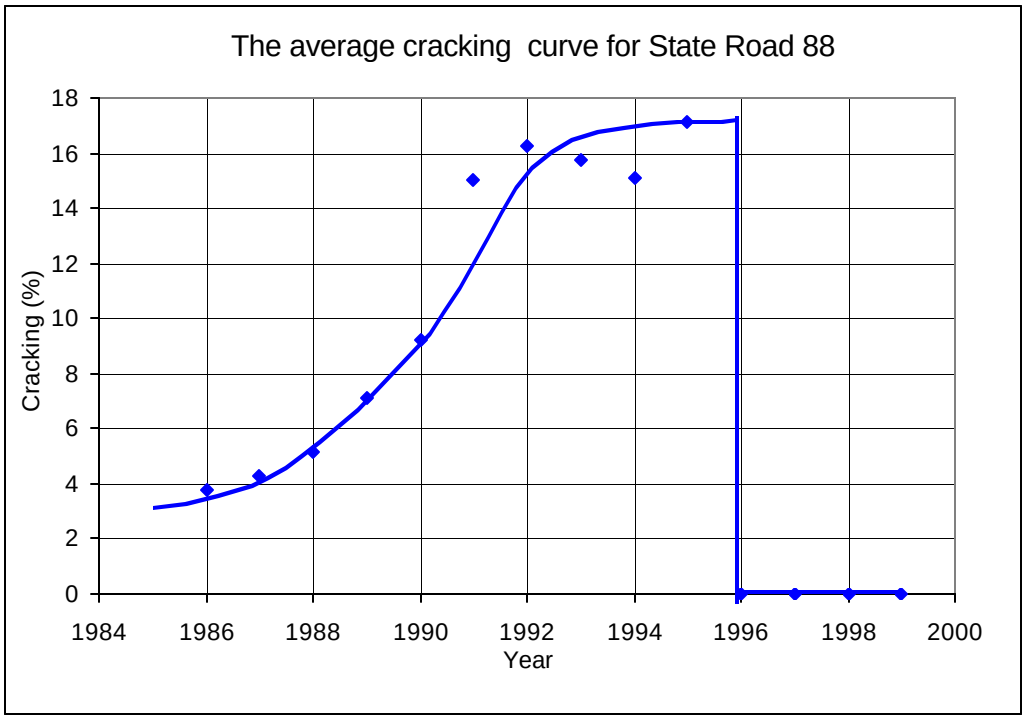

Figure 8.2 The Average Cracking Curve for State Road 88

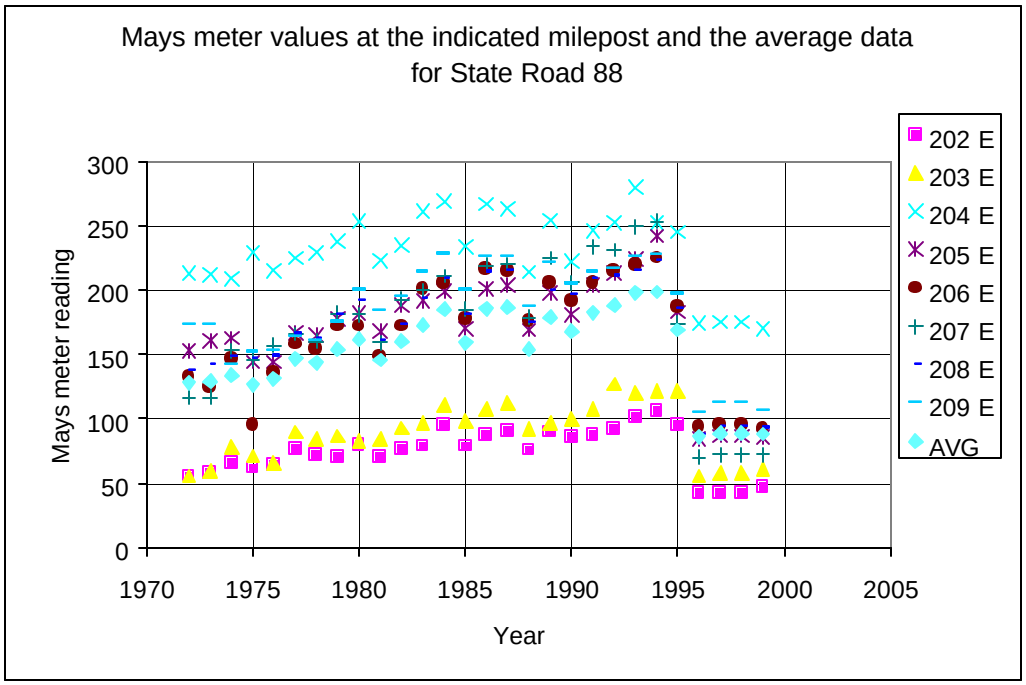

Figure 8.3 Mays Meter Values at the Indicated Milepost and Average Data

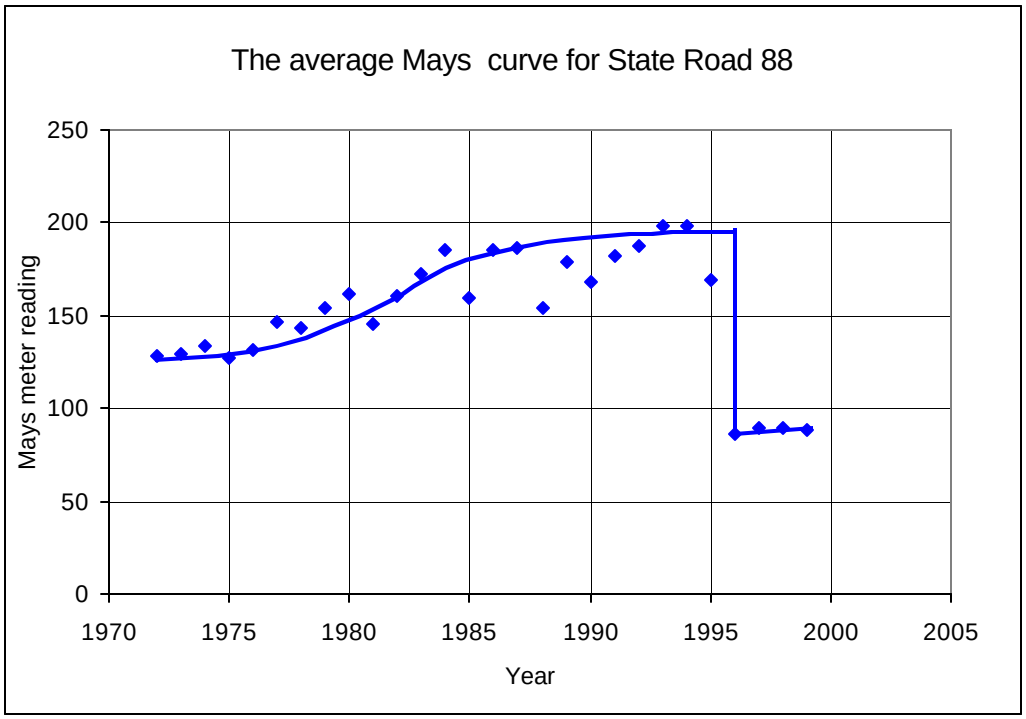

Figure 8.4 The Average Mays Curve for State Road 88

8.3.4 California Department of Transportation Visit

General and Background

The California Department of Transportation (Caltrans) used to conducts a biennial manual Pavement Condition Survey (PCS). Since 1999, the survey has been done annually. The survey crews travel the state and observe and record the condition of the pavements. For rigid pavement, continuous data are collected on each slab. For flexible pavements, the distress data along 100-feet of each 1/3-mile pavement are collected. Along each unit survey, the survey crews collect the severity and extent of various structural distresses. Caltrans combines the individual distress data and classify the roads into three levels of structural distress none, minor and major. In addition, a van travels the highways and collects the pavement profile data. The data are used to calculate the ride quality, which is expressed by the International Roughness Index (IRI). A concrete road with IRI values of 213 or more and a flexible road with IRI values of 224 or higher are unacceptable.

Since 1982, Caltrans classifies the pavement network into three maintenance service levels or classes as follows:

- Maintenance Service Level 1 (MSL 1) or Class 1 – This class consists of all interstates, freeways and principal arterial routes with more than 5000 ADT.
- Maintenance Service Level 2 (MSL 2) or Class 2 – Typically this class consists of those roads, which are minor arterials.
- Maintenance Service Level 3 (MSL 3) or Class 3 – This class consists of all local connector roads.

Caltrans uses two criteria, ride quality and structural condition to establish the condition of each pavement section. The information is combined with the MSL to establish the priority level of that pavement. The priority level is then used to establish the likely class of preservation option to be used. Rehabilitation is assigned to the highest priority followed by Capital Preventive Maintenance (CAPM) action and followed by major maintenance action. Finally, routine maintenance actions could be taken on any road that does not receive priority level.

The CALTRANS Pavement Preservation Program

The Caltrans' pavement preservation program consists of four major categories as follows:

- Rehabilitation (either traditional or 'longer-life' treatments)
- Capital Preventive Maintenance (CAPM)
- Major maintenance
- Preventive maintenance

Preventive maintenance actions are taken on undamaged pavements or those with minimal damage. It is intended to preserve pavements already in good to excellent condition, postponing or avoiding the need for structural repairs. The four categories are addressed below.

Capital Preventive Maintenance

A new class of treatments implemented at Caltrans in the mid-1990's is Capital Preventive Maintenance (CAPM). Those treatments are used on pavements with minor to moderate damage. The cost of CAPM treatments is roughly $70,000 per lane-mile, providing five to seven years or more of service life extension. Recently, Capital Preventive Maintenance treatments have been determined to be effective as a 'preventive' action on undamaged pavements.

Major Maintenance

Major maintenance is performed on roads with a minor or major structural problem, or on a road that is developing a safety problem, such as low skid resistance. These treatments provide some increase in the structural capacity of the pavement, and substantially improve the wearing surface of the road. The cost of these treatments is roughly $4,000 to $45,000 per lane-mile, providing from one to five years or more of service life extension. The wide range in cost

is due to the broad range of materials and conditions under which they may be applied, and the various highway characteristics and pavement needs these treatments are capable of correcting.

Preventive Maintenance

Preventive maintenance is applied to pavements without significant structural problems in order to provide some improvement in condition, and also reduce the rate at which distress accumulates. For example, these treatments can be used to improve drainage and prevent water infiltration, or renew the wearing surface of the pavement. Since many of the treatments are the same as major maintenance strategies, the cost of these treatments is similar, roughly $2,500 to $30,000 per mile.

Preventive Maintenance Funding, Benefits and Actions

Effective with the 1999-2000 fiscal period, Caltrans received $50 million annual increase to support additional preventive maintenance activities on at least 2,000 additional lane miles each year. Preventive maintenance utilizes relatively frequent, low-cost treatments applied to pavements in good condition. These treatments delay the development of structural distress that requires more intensive and costly corrective strategies. Equivalent annual cost comparisons show that preventive maintenance treatments have a benefit-cost ratio of 6:1 or more. Further, preventive maintenance treatments are usually less disruptive of traffic, resulting in reduced user-delays and user-costs. In combination with properly timed corrective treatments, a comprehensive life cycle cost analysis spanning 20 to 30 years shows overall benefit-cost ratios can rise to 10:1. Over time, increased preventive maintenance activity will help minimize the average cost and total cost of the pavement maintenance program.

State Road 16

State Road 16, which carries about 5500 ADT, is an asphalt road that showed some alligator and longitudinal type cracks. The road was widened and rehabilitated in 1993. The project was divided into several test sections and preventive maintenance actions were taken using different asphalt mix overlays. Once again, the research team visited the project on April 10, 2000, drove over the pavement sections, and observed the condition of the pavement and its ride quality. The figures and the type of preventive maintenance actions are summarized in Table 8.2 below.

Table 8. 2 Preventive Maintenance Actions on State Road 16 in California

State Road 16				
Section number	**Figure number**	**Preventive maintenance action**	**Age (years)**	**Pavement condition**
1	CA.4	½-inch rubberized AC overlay	7	Very good
2	CA.5	¾-inch rubberized AC overlay	7	Very good
3	CA.6	1-inch rubberized AC overlay	7	Excellent
8	CA.7	½-inch rubberized AC overlay	7	Very good
9	CA.8	¾--inch rubberized AC overlay	7	Very good
12	CA.9	1-inch rubberized AC overlay	7	Excellent
Control	CA.10	4-inch AC surface	7	Patching in the wheel paths due to alligator cracking
East of 15	CA.11	1-inch conventional AC overlay	2	Longitudinal and transverse cracks

Route 32 (Strategic Highway Research Program SPS 3)

Route 32 in Butte County, California was included in the Special Pavement Study (SPS 3) of the Strategic Highway Research Program (SHRP). The objectives of the SPS 3 experiment included the analysis of various preventive maintenance actions on pavement performance. In 1992, route 32 was maintained using several types of slurry seal and chip seals and various thin asphalt overlays. Caltrans monitored the road performance using the pavement management system (PMS). Once again, it was not possible to download the PMS data because of the redesign of the PMS software. In any event, the research team visited the road on April 10, 2000. During the visit, the team members examined the condition of the pavement surface and obtained digital images of the various road sections.

Table 8.3 summarizes the various section of the SPS-3 experiment of the Strategic Highway Research Program (SHRP). For each section the summary includes:

1. The preventive maintenance action that was taken during the 1992 construction season
2. The corresponding figure numbers where the digital images can be viewed
3. The age of the preventive maintenance action
4. The type of the last preventive maintenance action that was taken in 2000
5. The general condition of the pavement that was observed during the April 2000 visit

Nevertheless, examination of the summary data in Table 8.3 indicate that nine years after construction, most pavement sections are in good condition except the control section (do-nothing) where the pavement condition is in fair to poor condition.

The above observation indicates that, relative to the control section, all of the preventive maintenance actions taken on Route 32 in Butte County, California have extended the service life of the pavement in a cost-effective manner.

Table 8.3 SPS-3, Preventive Maintenance Experiment on Route 32, Butte County, California

Route 32, Butte County, California				
Preventive Maintenance (PM) Action	Figure number	Age of PM action	Last preventive maintenance action (year)	Pavement condition
Slurry seal type 3	CA.12a, b, c and d	9	Extensive amount of crack seal (2000)	Fair
Polymer modified chip seal	CA.13a, b, c and d	9	Moderate amount of crack seal (2000)	Good
Control section	CA.14a, b and c	14	Extensive amount of crack seal (2000)	Fair to poor
Conventional chip seal	CA. 14a, b and d	9	Low amount of crack seal (2000)	Good
Boni-fiber overlay over the original chip seal	CA.15a, b, c and d	9	Moderate amount of crack seal (2000)	Good
Asphalt rubber chip seal	CA.16a, b, c and d	9	Low amount of crack seal (2000)	Good
Asphalt-modified chip seal	CA.17a, b, and c	9	Low amount of crack seal (2000)	Good
Emulsion with catalyst chip seal	CA.18a, b, and c	9	Moderate amount of crack seal (2000)	Good
1.2-inch open graded asphalt overlay	CA.19a, b, and c	9	None	Fair with some cracks
1.0-inch gap graded asphalt rubber overlay	CA.20a, b, and c	9	None	Fair with some cracks
1.2-inch dense graded overlay	CA.21a, b and c	9	Low amount of crack seal (2000)	Good

8.3.5 Georgia Department of Transportation Visit

General

The pavement network in the State of Georgia consists of 45,000 lane-miles divided into 2700 lane-mile of concrete and 42,3000 lane-miles of flexible pavements. The annual preventive maintenance budget is about $225,000,000, which represents about 25 percent of the total construction budget. The annual goal of the preventive maintenance program is to resurface about 10 percent of the network.

Preventive Maintenance

The preventive maintenance actions used in Georgia include:

For rigid pavements

- Slab replacement
- Under sealing
- Full-depth repair
- Partial-depth repair
- Joint resealing
- Diamond grinding to correct faulting and ride quality, a faulting index is calculated as the sum of the faulting of five consecutive joints in terms of 1/32-inch.

For flexible pavements

- Microsurfacing
- 1.5-inch overlay
- Milling, an interstate subjected to 3-inch milling would receive 3-inch leveling course plus 1.5-inch surface course. On state roads, the total overlay thickness may be less than the milling.
- Crack sealing
- Slurry seal
- Surface treatment
- Patching
- Chip seal
- Chip seal interlayer and HMS overlay
- Shoulder paving

The preventive maintenance project limits (boundaries) are within the old project boundaries with some flexibility depending on the given constraints. The time span from preventive maintenance project inception to project construction is one to two years with the majority of the projects within one year. Present and past practices indicate that the state or districts do some of the PM works and contractors do the majority.

The Georgia Dot Preventive Maintenance Strategies

The Georgia DOT achieved pavement smoothness overtime and through the consistent implementation of four pavement maintenance strategies that are presented below.

Optimum Timing of Action

The Georgia DOT fixes pavements before they are broken. Projects are selected for rehab or preventive treatments based on the amount of cracking in the surface, not the roughness of the ride. Many states use ride as the sole trigger for repair, or some combination of ride and distress. The Georgia strategy is important because it permits effective use of cheaper repair and preservation strategies. Thin overlays and preventive treatments will not work if the

pavement has severe cracking and bad ride. The important lesson is that while the public notices roughness, fixing pavement before it rides poorly will produce the smoothest pavement network.

Adequate Preventive Maintenance Budget

The Georgia DOT plan is to treat an average of ten percent of the pavement mileage every year to keep the pavement surface in tip-top shape. They do not do expensive pavement reconstruction, where the entire pavement is removed and replaced. To implement a plan similar to this, a state would begin to divert a portion of its pavement reconstruction budget to preventive work-fixing pavements that weren't broken. This could be phased in, but would mean that less expensive band-aid type projects with shorter life cycles might be used on pavements in bad shape to permit better pavements to be preserved. Georgia spends an average of about $200,000,000 on its pavement preservation program, which is about 20 to 25 percent of the total state construction and maintenance budget.

Fast Track Response

The Georgia DOT has established internal procedures to shorten the time between when a project is triggered for treatment and when that project hits the street for construction. Internally, their maintenance people are responsible for contracting out the project needed to do the job, and contracting methods and agency time clocks are set to get the needed repair out to bid within six months. This way the project fixes the distress before it progresses. No more huge over-runs on rehab projects.

Strict Specifications

The Georgia has developed a strict smoothness specification for new pavement and overlays. There is no bonus (incentives), but rough paving must be corrected, removed or diamond ground. This correction requirement is incentive for the contractors to do good work the first time. This tough smoothness specification produces some of the smoothest new pavement in the country. The benefits of preventive work and quick project delivery depend on paving smoothly. The data show that smoother pavements last longer, which contributes to the Georgia objective of providing a high level of service to the public.

8.3.6 Michigan Department of Transportation Visit

General

The operational structure in the pavement areas within the Michigan Department of Transportation (MDOT) can be divided into two categories; centralized and decentralized. For example, pavement management, pavement distress data collection, some of the pavement design and planning activities and some cost estimates are centralized. On the other hand, the selection of pavement projects and type of fixes are decentralized. However, all pavement activities are being streamlined by the increasing utilization of the candidate project list produced by the pavement management system (PMS) based on pavement needs.

It should be noted that the total expenditures of the preventive maintenance program have increased over time from $6,000,000 in 1992 to $60,000,000 in 2000 as shown in Table 8.4 below. The increases were justified based on the benefits and the successes of the preventive maintenance program.

Table 8.4 Expenditures of the Preventive Maintenance Program

Yearly Preventive Maintenance Budget	
Year	**Budget ($)**
1992	6,000,000
1993	16,000,000
1994	16,000,000
1995	16,000,000
1996	24,000,000
1997	25,000,000
1998	54,000,000
1999	60,000,000
2000	60,000,000

MDOT collects pavement surface distress data, longitudinal pavement profile data, rut depth, and friction data. These are detailed below in Table 8.5.

Table 8.5 Distress Types Collected in MDOT

Flexible	Rigid	Composite
Transverse tears Transverse cracks Longitudinal cracks Alligator cracks Block cracks Patches Raveling Flushing Surface treatment type Intensive miscellaneous cracks Rut depth	Transverse joints Transverse cracks Intensive transverse cracks Longitudinal cracks Delamination Reactive aggregate High steel Mudjacked areas Patches Corner breaks Popouts Scaling Surface treatment type	Transverse tears Transverse cracks Longitudinal cracks Block cracks Patches Raveling Flushing Surface treatment type Intensive reflective cracks Rut depth

Preventive Maintenance

The preventive maintenance program in the State of Michigan is considered as one of the most successful program in the nation, if not in the world. Over time, preventive maintenance program guidelines were developed, revised and improved. The January 2000 version of the guidelines is included in the appendix.

The MDOT definition of preventive maintenance is stated in the preventive maintenance program guidelines as follows: "Is the planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserve the system, retards future deterioration and maintains or improves the functional condition of the system without (significantly) increasing structural capacity." The guidelines specify the purpose of the capital preventive maintenance program as follows: "To protect the pavement structure, slow the rate of deterioration and/or correct pavement surface deficiencies. Preventive maintenance actions in Michigan are divided according to pavement type as shown below in Table 8.6.

The MDOT capital preventive maintenance program guidelines clearly state, "Project selection for preventive maintenance is now assisted by using data from the Pavement Management System (PMS)." The guidelines include a set of pavement condition levels for each preventive maintenance treatment. The set of pavement condition levels include the Remaining Service Life (RSL) of the pavement, the Distress Index (DI), the Ride Quality Index (RQI), and the rut depth. These condition levels have been identified to assist the engineer in selecting the optimum project boundary, time of action, and the most cost-effective type of PM work. It should be noted that the established condition levels were and are being calibrated based on the performance and cost-effectiveness of the PM projects.

Table 8.6 Preventive Maintenance Actions for the given Pavement Type

Flexible and composite	Rigid
Non-structural bituminous overlay Surface milling & non-structural bituminous overlay Chip seals Micro-surfacing Crack treatment Over-band crack filling Bituminous shoulder ribbons Ultra thin overlays	Full depth concrete pavement repair Concrete joint resealing Concrete spall repair Concrete crack sealing Diamond grinding Dowel bar retrofit Concrete pavement restoration Bituminous shoulder ribbons Open-graded underdrain outlet cleaning and repair

As stated above, the PMS data are used as a tool to determine the applicability of each preventive maintenance treatment. MDOT has established a preventive maintenance treatment matrix based on experience and pavement needs. The matrix is shown in Table 8.7.

State Road M-99, Crack Seal, and Chip Seal

The 1.8-mile long flexible pavement project on M-99 (Control Section 30033, Job Number 37955, BMP 7.9 and EMP 9.7) was constructed in 1954, surface treated in 1960 and resurfaced in 1979. The pavement was treated with preventive maintenance in 1995. The preventive maintenance treatment consisted of crack sealing and chip sealing.

Table 8.7 Selection of Preventive Maintenance Treatment Based on Pavement Condition and their Benefits

Flexible and composite pavements						
Treatment	**Pavement type**	**RSL (years)**	**Distress Index**	**RQI**	**Rut depth (mm)**	**Life extension (years)**
Non-structural bituminous overlay	Flexible Composite	3 3	<40 <25	<70 <70	<12 <12	5 to 10 4 to 9
Surface milling with non-structural bituminous overlay	Flexible Composite	3 3	<40 <30	<80 <80	<25 <25	5 to 10 4 to 9
Double chip seal	Flexible Composite	5 5	<30 <15	<54 <54	<3 <3	4 to 7 3 to 6
Single chip seal	Flexible	6	<25	<54	<3	4 to 7
Micro-surfacing (multiple)	Flexible Composite	5 5	<30 <15	<54 <54	<25 <25	4 to 6 No data
Micro-surfacing (single)	Flexible	10	<15	<54	<25	3 to 5
Crack treatment	Flexible Composite	10 10	<15 <15	<54 <54	<3 <3	Up to 3 Up to 3
Over-band crack filling	Flexible Composite	7 7	<20 <10	<54 <54	<3 <3	Up to 2 Up to 2
Ultra-thin bituminous overlay	Flexible Composite	7 7	<20 <10	<54 <54	<3 <3	No data No data
Rigid pavement						
Treatment		RSL (years)	Distress Index	RQI		Life extension (years)
Full depth concrete pavement repair		7	<20	<54		3 to 10
Concrete joint resealing		10	<15	<54		3 to 5
Concrete spall repair		10	<15	<54		Up to 5
Concrete crack sealing		10	<15	<54		Up to 3
Diamond grinding		12	<10	>54		3 to 5
Dowel bar retrofit		10	<15	<54		2 to 3
Concrete pavement restoration		3	<40	<80		7 to 15

Figure 8.5 shows the pavement distress data for the period 1992 to 1998. The solid line represents the best-fit curve while the dashed line represents the most likely scenario of the pavement performance. On the PMS distress data). Examination of the data in Figure 8.5 indicates that:

- The PMS distress data can be used to track the performance of preventive maintenance actions.
- Preventive maintenance improves the pavement condition, decreases the pavement rate of deterioration and increases the service life of the pavement.
- Preventive maintenance benefits can be estimated using the PMS distress data.

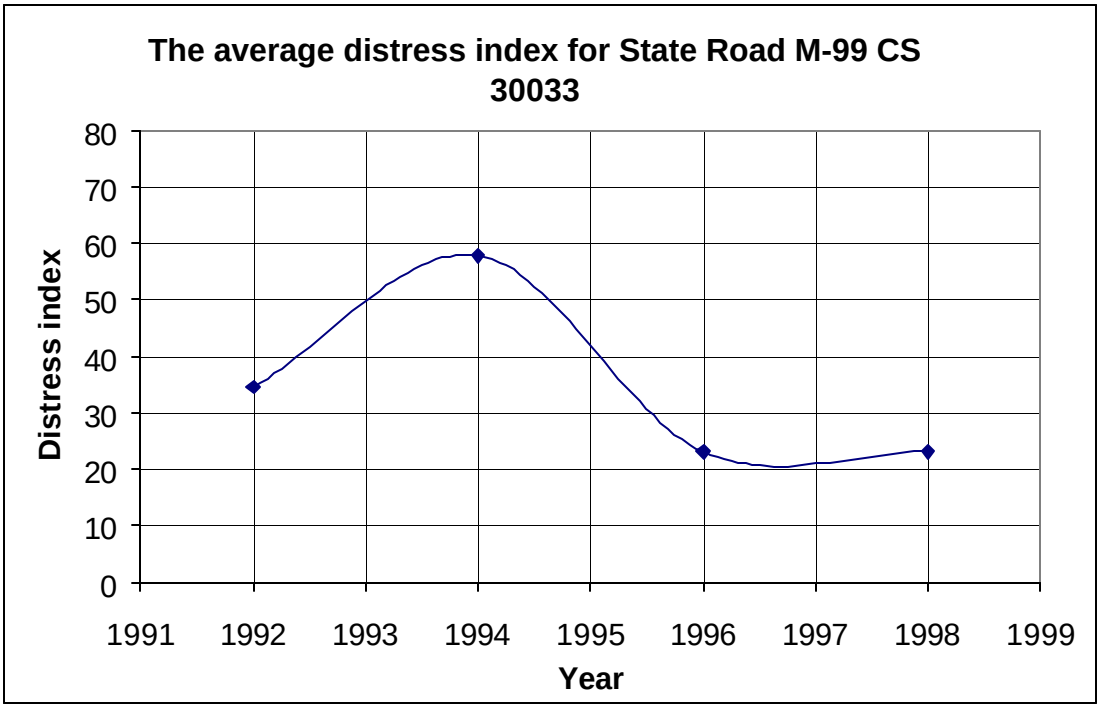

Figure 8.5 The Average Distress Index for State Road M-99

State Road 124 (M-124), Nonstructural Bituminous Overlay

Figure 8.6 shows the average distress index calculated along the project. Once again, the solid line presents the best-fit curve between the data points (Michigan collects distress data every other year). The dashed line presents the most likely scenario of the trend of the distress index.

Figure 8.7 depicts the average ride quality index (RQI) along the project. Once again, it can be seen that the RQI decreases from about 62 (fair ride) to about 36 (good ride) after construction and it increases to 39 (good ride) two years after construction.

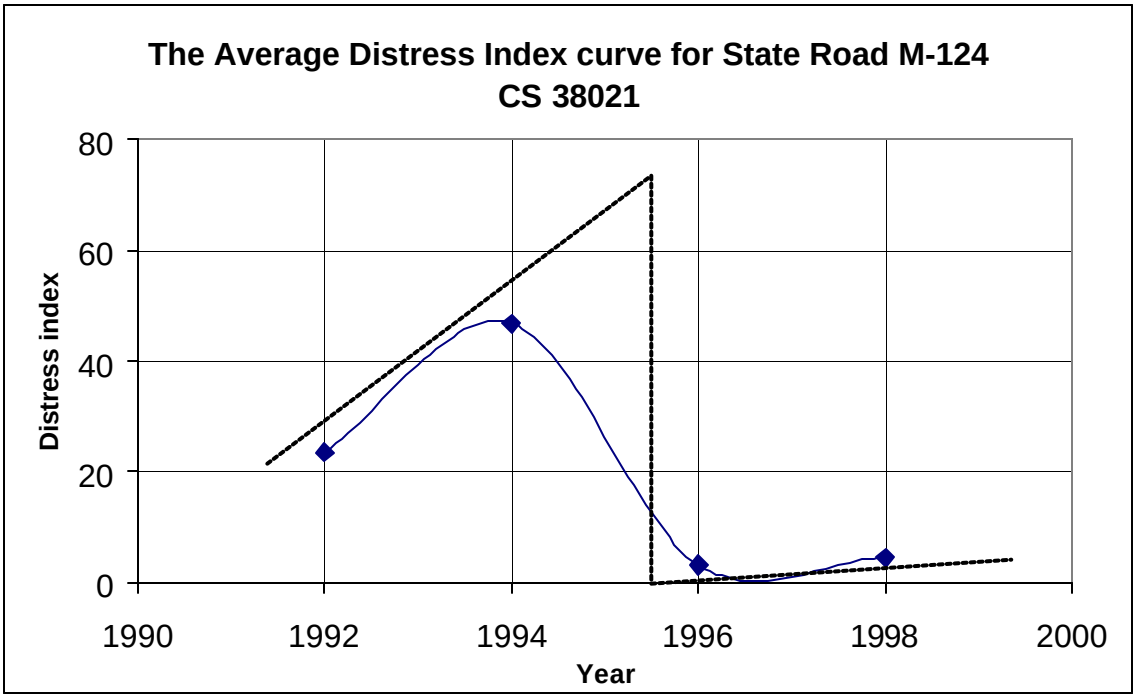

Figure 8.6The Average Distress Index Curve for State Road M-124

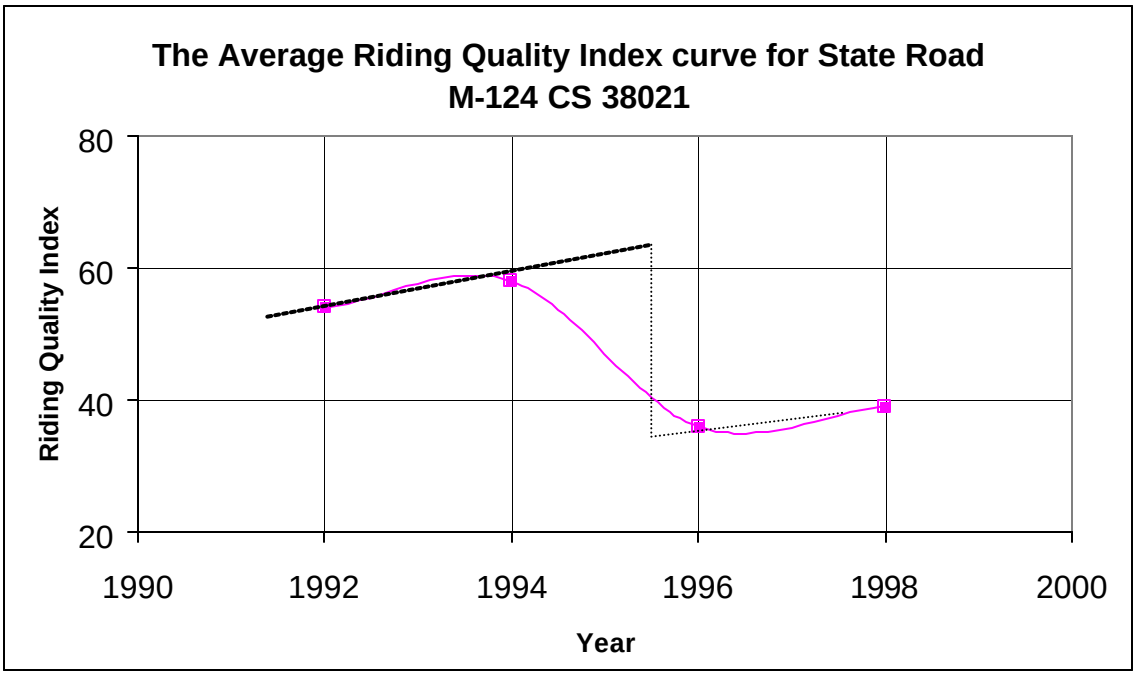

Figure 8.7The Average Riding Quality Index Curve for State Road M-124

8.3.7 Pennsylvania Department of Transportation Visit

General

The Pennsylvania Department of Transportation (PennDOT) divides the state into 11 districts. The operational structure in the pavement areas within the Pen DOT can be divided into two categories; centralized and decentralized. For example, pavement design and planning, cost estimates, pavement management, and pavement distress data collection are centralized. On the other hand, the selection of pavement projects, pavement designs, and type of fixes are decentralized and conducted at the district level (note that project pavement designs are generally performed at the Engineering District level, the Central Office reviews designs, establishes design policy, and provides support to the Districts). However, the two activities are being streamlined by establishing guidelines and policies regarding the selection of project boundaries and treatment type and by using the pavement management system (PMS) distress data. The streamlining has (overtime) decreased arbitrary project and treatment type selection.

Pavement Preservation

The pavement preservation program in the State of Pennsylvania consists of three components:

- Reconstruction
- Rehabilitation
- Preventive maintenance

In addition, the Pen DOT conducts routine maintenance activities such as crack seal, lane demarcation and guardrail enhancement.

Interstate Highway 78, Berks County

The asphalt concrete overlay on the composite pavement along IH78 (starting at segment number 100) was about 20 years old when it was maintained in 1997. The PMS distress data indicated that most joints in the concrete pavements are in very good to excellent condition and few joints are in need of full depth repair. Based on the PMS distress data, the following preventive maintenance actions were recommended:

- Mill the existing asphalt concrete
- Test all joints in the concrete pavement using FWD to determine the load transfer efficiency of each joint.
- Repair deteriorated joints along the project using full depth concrete patches.
- Place and compact an asphalt concrete overlay.
- Make single saw cuts in the asphalt concrete as to match the joints in the concrete pavement.
- Seal all saw cuts in the asphalt concrete.

Because of various constraints, the district office decided to do half of the project as it was recommended and the other half to eliminate the FWD testing and the full-depth joint repairs. Views of the pavement and a close-up of the pavement condition at a transverse joint about three years after construction. A visit showed both sections of the pavement and the sealed single saw cut made in the asphalt concrete are in perfect condition.

8.4 DISTRICT LEVEL INDEX TO SELECT TXDOT PROJECTS FOR PREVENTIVE MAINTENANCE

8.4.1 Background

Preventive maintenance, as defined by TxDOT, is often a thin layer or surface treatment placed to preserve the existing pavement structure. Maintenance forces make the required deep repairs in the surface and base before contracting the resurfacing projects outside the department.

The Fort Worth District of TxDOT uses the Texas Pavement Management Information System (PMIS) in combination with local area engineer's recommendations to select projects. The annual district allotment from the preventive maintenance program is approximately $7,000,000 for an eight county area that includes 8,100 lane miles of pavement [WIMSATT 96].

Most Texas pavement engineers consider it good practice to resurface an asphalt pavement every seven to ten years so as to minimize the distressed condition. Unfortunately, available funding does not always allow this timing and many pavements fail early and the maintenance funds are often spent there, rather than on true preventive maintenance. The TxDOT pavement management systems (PMS) selects projects based on pavement condition. Because there is a difference between the idea of performing preventive maintenance before distress occurs and the realism of limited funding, distress does occur on most projects. There, a new tool is needed to select the most critical projects for funding. The Forth Worth District of TxDOT uses for selecting projects, an index that includes both age and condition.

8.4.2 Project Selection Process

Each area engineer in the district manages approximately 1,000 lane miles of pavement within a given boundary (usually a county). Each year each area engineer submits a list of candidate projects to the district office. Each area engineer provides current cost estimates and recommends a priority for each of his projects. The district office, primarily the district PMS engineer, then selects the projects to be funded using the following index to aid in the selection of preventive maintenance projects.

$$Index = (Length / Cost) \times (LF) \times (age) \times (\% Needs)$$

where:

Length is	length of two-lane roadway in the project in miles
Cost	in dollars
LF	Lane factor (1.0 for two lanes and 2.0 for multi-lanes)
age	is the time since last treatment except under age of five the value is set to the value 1
% Needs	is the percentage of the pavement section requiring maintenance according to the PMIS.

In the process only projects with priorities 1 through 9 submitted by each area engineer are considered for funding. Pavements with an age of less than five years are given a low priority by setting the age to 1.0 in the equation for the index. The district engineer reserves the right to override the index with judgment but this is done very sparingly.

Key features in this process are the area engineers' cost estimates and priority recommendations along with current condition data from the PMIS. New condition data is required for shoulders, ramps or other auxiliary lanes that are not normally surveyed by the network PMIS. When PMIS data conflicts with high priority projects submitted by the area engineer, the pavement is re-inspected by the district pavement engineer.

8.4.3 Summary of Program Results

Table 8.8 lists the top seven projects out of the 126 projects submitted by the area engineers. As it turned out, only 32 projects can be financed with $7,000,000. The district office overrode the calculated index for the first three projects where the maximum index of 1.000 is shown. These three projects represent 15 miles out of 240 miles funded or about 5%.

Table 8.8 Priority List for Index (partial)

Description	Length Miles	Age Yrs	% Needs	Index	Cost '000$	Cum. '000$
Seal Coat	4.3	9	4	1.0	52.1	52.1
Seal Coat	6.6	12	4	1.0	148.1	200.2
Seal Coat	4.1	15	1	1.0	47.0	247.2
Seal Coat	6.5	11	50	0.069	103.3	350.5
SC+Fog Seal	12.4	11	36	0.028	174.2	524.7
Slurry Seal	5.8	7	91	0.024	309.5	834.2
SC+Latex	4.0	8	30	0.013	72.8	907.0

8.4.4 Findings

Some field engineers in TxDOT feel that the statewide PMIS does not answer all problems in programming preventive maintenance; therefore they have not bothered to use it. This example shows how the PMIS data can be used and is being used in combination with the local detailed analysis that may be more precise on specific projects. This real life example shows how a PMS can be used to make important engineering decisions even though the PMS database may not be the complete solution.

8.5 WISDOT'S PREVENTIVE MAINTENANCE STRATEGIES FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

8.5.1 Introduction

Continuously reinforced concrete pavements (CRCP) have been used exclusively on high volume roads in Wisconsin, their total length is 475 miles, representing 3.5% of the total statewide mileage.

Wisconsin has two generations of CRCP in existence today. The first generation was built with "black" (non-coated) steel in the 60's and the 70's. The second generation was built between 1984 and 1988 and utilized epoxy-coated (green) steel to prevent corrosion of the steel.

The information for this section has been subtracted from [WisDOT 96].

8.5.2 WISDOT PMS Overview

The Pavements Section of the Bureau of Highway Construction, Division of Transportation Infrastructure Development, collects, manages and analyzes pavement performance data on all State Trunk Highways (STH) or State Trunk Network (STN) for the Wisconsin Department of Transportation.

WisDOT is charged with ensuring that the state's highway network meets the transportation needs of the public. This has been established through a strong working relationship with the paving industry and the counties. This relationship enables the state to maintain and improve more than 13,700 roadway miles (two lanes) of state trunk highways.

WisDOT has been using seven basic pavement types, representing 52% asphalt pavements, 22% concrete pavements, and 26% composite pavements. These pavement types and a brief description are given in Table 8.9.

Table 8.9 Summary of Pavement Type Classification

Pavement Type	Description
1. AC/FB	Asphaltic Concrete over Flexible Base
2. ARM	Asphaltic Road Mix
3. AC/RB	Asphaltic Concrete over Rigid Base
4. JRCP	Jointed Reinforced Concrete Pavement
5. JPCP	Jointed Plain Concrete Pavement without Dowels
6. CRCP	Continuously Reinforced Concrete Pavement
8. JPCD/d	Jointed Plain Concrete Pavement with Dowels

Type 1 pavements (AC/FB) are most commonly used. Any pavement with an asphalt hot mix surface, single or multi-layered, above crushed base courses or other granular bases is part of this category. The methods of construction and of asphalt mix design went relatively unchanged from their early beginnings until the late 70's. The first major change in AC technology in Wisconsin took place in the early 80's when the department began recycling asphalt pavements. Originally the durability of those pavements was less than expected, but since that time recycled pavements have evolved to their present-day status. The next major innovation, prior to Superpave, was the development of Rut Resistant Hot Mix Asphalt (RRHMA) pavements in 1989.

Type 2 pavements (ARM) are composed of aggregates and a cold asphalt binder, formerly cutbacks, now emulsions. Roadmix pavements are generally used on low traffic roads.

Type 3 pavements (AC/RB) are composed of AC over a rigid base, which can be an existing concrete pavement, or a cement-stabilized, crushed aggregate base course.

Type 4 pavements (JRCP) are generally made with wire mesh reinforcement, doweled joints, and 80 foot spacings. They were built from the 50's to the early 70's. Most of the Interstates were built using this system.

Type 5 pavements (JPCP) are unreinforced concrete pavements with short joint spacings and without dowels. The first generation was built in the 40's and typically had a 25 foot spacing. The second generation was built between 1972 and 1987 and had either a 20 foot normal, or a random skewed (13'-19'-18'-12') spacing.

Type 6 pavements (CRCP) are continuously reinforced concrete pavements; the first generation was built with "black" steel in the 60's and 70'. The second generation was built between 1984 and 1988 and used epoxy-coated (green) steel to prevent corrosion. These pavements are used exclusively on high volume roads.

Type 7 pavements do not (yet) exist.

Type 8 pavements (JPCD/d) are similar as type 5, but do have dowels. This pavement has been the department's standard for rigid pavement construction since 1987. It has a random-skewed joint spacing of either 13'-18'-19'-12' or 17'-19'-20'-18', depending on thickness.

The current pavement design methodology can be summarized as following. For new construction, current practice is to design alternative pavement structures using the AASHTO design methods. The alternatives generally consist of asphalt pavement over a flexible base (Type 1) and jointed plain concrete pavements with dowels (Type 8). Associated with the structural design of concrete pavements, equivalent single axle loads (ESAL's) are used to determine if incorporating an open graded base course will drain the pavement structure. After the alternative pavement structures are designed, a 50-year life cycle cost analysis (LCCA) is performed to determine the most cost-effective alternative. This analysis assumes:

1. A standard sequence of maintenance and rehab activities;
2. A 5% discount rate; and
3. No user delay costs.

The most important factor in evaluating any product is how well it performs. If a pavement does not perform, other factors such as cost have little meaning. Pavement performance is generally based on smoothness of ride and structural integrity. Historically, the Present Serviceability Index (PSI), based on roughness measured in inches/mile, has measured ride quality in Wisconsin.

Since 1990, the International Roughness Index IRI is used as measure of ride quality. IRI is calculated in meters/kilometer.

Structural integrity is measured by observing various pavement distress elements (cracking, rutting, flushing, patching, raveling, distortion, faulting, etc.) which are combined into the Pavement Distress Index (PDI). A PDI value above 70 is an indication of considerable pavement distress. This value is often used as threshold for improvement projects on major highways. Districts use values above 80 as threshold for improvements on district-level roads. WisDOT uses both ride and structural integrity to assess pavement quality.

8.5.3 Performance of CRCP Pavements

The main performance indicators used by WisDOT are the Present Serviceability Index (PSI), the International Roughness Index (IRI), and the Pavement Distress Index (PDI). PSI and IRI are both related to the measured roughness of the pavement, whilst PDI is related to the structural integrity of the pavement, including distresses such as cracking, distortion, raveling, faulting, etc. Details about these indexes are given in Chapter 4.5.

The average PSI for all CRCP pavements over a period of 15 years was 4.0, which can be interpreted as an outstanding ride level. The statewide average over that period was 3.2.

The average PDI for all CRCP pavements over a period of 15 years was 43.8, which can be interpreted as a Fair rating. The statewide average over that period was 37, which means a Good rating. This relatively poor structural performance of CRCP was the main reason for the study.

Table 8.10 gives an overview of the reasons why CRCP pavements were rehabilitated (under the heading Constraints) and lists the condition of these pavements prior to rehabilitation. It also gives the Service Life of CRCP with epoxy coated reinforcing bars as established by WisDOT's Pavement Maintenance and Rehabilitation Policy Committee. This table shows that surface distress is the primary reason for rehabilitation. As indicated earlier, there are two era's for CRCP pavements, "black steel" and "green steel". The average age at which these rehabilitations were performed, 18+ years for 75% of CRCP pavements, implies that the majority of the pavements were for "black steel" pavements. Distresses such as punch-outs and surface distress characterize "black steel" pavements. Also note the low percent of pavements that were rehabilitated because of a poor ride. This emphasizes the fact that CRCP provides good ride throughout its life.

Table 8.10 Constraints/Distress Thresholds and Service Life for CRCP

CONSTRAINT	AVG AGE	AVG PSI	AVG PDI	Dominant PDI Indicators
0.7% Due to only PSI	10.0	1.38	33	Punch-outs
75.4% Due to only PDI	18.4	3.44	90	Surface Distress
4.5% Due to Both	18.2	2.14	89	Wide Cracks
19.4% Due to Neither	16.7	3.30	56	*Service Life: 25 years*)*

*) Based on the use of epoxy coated reinforcement

8.5.4 Historical Trends for CRCP Pavements

The historical trends for the PSI and the PDI levels of CRCP pavements are presented in Figures 8.8 and 8.9 respectively.

The service life, indicated by the vertical dotted line, is 25 years (see Table 8.9). Some 98% of the data used in the analysis fall within the service life. Of these data 83% are for CRCP with "black steel" prior to 1985, and 17% are for "green steel" since that date. Both the "black" and the "green" steel pavements demonstrate outstanding performance for PSI. The PDI trend for CRCP pavements is given in Figure 8.8.

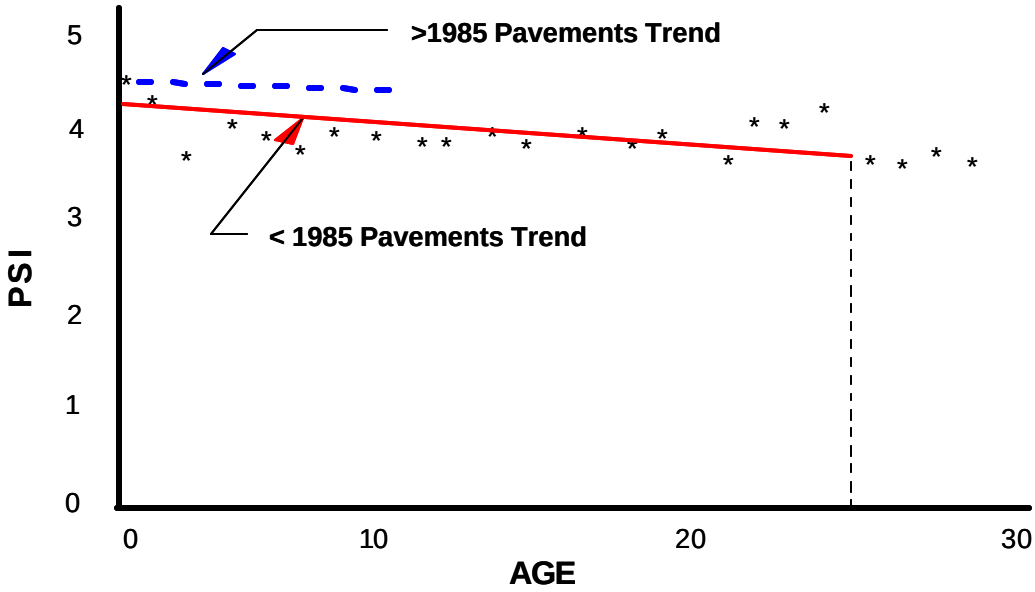

Figure 8.8 Ride versus Age for CRCP Pavements

For the period before 1985, the "black steel" trend shows that the rate of deterioration is greatest between ages three and ten. WisDOT believes that this occurs because by age three, cracks have formed, and the steel reinforcement begins to corrode. This corrosion leads to structural problems, such as punch-outs and delamination.

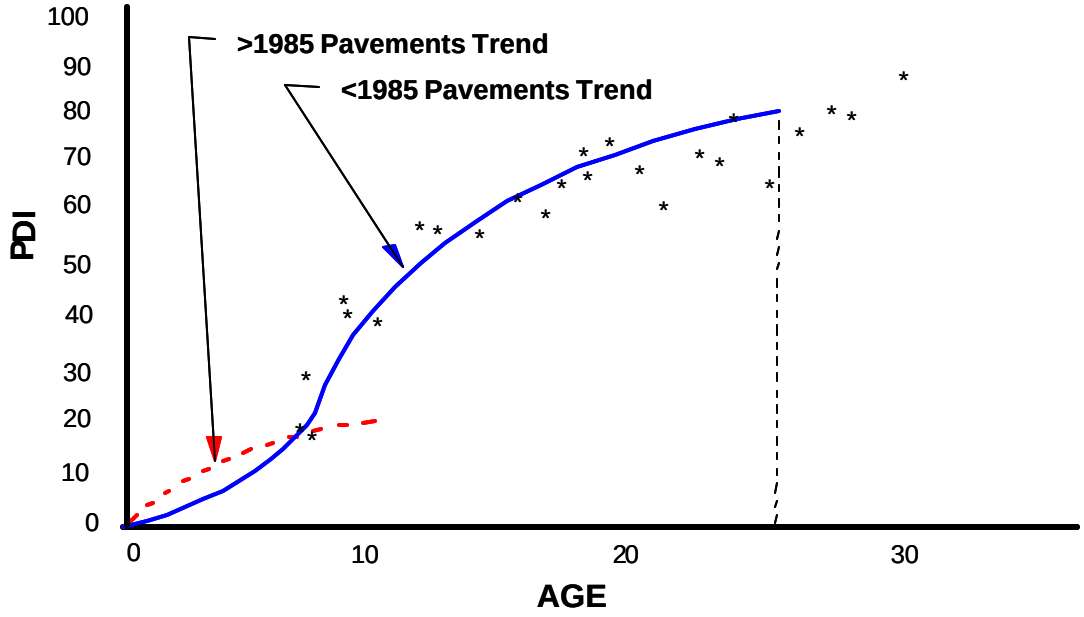

Figure 8.9 Distress versus Age for CRCP Pavements

The "green steel" PDI trend shows less distress development. Review of individual pavement distress indicators reveals that these sections have less punch-outs and surface distress, but more diagonal cracking than "black steel"

sections through the first 10 years of life. Punch-outs and surface distress can be traced back to corrosion of the steel reinforcement, while diagonal cracking can be linked to local base failures.

8.5.4 Preventive Maintenance and Rehabilitation Techniques for CRCP

Most of the maintenance and rehabilitation efforts for CRCP pavements is targeted at the "black steel" pavements, constructed prior to1985. In 1987 a research study was carried out to test rehabilitation techniques and preventive maintenance for distressed CRCP. Several rehabilitation projects were constructed in 1988. The methods of rehabilitation used were:

- Intensive and thorough concrete super-patching,
- Asphaltic Concrete overlays of various thicknesses,
- PCC bonded thin overlays,
- Use of impermeable membranes in combination with AC overlays,
- Rubblize the existing CRCP, compact it, and place an AC overlay.

The methods of preventive maintenance were:

- Cathodic protection of the black steel rebar mat
- The use of a corrosion inhibitor in deicer salts.

The thin asphalt concrete overlay rehabilitations were the most successful and are now recommended for use by WisDOT as the preferred rehabilitation strategy.
The preventive maintenance part of the study concluded that cathodic protection and corrosion-inhibiting deicers were ineffective in reducing distress.

8.5.5 Conclusions

- Continuously Reinforced Concrete Pavements in Wisconsin provide an exceptional ride and will last at least 25 years, provided the reinforcing steel is protected with an epoxy coating,
- Thin asphalt concrete overlays are the preferred rehabilitation treatment for CRCP pavements showing distress as a result of corrosion of the reinforcing steel,
- Cathodic protection of the steel and the use of corrosion inhibitors in deicing salt are not effective in reducing distress.
- For these studies the PMS database forms an essential element for the monitoring and prediction of pavement performance.

8.6 REFERENCES TO MODULE 8

[AASHTO 98] AASHTO Strategic Highway Safety Plan – A Comprehensive Plan to Substantially Reduce Vehicle-Related Facilities and Injuries on the Nation's Highways. AASHTO 1998, Washington, DC.

[AASHTO 93] Guide for Design of Pavement Structures, AASHTO 1993, Washington, DC.

[AASHTO 87] Maintenance Manual. AASHTO 1987, Washington, DC.

[Baladi 01] The Role of PMS in Tracking the Performance of Preventive Maintenance Actions, Final Report, by Gilbert Y. Baladi, March 2001, carried out for the FHWA, Washington, DC.

[FHWA 99] Pavement Preservation: A Road Map for the Future. FHWA-SA-99-015. Forum held October 26-28, 1998, Kansas City, MO.

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[FedReg 89] Federal Register. 1/13/89. 23 CFR Part 626. Washington, DC.

[Geoffroy 96] Geoffroy, D. N. 1996. NCHRP Synthesis of Highway Practice 223: Cost Effective Preventive Pavement Maintenance. Transportation Research Board, Washington, DC.

[Wimsatt 96] Meetings on September 18, 1996 with Andrew Wimsatt, Pavement Management Engineer, Fort Worth District, TxDOT.

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9 PAVEMENT PRESERVATION STRATEGIES

9.1 INTRODUCTION

This chapter will discuss examples of engineering analysis in the area of pavement preservation and rehabilitation, following an overview of this subject. The examples come from DOT's in Pennsylvania, and Montana. Since the State of Montana has not featured yet in this Notebook, their example will be preceded by a brief discussion of their PMS activities

9.2 OVERVIEW OF PRESERVATION AND REHABILITATION

With the maturation of the highway system in many countries, the emphasis in highway construction is shifting from new facilities to the maintenance and restoration of existing facilities. Thus, methods for the design of rehabilitation of existing pavements are now critical in pavement management.

As pointed out in previous sections, pavement management covers the full life cycle of the pavement, including the performance period following rehabilitation. The methods described in this section are subsystems of a project level pavement management program. Such rehabilitation subsystems are used in the System Analysis Method for Pavements (SAMP) [Lytton 75], the Rigid Pavement Design System (RPS) [Hudson 72], and the Flexible Pavement Design System (FPS) [Scrivner 68] among other project level pavement methodologies.

9.2.1 The AASHTO Guide's Structure for Rehabilitation Design

The life-cycle of a pavement involves several types of behavior and different levels. The initial stage is typified by good performance with low maintenance required. As the pavement ages, the need for maintenance increases as more of the pavement surface is affected by distress that affects the ride quality. At some point in the pavement's life, the condition of the surface deteriorates to the point where routine maintenance is no longer cost-effective and major treatment is needed to restore the quality of the pavement. The most economical rehabilitation treatment will depend on the condition of the pavement, anticipated traffic loads, and environmental conditions. Rehabilitation design is a complicated process because the engineer is faced with several options and sometimes with incomplete data on the pavement section. *The AASHTO Guide for Design of Pavement Structures* states,

> "A considerable amount of both analysis and engineering judgment must be applied to each project. Due to state of the art limitations relative to the entire rehabilitation process, a definite need exists for continuous feedback from agencies on the performance of various rehabilitation methods."

Figure 9.1 shows the overall process for the selection of a rehabilitation strategy [AASHTO 86]. This is a three-step process, define the problem, examine potential solutions, and select the preferred solution. Similarities should be noted between these steps and the overall systems method. The problem definition process essentially establishes the environment for solutions. Defining the problem requires data collection and analysis of the condition of the existing pavement. Generating potential solutions requires generating and analyzing alternative approaches to the problem of rehabilitating the pavement. Based on the results of the preliminary analysis, the detailed design is performed and the solution implemented.

The *AASHTO Guide for Design of Pavement Structures* broadly classifies pavement rehabilitation into overlay methods, non-overlay methods, and complete reconstruction. The design procedures for new pavements are used for complete reconstruction projects. Overlay design methods determine the amount of additional pavement structure required to serve the future traffic. The major non-overlay methods include:

- Full depth repair
- Partial depth patching
- Joint and crack sealing
- Subsealing and undersealing
- Grinding and milling
- Subdrainage
- Pressure relief joints
- Load transfer restoration
- Surface treatments.

The AASHTO Guide methodology is presented here because it is widely known and has been widely examined by state DOTs. That methodology has been replaced based on an NCHRP study completed in 1992 [Darter 92a, 92b]. Thus, nothing herein is intended to support or deny the specifics of the 1986 AASHTO methodology. The chart and the method, however, do serve to illustrate the complete aspects of a rehabilitation method.

In considering the data factors, the economic analysis, and the life-cycle cost analysis to be used in rehabilitation, it is essential that the data and the methodology be compatible with the data and methodology available in the project level pavement management system. It cannot be over emphasized that the rehabilitation design methodology used in project level pavement management must be compatible with the balance of the pavement management system.

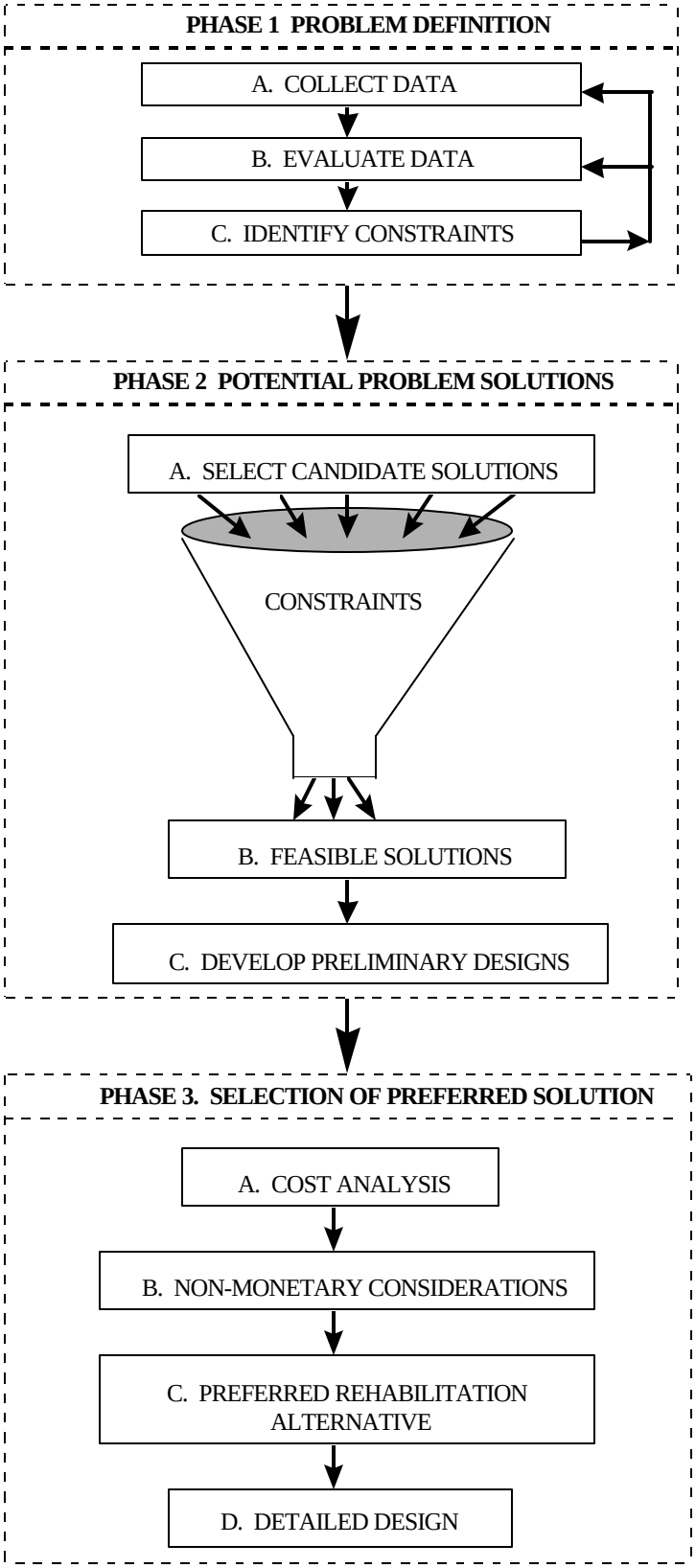


Figure 9.1 AASHTO Guide Structure for Rehabilitation Design [AASHTO 86]

9.2.2 Rehabilitation Design Inputs and Field Data Required

The design of a pavement rehabilitation project starts with the collection of data about the condition of the project. If the agency has a network level pavement management system, a summary of required information may be available from the database. However, it will probably be necessary to supplement this information with data taken from the specific project under review.

The AASHTO Guide includes data collection guidelines for evaluating pavement rehabilitation projects [AASHTO 86]. In addition, the Federal Highway Administration developed a training course on pavement rehabilitation [ERES 87] that contains valuable information on pavement repair and rehabilitation methods.

The AASHTO Guide defines several elements of data collection for rehabilitation design:

Delineation of analysis units
Drainage evaluation
Pavement distress survey
Nondestructive testing
Field sampling and testing

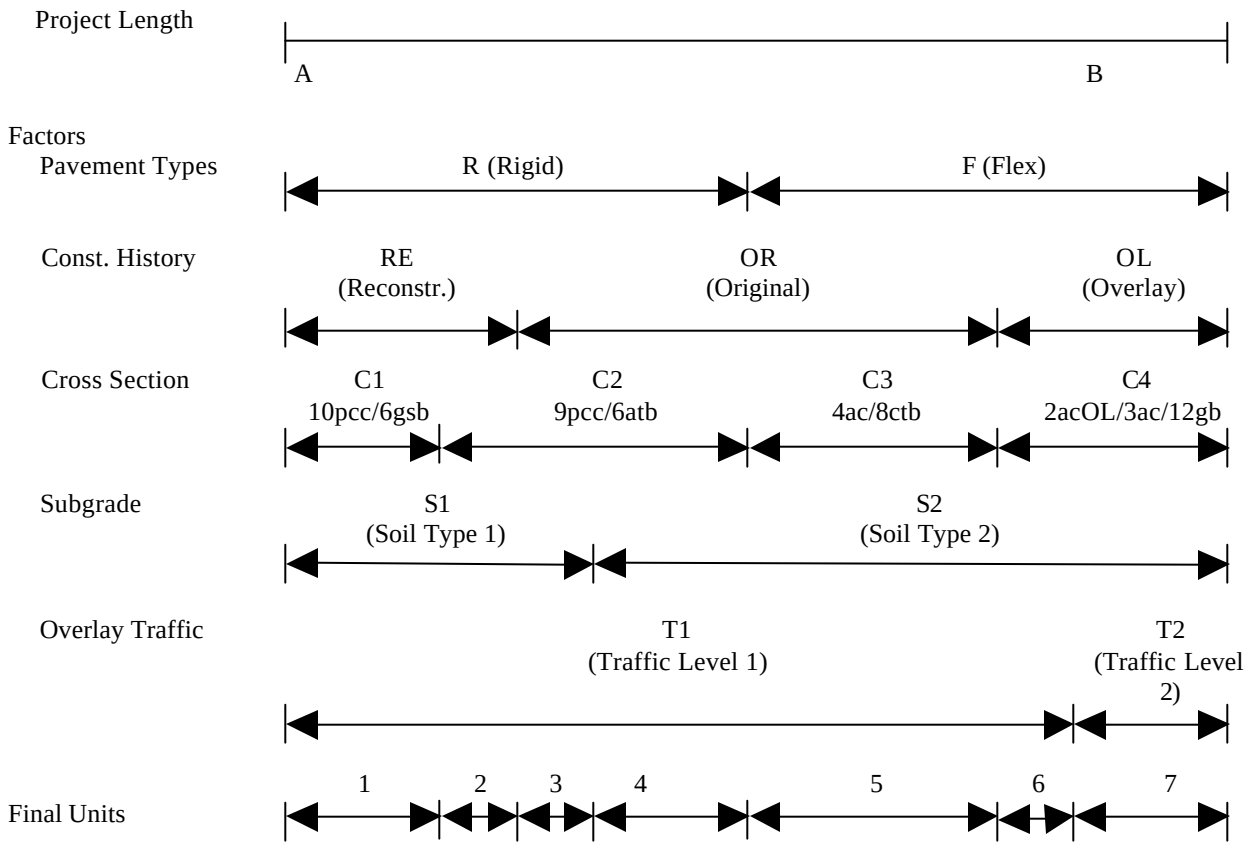
As previously discussed, the data collection requirements for project level work are more intense than are required for network level analysis.

9.2.3 Delineation of Analysis Units

Frequently in large pavement rehabilitation projects, the properties of the pavement will vary along the length of the project. If the variations in these properties are not recognized in the selection of the rehabilitation alternative, then parts of the project will be over or under designed. Thus, it is important to isolate unique factors that influence potential pavement performance into separate sections called analysis units. The factors that can be used for delineation include:

1. Pavement type
2. Construction history, including rehabilitation and major maintenance
3. Pavement structural characteristics, layer material type, and thicknesses
4. Subgrade soil type
5. Traffic
6. Pavement condition

Figure 9.2 demonstrates how these factors can be used to separate a project into analysis units. Historical records can be used to assist in selecting analysis units. Generally, the subgrade support conditions are the most difficult factor to assess.



Unit Attributes:

Unit Number	1	2	3	4	5	6	7
Pavement Type	R	R	R	R	F	F	F
Construction Hist.	REC-5	REC-5	OR-15	OR-15	OR-15	OL-20	OL-20
Cross Section	C1	C1	C2	C2	C3	C4	C4
Subgrade	S1	S1	S1	S1	S2	S2	S2
Overlay Traffic	T1	T1	T1	T1	T1	T2	T2

Figure 9.2 Defining Analysis Units for Rehabilitation Projects

Even in areas with uniform soil types, other factors such as drainage, compaction, cut and fill sections, and topography can cause variations in the subgrade soil. Usually the variability in subgrade support should be assessed with deflection measurements taken along the length of the pavement.

Deflection and other pavement response variables, such as friction or distress surveys, can be used to define boundaries in the pavement condition using a variety of methods. The first step is to plot the response variable versus the distance along the pavement as shown in Figure 9.3. The analysis units can then be identified

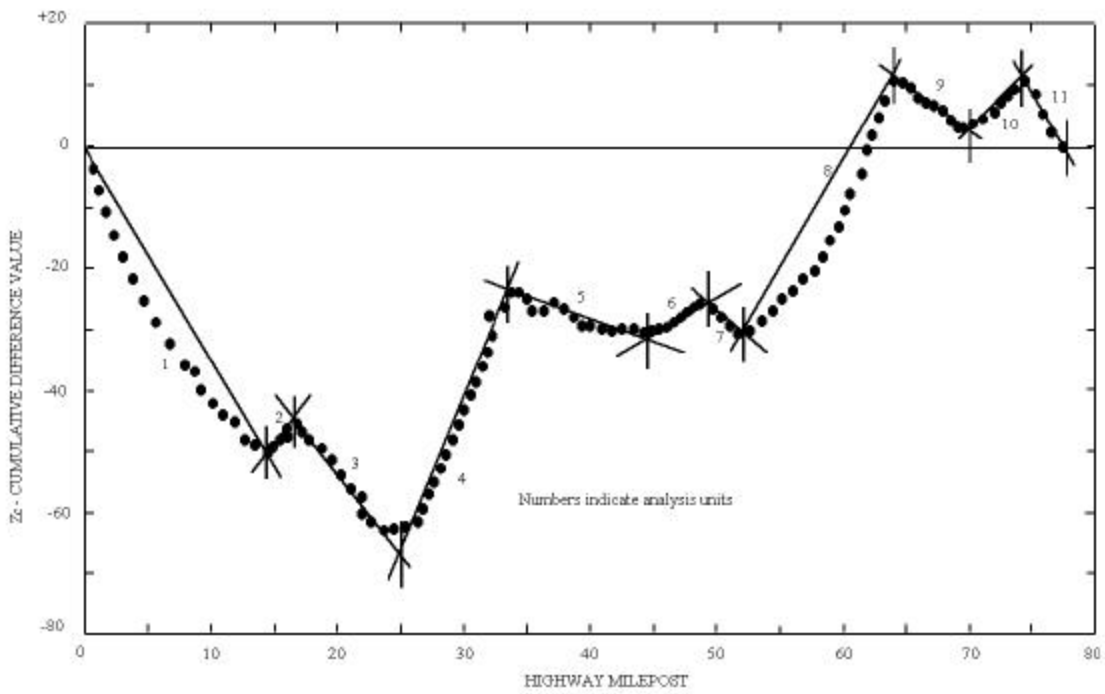


Figure 9.3 Cumulative Difference Approach for Establishing Analysis Units [AASHTO 86]

subjectively or with an analytical procedure such as the cumulative difference method recommended in the AASHTO Guide. In this method, the response variable is transformed or normalized into a new parameter, Z_C, defined as the difference between the area under the response curve at any distance and the total area developed from the overall project average response at the same distance. The transformed variable is plotted versus the distance along the project and points where the slope of the resulting curve changes sign define the boundaries of the analysis units, as shown on Figure 9.3. Once the analysis units have been established through a consideration of the above factors, the engineer needs to review the location of the boundaries to ensure the sections are logical. Sections may need to be combined for practical construction considerations or economic reasons.

9.2.4 Drainage Evaluation

Moisture in a pavement structure is a major contributor to pavement deterioration. All rehabilitation projects should consider the drainage of the pavement structure. Distresses in flexible pavements that are caused or accelerated by moisture include stripping, rutting, depressions, fatigue cracking, and potholes. Distresses in rigid pavements that are moisture-related include D-cracking, joint deterioration, faulting, and corner breaks. Freeze-thaw, differential frost heave, and spring breakup all indicate the pavement structure retains excessive moisture. Sometimes, the need for drainage improvements is related to excessive pavement distress. However, even in the absence of pavement distress, the drainage features of the existing pavement should be evaluated and any needed corrections identified.

The factors that should be evaluated in a drainage survey include the existing drainage features, topography, permeability of the pavement materials and subgrade soil, and the climatic zone. There are several excellent

references on drainage surveys and drainage design [Cedergren 74] [Moulton 80]. The pavement manager should appreciate the importance of drainage and the need to include drainage considerations in the design of a pavement rehabilitation project.

9.2.5 Detailed Distress Survey

Distress surveys have been previously defined as an important feedback data item related to pavement management. Once a project section for project level pavement management or rehabilitation is selected, it will be necessary to conduct a detailed distress survey. One of the more detailed methodologies should be adopted and a method of summarizing the data or evaluating possible design response to the observed distress types should be selected. More details related to rehabilitation selection based on the distress type can be obtained from various sources [Yoder 75] [AASHTO 86].

9.2.6 Nondestructive Pavement Evaluation

Nondestructive pavement evaluation through deflection measurements is an extremely valuable and cost effective tool for determining the structural condition of an existing pavement. For the design of rehabilitation projects, deflection evaluations provide information on the structural capacity of the existing pavement for overlay design, the location of differences in the subgrade characteristics and voids and load transfer problems in rigid pavements. The sampling plan is an important feature of the nondestructive pavement evaluation. The engineer must make a tradeoff between the cost of the data collection and the number of test locations required for the design of the pavement rehabilitation strategy. The number of samples depends on the design situation. For flexible pavements on a relatively uniform subgrade soil, the Arizona Department of Transportation collects five deflection measurements per mile. Other agencies collect as many as 20 deflection measurements per mile. Agencies investigating void location and joint load transfer of rigid pavements may need to take two or more measurements per slab.

9.2.7 Field Sampling and Testing

Even with an effective nondestructive testing program, it will often be necessary to collect some cores of the pavement structure. These are required for verifying the thicknesses of the pavement layers and for providing samples for laboratory analysis. The extent of the sampling program required depends on the condition of the existing pavement. If the pavement is in relatively sound condition, then the majority of the information required can be determined with historical records and nondestructive testing. However, if the pavement is displaying a materials related problem, such as stripping or major rutting, then a more thorough analysis of the materials may be required.

9.2.8 Performance or Deterioration Models

The ability to design a rehabilitation strategy depends on the availability of models for predicting the performance of the rehabilitation strategy. Without knowledge of the performance of the rehabilitation strategy, the life of the repair cannot be estimated and, therefore, the cost-effectiveness cannot be compared to other alternatives. Unfortunately, there is not a reliable set of performance models for rehabilitated pavements that are universally accepted. Whereas the AASHO Road Test provides us with a great deal of information on new pavements, there is no comparable data set for rehabilitated pavements. Thus, most of the models used in rehabilitation performance prediction are based on modified road test equations or on sketchy observations of small experiments in the field. The current LTPP studies are developing a new database that includes rehabilitated pavements. This will eventually provide a fundamentally new set of detailed information to use as a basis of rehabilitation performance models. In addition, some states have developed an historical PMS database with extensive performance data for rehabilitated pavements. The case study for Pennsylvania presented in Module 6 is a prime example of this. New tools such as the Visual Modeler [TRDI 97] software system are available in the public domain to help analyze historical PMS data to develop such models for rehabilitation.

Restoration Performance Models

Restoration is generally a term that is associated with rigid pavements and the restoration process is one that has been developed by the industry that involves under-sealing, joint repair, and slab replacement. While we call the reader's attention to some of these concepts, we will not present additional information at this point because there is a lack of broad information showing proven performance records for such pavements. The case study for Pennsylvania presented in Module 6 of this course shows the performance history for some particular cases that include concrete pavement restoration. The AASHTO Guide [AASHTO 86] presents one model for the prediction of distress of restored rigid pavements.

Overlay Performance Models

Conceptually, the performance model for an overlay may be different than the model for a new pavement. The subgrade soil, subb/ase, and base have been subjected to repeated stress applications, which should densify the material and reduce the potential for further deformation. On the other hand, if the existing pavement has cracked, the unbound layers may be saturated and, therefore, have a lower strength than when they were new. The bound pavement materials have been subjected to repeated traffic applications and, therefore, some of the fatigue resistance of these materials has been used up.

While the relative performance of a new pavement versus an overlaid pavement is subject to speculation, the pavement manager must have a performance model for use in the pavement management system. The basic overlay equation used in the AASHTO Guide is:

$$SC_{OL}^{n} = SC_{y}^{n} - F_{RL}(SC_{xeff})^{n} \qquad \text{(Eq. 9.1)}$$

Where:

SC_{OL} = The structural capacity that the overlay must provide.

SC_y = The total structural capacity required to support the overlay traffic over the existing subgrade (foundation) conditions.

SC_{xeff} = The effective structural capacity of the existing pavement immediately prior to the time of the overlay, which reflects the damage to that point in time.

F_{RL} = The remaining life factor which accounts for the damage of the existing pavement as well as the desired degree of damage to the overlay at the end of the future traffic. It is always less than or equal to 1.0.

n = A coefficient to account for the bonding condition between an existing rigid pavement and a rigid overlay.

The structural capacity is expressed either as structural number for flexible pavements or as surface thickness for rigid pavements. The required structural capacity is determined from the design procedures for a new pavement. The effective structural capacity and remaining life factor of the existing pavement is determined from an analysis of the pavement condition. Examination of Equation 9.1 reveals the performance of the overlay pavement is estimated based on the same performance equation as a new pavement. In other words, once the condition of the existing pavement is established, the rate of serviceability loss with traffic is equal to the serviceability loss for a new pavement.

9.2.9 Summary of Rehabilitation Design

This section demonstrates the lack of good performance information or models for rehabilitation techniques. This is an area of strong potential research for the future and pavement managers should keep this in mind. It is necessary, however, to have performance models for second and third analysis periods after a pavement is rehabilitated. At this point, it is best to look at the case study examples that treat specific engineering applications of pavement management systems. In some of these examples, a set of rehabilitation models have been selected based on analysis of the best available information and a cohesive project level pavement management approach has been adopted. The pavement manager is warned at this point not to adopt one set of models from one methodology and

use it with another set of models from another methodology. The total project level pavement management system must be cohesive if the results of the pavement management process are to be effective.

9.3 PENN DOT: CONCRETE PAVEMENT REHABILITATION STRATEGIES

9.3.1 Introduction

This section is extracted from an unpublished 1996 paper written by Gaylord Cumberledge and Dennis A. Morian [Cumberledge 97]. It is a good example of using both network and project level PMS data for rehabilitation evaluation and design.
Two case studies of pavement rehabilitation are made:

Case Study I. Concrete Pavement Restoration (CPR)
Interstate 79
Mercer County
Jointed Concrete Pavement
Case Study II Rubblization Concrete with Concrete Overlay
Interstate 90, a major east - west freight hauling route
Mercer County
Jointed Concrete Pavement

Highlighted are PennDOT's methods of using pavement management data in the design and evaluation of rehabilitation construction. Where available pavement distress condition indexes are shown for both before and after rehabilitation.

These examples show that additional work was needed to enhance pavement management data in making assessments of the rehabilitation methods.

9.3.2 Selection of Pavement Rehabilitation Strategies

The need for rehabilitation is triggered by unacceptable pavement performance based on criteria established by management and engineering practice.

The key to selecting the correct rehabilitation design strategies is a thorough evaluation of the existing pavement along with evaluations of the performance of previous rehabilitation methods on similar pavements. This includes understanding the total history of the pavement and the design and construction methods used. It also may require a detailed field investigation, which in many cases includes material sampling and lab testing. The depth of this investigation will depend on the problems encountered and the experience of the designer. Pavement performance history obtained from the PMS is one of the key items in the case studies. The following is a list of key features considered in these case studies.

- Pavement distress and condition history.
- Pavement history, including construction, rehabilitations, and maintenance.
- Understanding of design history.
- Material used and exiting condition.
- Construction methods.
- Traffic history.
- Climatic history (at least in general).
- Subgrade type, characteristics, and behavior.
- Drainage (design and effectiveness).

9.3.3 Case Study I: Concrete Pavement Restoration (CPR)

Restoration of sound concrete pavements can be an effective means of extending pavement performance. Restoration is the correction of a number of deficiencies that exist in the pavement. These deficiencies may exist purely due to fatigue or may be due to design or construction problems. A CPR project in Pennsylvania may include major pavement patching, joint rehabilitation, under-sealing, small repair, drainage improvements, tied concrete shoulders, and diamond grinding. Often these projects exceed one million dollars per four-lane mile and result in extensive traffic delays and lane closures.

The key to successfully applying CPR lies in determining that pavement indeed continues to be in sound condition. This includes the evaluation of traffic, loading, materials performance, and structural adequacy. Drainage features have been a major consideration in all CPR project designs.

Table 9.1 lists the performance data for a number of Concrete Pavement Restoration (CPR) projects in Pennsylvania. The table provides the project information and date of activity for each project. Dates provided are for the original construction, the CPR, and any subsequent activity. The table provides performance data based on PennDOT's Overall Pavement Index (OPI). OPI is an index used by PennDOT based on distress, rutting, roughness, and friction parameters. The history of OPI data for six of the pavements is plotted in Figure 9.4.

Table 9.1 shows the performance for each section prior to the CPR repair, and subsequent rehabilitation. From this information, the performance life of the CPR repair can be determined, as also shown in the last column.

In the case of I 79 in Mercer County, the project continues to provide good service eleven years after rehabilitation was performed. This project included the addition of tied concrete shoulders during the rehabilitation. While this is not necessarily recommended for all CPR candidates, is has worked quite well on this project. This pavement section was determined to be a good CPR candidate, containing minimal structural distress at the time of rehabilitation. Observations seem to confirm that addition of tied shoulders has contributed to the long term performance of the restoration.

After several years of experience during which each of the individual elements of CPR were honed and perfected to the existing state of the art, PennDOT developed a methodology for evaluating the suitability of the existing pavement for the CPR strategy. Since CPR provides no enhancement of structural capacity to the existing pavement, it is important to evaluate the potential for further load carrying capacity of the pavement prior to making a large investment in repairing it. Many CPR projects were observed to continue to develop the same type of distress adjacent to the new repairs made. For example, slab cracking, spalling, and deterioration often occurred immediately adjacent to a new pavement patch. In addition to changes in internal slab stress, which result from introducing the new load transfer location, we continue to load old slabs next to the patch, which have expended most of their load carrying capacity.

9.3.4 Case Study II: Rubblization with Concrete Pavement Overlay

The rubblization rehabilitation method was developed out of necessity, for projects such as discussed in this case study. The first successful I-90 project was constructed in 1986-87 that carries 13,500 ADT with 22% trucks. Highway I-90 has a long history of very poor subgrade performance, which showed up as severe pavement pumping and joint faulting prior to rehabilitation. I-90 is a major east-west highway providing a major freight hauling route connecting eastern commercial and industrial centers in New York and New England with Midwestern centers like Chicago, Milwaukee, and Minneapolis.

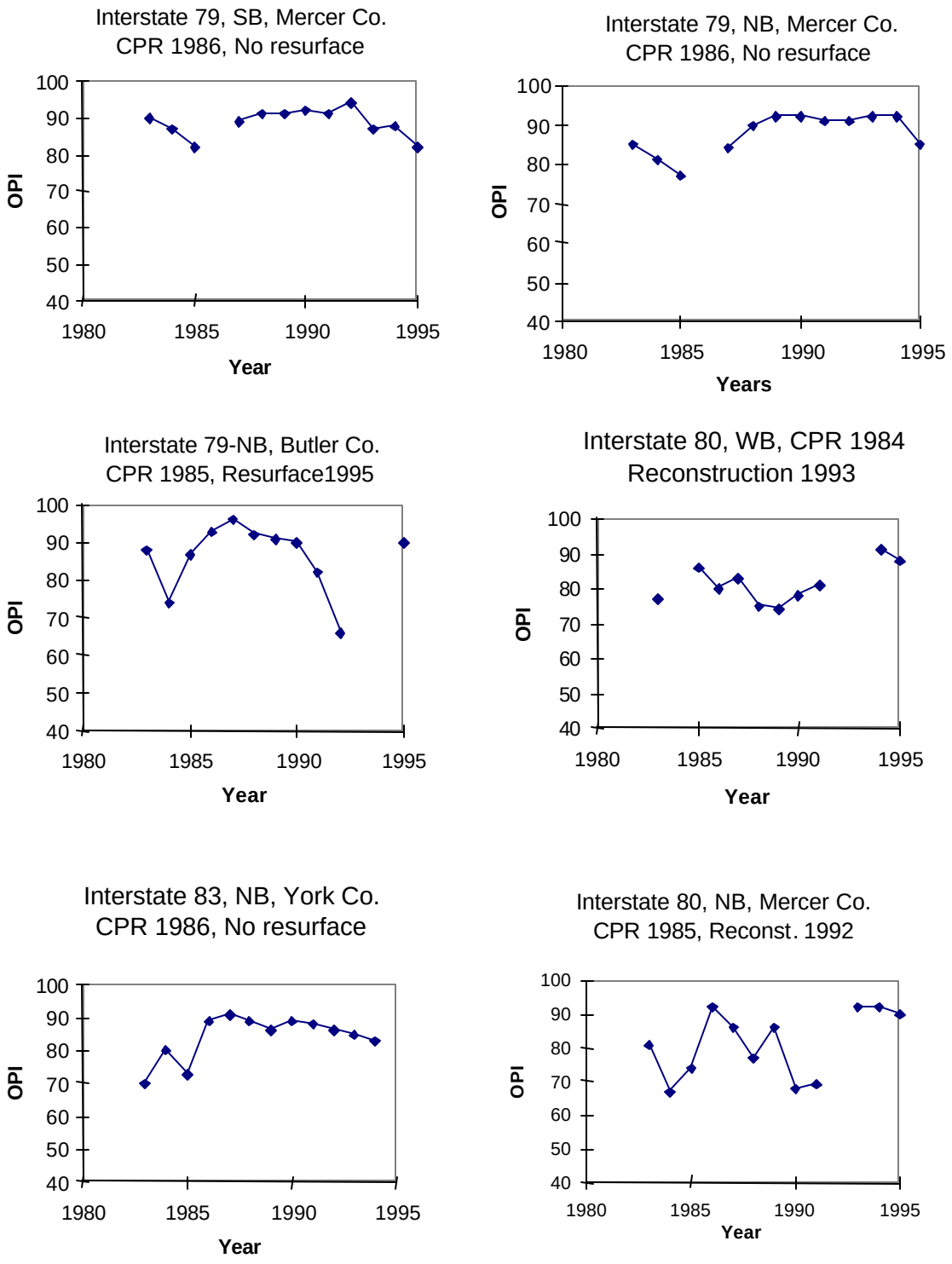


Figure 9.4 Performance of Six CPR Projects in PennDot in Reference to Case 1

Table 9.1 Performance History of Concrete Pavement Restoration Projects (CPR) in PennDot

State Route No.	Seg.	Dir.	CPR Year	Age of Pvmt at CPR	Resur. or Reconst.	Year	OPI (Overall Pavement Index)													Avg. ADTT	AATT Group	Time from CRP to Rehab/
							1983	1984	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995			
79	1130	NB	1986	21	none	n/a	85	81	77		84	90	92	92	91	91	92	92	85	785	Light	n/a
79	1115	SB	1986	21	none	n/a	90	87	82		89	91	91	92	91	94	87	88	82	1093	Light	n/a
79	1010	NB	1985	21	Resurf.	1995	88	74	87	93	96	92	91	90	82	66			90	1796	Light	10
79	1015	SB	1985	21	Resurf.	1995	83	69	87	92	89	92	88	86	85	80	76		88	1839	Light	10
80	2445	WB	1984	20	Reconst.	1993	77		86	80	83	75	74	78	81			91	88	2380	Med.	9
80	2421	WB	1984	20	Reconst.	1993	86		87	81	82	69	72	80	82			89	87	2380	Med.	9
83	361	SB	1986	35	none	n/a					82	83	66	83	79	74	59	71		2777	Med.	n/a
83	370	NB	1986	35	none	n/a	70	80	73	89	91	89	86	89	88	86	85	83		3069	Med.	n/a
80	441	WB	1985	20	Reconst.	1994		82	85		85	79	77	82	82	47			89	4155	Heavy	9
80	91	WB	1985	?	Reconst.	1992	81	67	74	92	86	77	86	68	69		92	92	90	4303	Heavy	7
80	60	EB	1985	?	Reconst.	1994	89	63	81	94	87	79	84	71	75			93	91	4319	Heavy	9
80	424	EB	1985	20	Reconst.	1993	80.8	80	79.3	87	86	81		91	87	80.3	77		90	4459	Heavy	9

The original concrete pavement on I-90 was constructed in the late 1950's. The subgrade soil is a blue shale clay found along this corridor and is know as having high moisture content, and generally providing poor support for the concrete pavement. The geotechnical report prepared prior to the original construction recommended a 1' to 2' granular blanket be provided to support the pavement. For numerous reasons, this recommendation was often not followed. The omission of the granular blanket has resulted in years of pavement problems.

Several rehabilitation strategies in the late 1970's and early 1980's had previously been tried along 47 miles of the I-90 route. Each had recurring distress and structural failures because of the poor subgrade support and high traffic loadings. One 30-mile project to the west of the rubblization project constructed in the late 1970's with an un-bonded CRCP overlay was unsuccessful. In fact punch-out distresses developed before construction activities were finished because the original rigid pavement was never successfully stabilized. To the east in 1980-82, 12 miles of badly broken pavement was cracked and seated with a 13" asphalt overlay. By 1988, an unsuccessful fog seal had been applied, and a mill and resurfacing strategy had been applied twice.

The five-mile section, which was later to have rubblization, was overlaid with 10.5" asphalt concrete at the same time as the 30-mile un-bonded CRCP overlay was being placed. By 1983, this asphalt overlay had experienced severe stripping above the joints in the original concrete pavement. The damage appeared as depressions, extending one inch on both sides of the joint.

The challenge was to develop a design that would correct the lack of support to the concrete pavement slab and address the weak subgrade. The strategy chosen was to remove the existing asphalt overlay and effectively seat the concrete pavement on the subgrade. Earlier experience with cracking and seating of a 61.5' jointed concrete pavement frequently resulted in reflective cracking at the old joints. However, fracturing the pavement into smaller fragments destroyed the bond to reinforcement and allowed more uniform establishment of a sound base. The total rehabilitation strategy included removing the 10.5" of asphalt overlay, rubblization of the existing jointed concrete pavement, new pavement drains and a 12" plain jointed concrete pavement. The new 12" pavement, placed in 1986-1987, had 20' joint spacing with 6:1 skewed joints, and tied concrete shoulders.

The advantages to this design were:

- Improved k value as compared with the native subgrade.
- Enhanced drainage capability to remove the water in the subgrade.
- Increased elevation separation of the pavement from the subgrade water table.
- Placed traffic loads farther from the slab edge with concrete shoulders.

In addition, in-place recycling of the existing concrete pavement could be accomplished. The performance history for 1991 to 1995 measured by IRI and OPI of I-90 is shown in Figures 9.5 and 9.6. Unfortunately data was not available for prior to 1991. The concrete overlay has resulted in good pavement performance to date, and indications at this time are that this performance will continue. It is noted that the performance (OPI) has remained fairly constant over the four-year period and roughness (IRI) has increased only slightly. In 1996 the most noticeable distress is wheel path wear resulting from studded tire use.

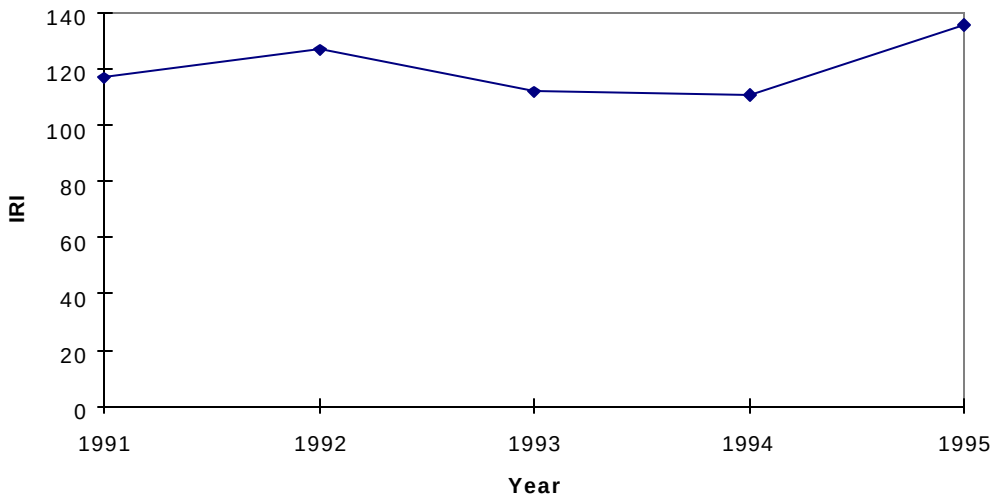


Figure 9.5 IRI History for I-90, Case II: Rubblization with Concrete Pavement Overlay

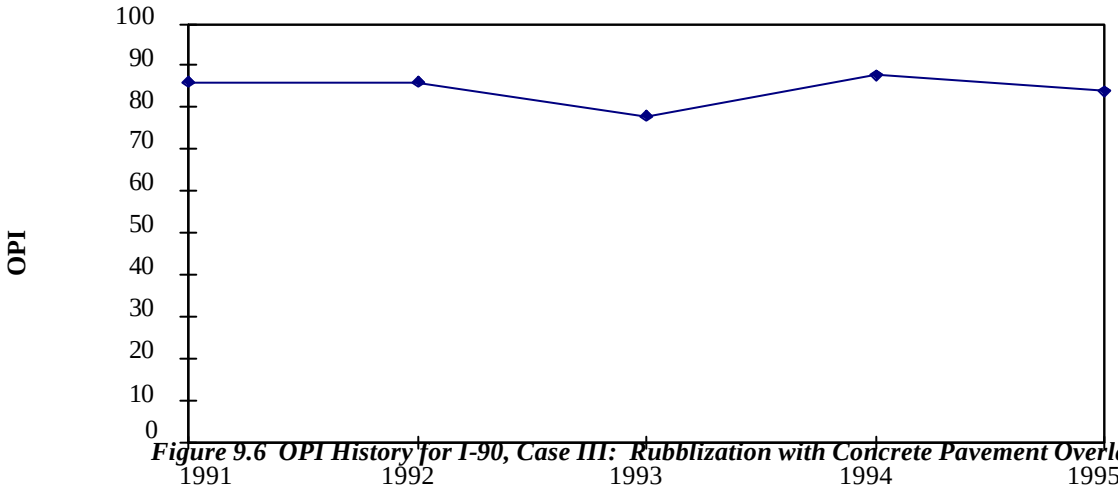


Figure 9.6 OPI History for I-90, Case III: Rubblization with Concrete Pavement Overlay

9.3.5 Conclusions

Concrete pavement restoration, even without a new surface, can substantially increase the service life of a pavement as long as its structural capacity prior to restoration is basically sound.

For a concrete pavement with structural failures a thorough rubblization and effective seating of the existing pavement, covered by a 12" concrete overlay, 20' skewed joints, and tied shoulders, has so far provided a good solution.

9.4 MONTANA DOT: OPTIMIZE PRESERVATION WITH LIFE CYCLE COST ANALYSIS (LCCA)

9.4.1 Introduction

Montana is the fourth largest state in the United States in terms of area, and has a population of only about 800,000. The largest metropolitan area in Montana has a population of approximately 75,000. Montana has a roadway network of over 34,000 miles including state-maintained highways, Indian reservations, park service roads, and urban streets. The MDT is responsible for approximately 24,000 lane miles of state maintained highways, with an additional 4,000 lane miles of urban roadway that are eligible for federal funds, but are not state maintained. The MDT divides the state into five districts and 11 maintenance divisions.

Montana is primarily a bridge state for travelers; meaning that a majority of travel does not originate or end in Montana. The MTD has the responsibility of insuring that all national highway system and other state and local owned federal-aid roadways are under acceptable levels of pavement management. In 1989, the ISTEA legislation was passed and new requirements for pavement management were to be implemented by every state highway agency. The FHWA evaluation of Montana's management system determined that improvements were required to the existing process. The department established the function of Pavement Management Supervisor and contracted for the development of a new state-of-the-art system that could be tailored to the needs of the state of Montana.

9.4.2 PMS Overview

Data Collection and Management

The data collection and management component of the PMS is based on procedures that have developed and evolved over the past 12 years. These processes have been refined and incorporated into a modern client/server relational database management structure for use in the updated PMS.

Inventory

The route-milepost referencing method used by MDT is a workable system that can serve the needs of PMS. All referencing systems in the state, related to PMS, use this method. This avoids confusion and makes the exchange of data easy to handle.

Separate data items in the database (e.g., inventory, structural history, condition data, traffic files) are all maintained individually in the database using a common referencing system. The PMS software dynamically combines this information based upon homogeneous construction sections during analysis when needed.

Inventory data maintained by the department includes roadway information such as route identification, functional classification, width, number of lanes, MDT district information and other basic variables used for reporting and analysis. This information is stored using as many sections as necessary to define the included variables completely. During analysis the PMS system combines this information with structural history, condition, traffic and other information to produce project sections typically based upon homogeneous construction history and historical performance. The current method of defining pavement sections based on homogenous construction history is a valuable way to store information for developing performance models and pavement designs. The information is updated at regular intervals to incorporate new or corrected data. The new PMS system allows MDT to update and maintain their inventory information more efficiently through improved software to access the information [TRDF 96] [TRDF 96a].

History File

Detailed history files have been maintained covering project locations, dates, materials, type of work performed, and costs. This information is incorporated into the updated PMS and new routines have been added to access, review

and update this information as new projects are completed. In addition these project files are used extensively by the PMS to maintain and update the project sections as new projects are completed and entered into the relational database system.

Structural History information is stored by contract number and is referenced to the beginning and ending milepoints of each contract. These files contain information about each layer placed (or removed) during the construction including the layer thickness, material type, width and other as-built information.

Condition Survey

For pavement condition evaluation, a 200 ft. sample, taken at every milepoint, is visually rated using a walking survey. The sections used for pavement condition evaluation are easy to locate in the field, and remain consistent from year to year since they are located by the mile-markers in the field. For network level analysis, sampling once per mile provides a reasonable sample for describing the condition of the network. Also the condition surveys are recorded and maintained in the database on a mile-by-mile basis that allows the PMS to query and return condition information for any specified length of roadway. By maintaining the condition data in its raw form, PMS can adjust project section breaks without losing condition information.

The current MDT system is based on manual, visual distress surveys that tend to be subjective. Therefore, a thorough training and quality assurance program is in place. Raters are given a week of intensive training including office discussions and field exercises. As surveys progress, a 2% sample of sections is randomly selected by the PMS section data coordinator for verification. If significant errors in ratings are found, retraining is undertaken immediately and previous ratings are thoroughly checked. This manual method will be continued until the capabilities of automated distress evaluation have been investigated and their results are validated.

Distress data collection will be automated as soon as the technology for fully automated crack detection has been shown to be accurate and repeatable. However, automated distress identification is an area of "emerging technology" and the reliability of the output of these devices must be thoroughly evaluated. A transition to semi-automated and/or fully automated distress surveys will take place in the future. The distresses currently collected and the method used to distinguish severity and extent will be re-evaluated every year to consider state-of-the-art technology. The PMS database and analysis is flexible enough to allow a variety of distresses and rating procedures to be used and defined in an easily customizable manner.

Network-wide roughness measurements are collected in 0.1 mile increments and are extensively used for network evaluation and project selection purposes. With the acquisition of new laser sensors for the MDT South Dakota profiler the department will evaluate the roughness of the entire interstate and primary networks as well as all other federal-aid roadways on an annual basis. Since roughness measurements are relatively economical to obtain and provide valuable information on the overall condition of the network, MDT will collect this information annually for all NHS and federal-aid roadways.

A network level structural rating is being performed by routine deflection testing. The network-wide structural information is used to help identify structural problems early and adjust assigned minor rehabilitation and maintenance treatments to extend the life of the pavement before major rehabilitation or reconstruction is required.

Traffic Information

The PMS system stores and maintains a traffic information file that contains AADT and equivalent single axle load (ESAL) estimates on the network. These values are used for both network and project level pavement management analysis. Traffic counts, classifications, and truck weight information has been collected by the traffic division and is stored in the central Oracle database, which is accessed by the PMS software.

Database System

The entire pavement management database resides on a centralized relational database management system (RDBMS) using Oracle 7.2 on a DEC Alpha server system. Such a client/server configuration is ideal for maintaining a single, well controlled database that all PMS users can access. It also serves as a vehicle to allow effective coordination and integration with the other required management systems that may be developed in the

future. The PMS analysis software resides on microcomputers networked directly to the server and operates directly on the primary Oracle database. No downloading or uploading of information is required.

The PMS section is responsible for maintaining most of the PMS related data in the central database. However, each specific data table may be maintained by other sections within the department. This level of coordination between the PMS section and other sections provides strong incentive for good communication. For example, traffic information and contract work data is maintained by other groups within MDT. The PMS section keeps open communications with these groups to ensure that all data needs are being met. The MDT Information Services Bureau is the central coordinator and overall database administrator for the MDT database on which all PMS related tables reside. They have ultimate control over the database backup and security.

Pavement Management Analyses

PMS is an important activity that spans many groups within MDT and acts as a coordinating and communicating tool. The PMS has information and results that can benefit a large portion of the MDT organization. These benefits can even extend to the maintenance residencies including the maintenance supervisors and crews. These analyses are discussed in the following sections. [Haas 94]

Pavement Condition Analyses

Extensive condition analysis capabilities are available in the PMS software. Condition indices are defined based upon individual distresses and condition measures or any combination of those measures. Such condition indexes include roughness, fatigue cracking, environmental cracking, rutting, structural index, combined roughness and cracking, and an overall pavement index. Other combinations of indexes will also be defined and tested for their effects on the network results. The weighting factors, deduct values and threshold values used in the analysis models have been customized by MDT to fine tune the analyses based on improved experience.

An overall condition score is adequate for global communications but not for selecting projects and specific actions. It is more important to look at the individual distress mechanisms that provide more information and are necessary to properly select the treatment that will correct the observed problems.

As part of the development of the PMS, techniques are being developed for considering alternative pavement preservation strategies depending on the types and progression of the pavement condition indexes as well as functional considerations.

These include use of:

1. Preventive maintenance treatments, such as seal coats,
2. Various overlay thickness'
3. Recycling of the existing surface,
4. Use of alternative materials, such as asphalt-rubber and polymer modified asphalt,
5. Treatments for rigid pavements, and
6. Reconstruction.

Pavement performance models are being developed for each of the alternative strategies considered by the system. These models are updated with new information each year as new condition and structural data is added to the database. This provides the critical feedback mechanism needed for any successful PMS.

Pavement Performance Analysis

Pavement performance models are required for the network level analysis. The structure of the models is dependent on the type of network level analysis that is being used. For example, the Markovian approach has been implemented for network optimization, and transition probability matrices have been developed. In addition, deterministic models are used for estimating remaining service life, project selection and treatments assignment as well as prioritizing these projects over a multi-year period. The integrated pavement performance modeling software in the PMS helps MDT to develop the required models from the databases.

In addition to the network level analysis, the updated pavement management system also addresses project level analysis. Performance models are also used for a risk based life-cycle cost analysis system that helps pavement designers select appropriate rehabilitation strategies based upon pavement life-cycle costs.

Investment Analyses

One of the most important areas addressed in the pavement management system is a method for formulating the pavement preservation budget. The budgeting integrates the actions of maintenance and construction into an effective technique for selecting the candidates for pavement preservation treatments. Several approaches have been developed for model formulation, based on simple priority formulas to near and full optimization. The PMS software has the capability to define multiple analysis scenarios, which utilize any of the various techniques most commonly employed, by state DOT's. MDT uses the software to execute the network analyses in a variety of forms and evaluate the results to select the most appropriate methodology for the state.

The flow of information from network to project level will start at the network level and involve, from the outset, good communications with the district personnel who will receive the initial network level budget allocations and splits. This communication and coordination with the districts before and during development of the network recommendations has the effect of minimizing problems and dissatisfaction with the initial recommendations provided to the districts from the central office.

Engineering Analyses

The scope of the pavement management system includes the depth required to do engineering analyses in terms of project level life cycle cost analysis, pavement structural review, and integrated maintenance analysis. Pavement management covers both the network and project levels. Project level pavement management includes the evaluation of various alternatives and the structural requirements of the selected treatment. It also encompasses the selection and implementation of routine and preventive maintenance treatments.
The details of the project level system provide extensive feedback to improve the network level analyses. When a detailed evaluation is performed at the project level, this information is used to upgrade the database for that specific project and assist in network-level solutions. Over time, as more and more project level information is input to the database, the network level analyses will be used to make global decisions with a more detailed understanding of individual segment structural properties and performance histories.

PMS Update/Feedback Process

It is important to have an adequate feedback loop within the PMS. This loop will track what happens after the global budget allocations and the draft plan or guidelines for project and treatment selection is provided to the districts. The feedback will identify for the central office 1) the final plan developed at the district level, 2) what goes out to contract and estimated costs, 3) what gets built on the roads including costs and 4) how the as-built roads perform throughout their life-cycle. Other types of feedback mechanisms are incorporated in the PMS to help evaluate the accuracy of the various models, variables, and constants used in the PMS database and analyses. Update procedures are included to refine these parameters as better information is developed in order to ensure continual improvement of the overall PMS process with time. Examples of such parameters may include unit costs, available treatment alternatives, pavement performance indexes, pavement performance models, decision tree structures, index thresholds, priority indexes, optimization objective functions, and others. The PMS software has enough flexibility to accept these updates and expand in capabilities in coordination with the capabilities of MDT.

9.5.3 Montana Life Cycle Cost Analysis - Visual/ LCCAÔ Software

The Visual/LCCA™ software developed for the Montana Department of Transportation [TRDF 96] incorporates life cycle cost analysis (LCCA) procedures. A pavement ***project*** is a specific length of highway that is to be rehabilitated or reconstructed and each is analyzed for life-cycle costs. An alternative is a specific design that meets requirements of the project. Each project can have any number of alternatives.

The inputs to the LCCA program can be broken into four categories:

1. Pavement performance inputs - The performance of the pavement section determines when pavement improvement and maintenance costs will occur. This defines the time when cash outlays are required by the agency. To obtain these inputs the software uses the performance models from the pavement management system and calculates expected life of each available treatment based upon a user-defined threshold. The user may also associate maintenance activities with each treatment and specify when those activities will occur with respect to treatment application.

2. Cost inputs - These define the magnitude of the costs that are incurred as each rehabilitation and/or maintenance action is performed on the section over the analysis period. Each treatment and maintenance activity have associated unit costs which are defined by the user. Costs used in the LCCA Program are:

   ? Treatment initial cost
   ? Administration cost (design, planning etc.)
   ? Maintenance cost
   ? User delay cost (calculated based on traffic composition, length of construction etc.)

3. Project Inventory - These are parameters that define the project to be analyzed and do not depend upon the alternative being considered. The project inventory includes the following:

   ? Length
   ? Width
   ? Traffic level
   ? Traffic composition (% trucks, etc.)
   ? Description
   ? Discount rate distribution
   ? Traffic growth rate

Of the input parameters, the following have been identified as uncertain variables, therefore distributions instead of point values may be assigned to them in the life cycle cost analysis. The user may specify the distribution of each variable.

1. Discount rate
2. Traffic growth rate
3. Treatment life, the number of years it takes for a pavement to deteriorate to the minimum acceptable level after a treatment is applied
4. Unit cost of treatments (or material and labor cost)

For this version of the LCCA program, two distributions for uncertain variables may be used: 1) triangular distribution and 2) normal distribution. For triangular distribution, three variables: minimum value (a), most likely value (b), and maximum value (c) need to be specified, while for normal distribution the mean (μ) and the variance (σ^2) need to be defined. The user may elect to use either form of the distributions for simulation analysis. [Weston 90] [MDT 94] [OMB 94] [Haas 94]

Use of Input Distributions for Simulation Analysis

The simulation analysis randomly calculates a distribution of the life cycle cost as discussed earlier. During each run of the simulation all uncertain variables are randomly selected from their respective input distributions. Then these values are used to calculate a life-cycle cost. This process is repeated numerous times and the results tabulated to give the expected distribution of the total life-cycle cost of the alternative.

Simulating a Triangular Distribution

To calculate a random variable X from a triangular distribution the following is used. Figure 9.7 shows the probability distribution function (pdf) of a triangular distribution Its equation is given by:

$$f(x) = 2(x-a)/(b-a)(c-a) \quad \text{for } a \le x \le b$$
$$f(x) = 2(c-x)/(c-a)(c-b) \quad \text{for } b \le x \le c$$
$$f(x) = 0 \quad \text{otherwise.}$$

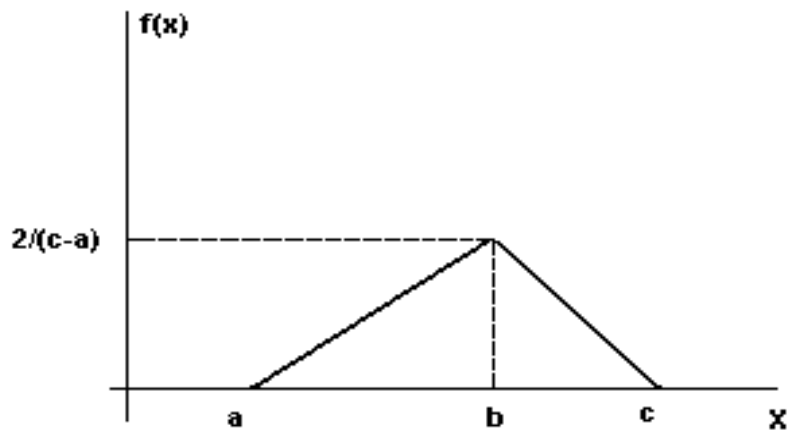


Figure 9.7 A Triangular Distribution

The cumulative distribution function (cdf) for this distribution is given by:

$$CDF(x) = 0 \quad \text{for } x \le a$$
$$CDF(x) = (x-a)^2/(b-a)(c-a) \quad \text{for } a \le x \le b$$
$$CDF(x) = 1 - (c-x)^2/(c-a)(c-b) \quad \text{for } b \le x \le c$$
$$CDF(x) = 1 \quad \text{for } x \ge c$$

The CDF for a triangular distribution is a closed form, meaning that it exists for a finite range of x. If we generate a random number r that is between 0 and 1 and let r be equal to the cumulative probability of a random variable x that has a triangular distribution then the following is true:

$$r = (x-a)^2/(b-a)(c-a) \quad \text{for } a \le x \le b$$
$$r = 1 - (c-x)^2/(c-a)(c-b) \quad \text{for } b \le x \le c$$

solving for x, we get a random variable generator if r is a random number between 0 and 1:

$$x = a + [(b-a)(c-a)\,r]^{1/2} \quad \text{for } 0 \le r \le (b-a)/(c-a)$$
$$x = c - [(c-b)(c-a)(1-r)]^{1/2} \quad \text{for } (b-a)/(c-a) \le r \le 1$$

Simulating a Normal Distribution

For a normal distribution with mean μ and variance σ^2, the commonly used method to generate a normally distributed variable X is the direct method. This method produces exact normal random variates and it is easy to apply and execute. First we generate two uniformly distributed random numbers, r_1 and r_2, that are between zero and one. These are transformed into a normal random variate, Z, with mean 0 and variance 1, using the following:

$$Z = (-2 \ln r_1)^{1/2} \sin 2\pi r_2$$

Those standardized normal variates are then transformed into a normal variate X with a distribution having mean μ and variance σ^2 using:

$$X = \mu + \sigma Z$$

Simulation Analysis Procedure

The life cycle cost model used to prepare the estimates is based upon a cash flow diagram and the time value of money. To compare life-cycle costs for varying design alternatives it is necessary to discount all expected cash flows for each alternative to their present value so they may be compared using a common metric. This discounting uses the expected magnitude, time and discount rate for each cash flow in the life cycle of the design alternative.

For situations with constant discount rates over the life cycle the following may be used to calculate the present value of an investment in year n:

$$PV = \frac{FV}{(1+d)^n}$$

where:

PV = present value
FV = future value
d = discount rate used over a period of n years (or discounting periods).

This formula is valid only when the discount rate is constant for all years in the analysis. To make the model more general the following formula will be used so that the user may vary the discount rate according to a specified input distribution for each year of the analysis.

$$PV = \frac{FV}{\prod_{i=1}^{n} (1 + d_i)}$$

where all values have been defined above.

For each year of the analysis a randomly sampled discount rate d_i is used in the formula. The effective discount rate over the analysis period is calculated by multiplying each year's discount rate from the beginning of the analysis to the year of the cash flow. Note that this formula is equivalent to the previous formula when the discount rate is held constant over the analysis period.

Using this formula it is then possible to reduce each estimated cash flow to its present value. Then the total life-cycle cost is the sum of all the present values of expected cash flows over the analysis period minus the discounted salvage value of the pavement at the end of the analysis period. The salvage value is calculated using a straight-line depreciation from the last treatment applied to the pavement. [OMB 94]

Simulation Process

There are four steps to the sampling simulation. This section will describe each step briefly to illustrate the simulation process.

Step 1: Sample Project Variables from Input Distributions

The first step in the simulation process is to generate estimates for the discount rate and the traffic growth rate for each year of the analysis period. Note that these two variables are used for all alternatives being studied since the selection of the discount and traffic growth rates are independent of the alternative chosen.

Step 2: Construct Cash Flow Diagrams for Each Alternative

Based upon defined alternatives the software will construct cash flow diagrams for each alternative. The process begins by sampling the design lives of each treatment based upon performance models or user input and determining when each major cash flow will occur. To determine the time at which a treatment n will be applied the following is used:

$$T_n = \sum_{i=1}^{n-1} t_i + \sum_{i=1}^{n-1} e_i$$

Where:

T_n = time in years after the analysis period begins until the application of treatment n.
t_i = expected life of each treatment preceding treatment n. This value is fixed at the expected life value input from either performance models or the user.

ε_i = number of years the actual life of treatment i deviates from the expected life of treatment i. This value has a mean of zero and varies according to the triangular or normal distribution specified by the user during the definition of each treatment.

n = number of treatments from initial construction to treatment n.

After the timings of the cash flows are found the costs are determined using the following:

$$C_n = \sum a_j c_j + \sum a_j e_j$$

Where

C_n = is the total cost for treatment n at time i (see previous formula).

a_j = is the units for cost component j within treatment n.

c_j = is the unit cost for cost component j for treatment n at time i.

ε_j = is the deviation of each unit cost from its expected value. This value is determined from the input distribution for each unit cost. As with treatment life this has a mean of zero and either a standard deviation or range as specified by the user.

Step 3: Calculate Discounted Present Values for Each Cost

For each cost calculated in step two the discounted present values are computed and summed to arrive at the life-cycle cost of each treatment.

Step 4: Repeat Simulation Process Until Stable Output Distribution is Found

These three steps are repeated until the maximum number of iterations or the output distribution becomes stable within a specified tolerance.

Example

Table 9.3 shows two possible alternatives for a rehabilitation project in Montana. The project is 5.8 miles long with a pavement width of 32 feet including shoulders. Alternative 1 consists of two thin resurfacing treatments to maintain ride quality until a complete reconstruction can be done in year eight. The second alternative is a complete reconstruction for the initial treatment followed by a thick overlay after an expected life of 20 years. The cost of each treatment along with its associated maintenance activities is shown in Table 9.4.

Table 9.2 Alternative Definitions

Year	Treatments Assigned	
	Strategy 1 – Thin Overlay	*Strategy 2 - Reconstruction*
0	Thin Resurfacing (Design life four years)	Reconstruction (Design life 20 years)
4	Thin Resurfacing (design life four years)	----------
8	Reconstruction (design life 20 years)	----------
20	----------	Thick overlay (design life 15 years)
28	Thin Resurfacing (design life four years)	

Table 9.3 Overview of Construction, Administrative, Maintenance and User Costs for Selected Treatments

	Cost Total (Unit Cost $/SY)			
Treatment	**Construction Cost**	**Administrative Cost**	**Maintenance Cost**	**User Delay Cost****
Thin Resurfacing	$1,200,000 ($11.76)	$250,000 ($2.45)	----	$99,517 ($0.97)

Reconstruction	$2,400,000	$350,000	----	$199,033
	($23.52)	($3.43)		($1.95)
Crack Seal			*$11,000*	*$6,634*
(Years 3,10,17)			*($0.11)*	*($0.06)*
Seal and Cover		*$40,000*	*$200,000*	*$11,057*
(Years 7, 14)		*($0.39)*	*($1.96)*	*($0.11)*
Thick Overlay	$1,800,000	$350,000	----	$132,689
	($17.63)	($3.43)		($1.30)
Crack Seal			*$11,000*	*$6,634*
(Years 3,10,17)			*($0.11)*	*($0.06)*
Seal and Cover		*$40,000*	*$200,000*	*$11,057*
(Years 7, 14)		*($0.39)*	*($1.96)*	*($0.11*

**Calculated using Pennsylvania DOT method [PennDOT 96]

Uncertainty in Treatment Costs and Design Lives

Currently there is a lack of adequate cost history and performance history to estimate variability in the cost and design lives of the treatments for both strategies. Cost standard deviations were estimated based on a maximum likely change of approximately 10%. Performance variability was estimated based upon the opinion of MDT staff engineers given the existing condition of the pavement project. Subjective opinion was used to arrive at the standard deviations for each element listed in Table 9.5 below. It is hoped that as more data is accumulated through the Pavement Management System and through other activities currently underway at the department that both costs and performance variability estimates can be made through the use of historical data. The PMS is currently capable of producing estimates of design life by reviewing similar projects' rehabilitation histories; however, variability estimates are difficult to make at this time.

Table 9.4 Specification of Variability for the Example Project

Element	Mean (m)	Standard Deviation (s)	Maximum Likely Value (m+3s)	Minimum Likely Value (m-3s)
Thin Resurfacing				
Design Life	*4 Years*	*0.33 Years*	*5 Years*	*3 Years*
Construction Cost	*$1,200,000*	*$120,000*	*$1,560,000*	*$840,000*
Administrative Cost	$250,000.00	*$25,000*	*$325,000*	*$175,000*
User Delay Cost	$99,517.00	*$10,000*	*$129,517*	*$69,517*
Reconstruction				
Design Life	*20 Years*	*1.66 Years*	*25 Years*	*15 Years*
Construction Cost	*$2,400,000*	*$240,000*	*$3,120,000*	*$1,680,000*
Administrative Cost	*$350,000.00*	*$35,000*	*$455,000*	*$245,000*
User Delay Cost	*$199,033.00*	*$19,903*	*$179,130*	*$218,936*
Thick Overlay				
Design Life	*15 Years*	*0.66 Years*	*17 Years*	*13 Years*
Construction Cost	*$1,800,000*	*$180,000*	*$2,340,000*	*$1,260,000*
Administrative Cost	*$350,000.00*	*$35,000*	*$455,000*	*$245,000*
User Delay Cost	*$132,689*	*$13,269*	*$172,495*	*$92,882*

Analysis Results

The results from the analysis show that the expected present value of Alternative 2 is considerably lower than that of Alternative 1. The expected distribution of the life-cycle costs is also smaller as shown by the standard deviation. The results for the simulation and scenario analysis are summarized in Table 9.6. An example of the sensitivity analysis for discount rate is shown in Figure 9.10. Finally a graph of the possible distributions of total life cycle costs for each alternative is shown in Figure 9.11.

From the scenario and simulation analysis results shown in Table 9.6 we can see that Alternative 2 has a lower expected net present value over the analysis period. More significant, however, is the fact that Alternative 2 also has a smaller standard deviation as estimated during the analysis. This indicates that the alternative has significantly less risk.

As shown in this graph Alternative 2 always has a smaller net present value over the life cycle regardless of the discount rate experienced. Overall the net present value of the project is affected the most by the discount rate.

Table 9.5 Simulation Analysis Results

Alternative	Mean Cost	Cost St. Dev	Worst	Best
1 (Thin Resurface)	$4,677,955	$302,302	$4,085,443	$5,270,467
2 (Reconstruct)	$3,669,242	$168,943	$3,338,114	$4,000,370
Difference(1-2)	**$1,008,713**	**$133,359**		

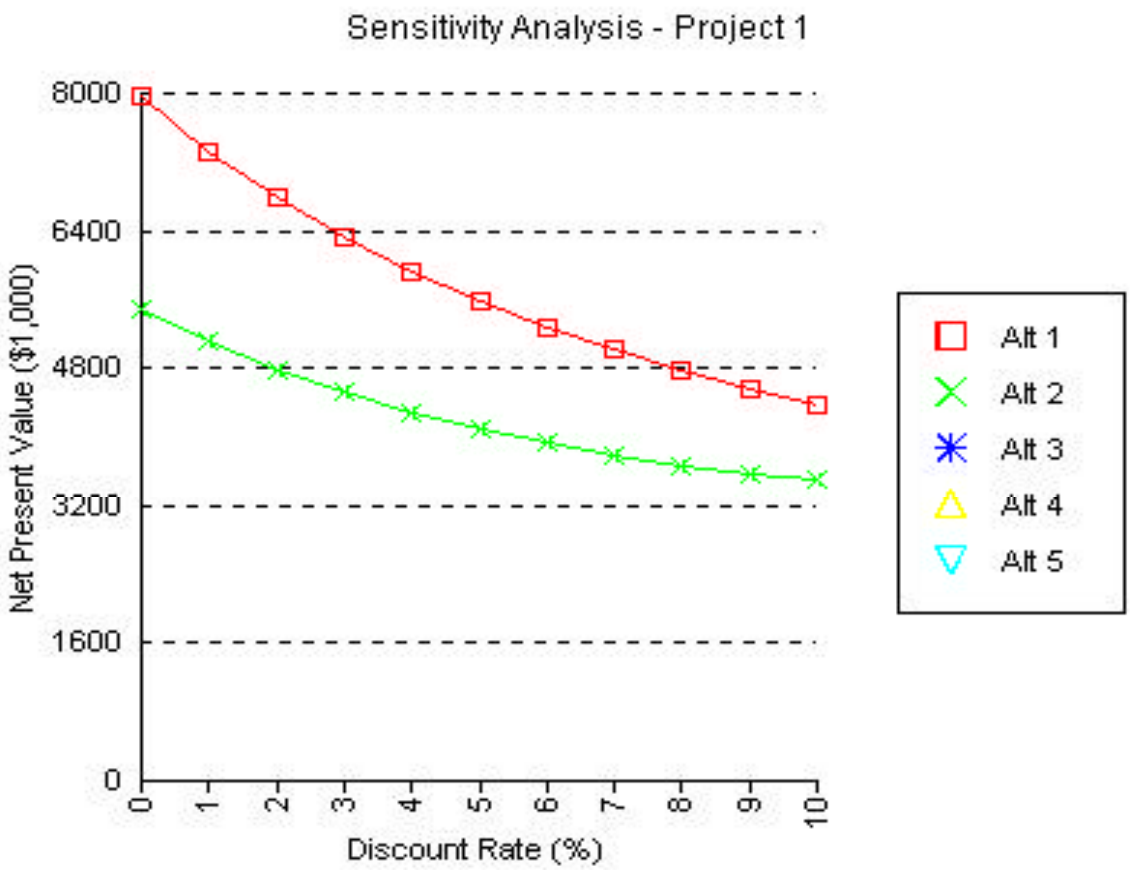


Figure 9.8 Sensitivity Analysis of Discount Rate

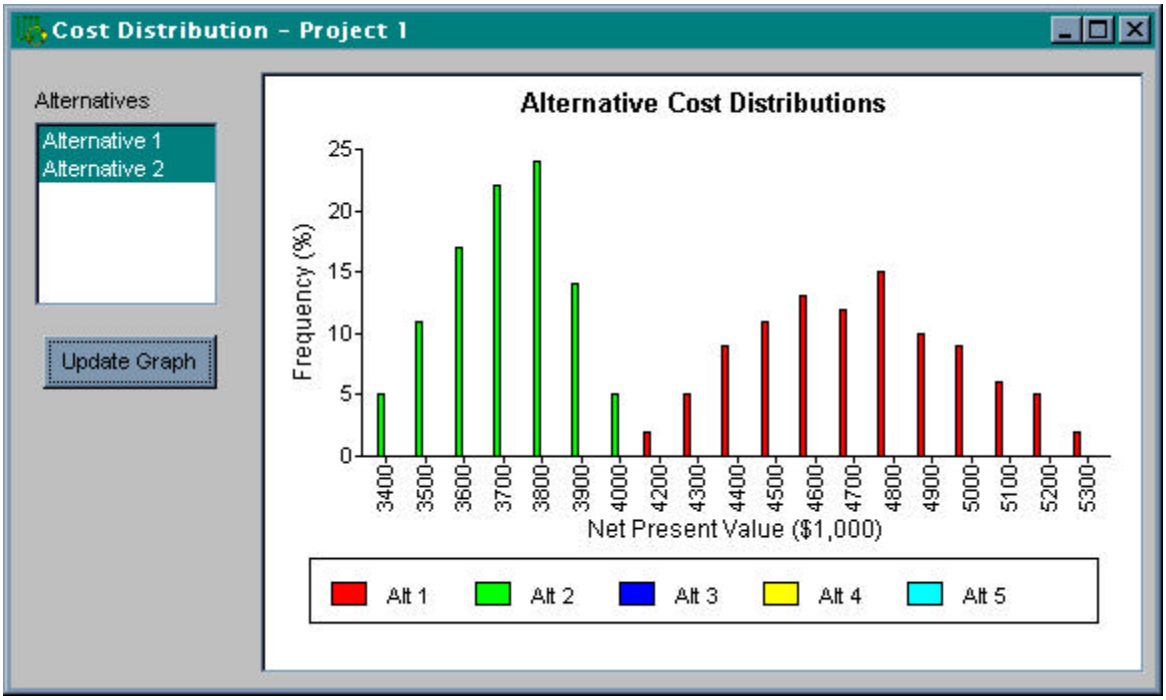


Figure 9.9 Simulation Analysis Results

LCCA Summary

The risk based life-cycle cost methodology and corresponding Visual/LCCA™ software developed for the Montana DOT is an effective tool for project level pavement management life-cycle cost evaluations. The department's recent implementation of a comprehensive pavement management process and software system is collecting the types of data needed to support the required input distribution estimates to utilize the full power of this program. Regardless of the availability of historical data to estimate variations in costs and pavement performance the Visual/LCCA™ package is an excellent tool for inspecting the sensitivity of analysis results to critical variables such as discount rate and pavement performance. It allows for a variety of "what-if" types of analyses in an easy to use environment and provides clear results that can be used to identify risk in decision making.

Conclusions

- The risk-based life-cycle cost methodology and corresponding Visual/LCCA™ software form an effective tool for LCCA.
- The analysis would be very cumbersome without the availability of a reliable and comprehensive PMS database.

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10 EFFECT OF MAINTENANCE ON PAVEMENT PERFORMANCE

10.1 INTRODUCTION

Maintenance is a very important activity, it covers a broad spectrum of work, and the planning and optimization of maintenance activities is a complex task. Still, a properly executed maintenance program will save money by lengthening pavement life, and at the same time improve safety and comfort for the users. Fortunately an increasing number of agencies have realized the potential of maintenance management, and of preventive maintenance activities. One can also see a trend of using more sophisticated maintenance management systems, and the software for such systems has improved considerably also.

Since maintenance work results in considerable exposure to the road users, the image of a transportation agency is often determined by the way they plan and carry out this work. The quality concept “Level of Service” is also greatly influenced by maintenance work.

In the following, an overview of maintenance management will be presented, after which examples of engineering analysis on maintenance-related activities in Montana DOT will be given.

10.2 OVERVIEW OF MAINTENANCE ACTIVITIES AND POLICIES

10.2.1 Introduction to Maintenance

Many authorities recognize that the type, frequency, and degree of maintenance on pavements can significantly influence performance. In addition, it can influence the time at which major rehabilitation, such as an overlay, is required.

The definition of *maintenance* varies among agencies [Haas 94]. In a physical sense, maintenance consists of a set of preventive activities directed toward limiting the rate of deterioration of a structure, or corrective activities directed toward keeping the structure in a serviceable state. For pavements, this includes such preventive work as chip seals and such corrective work as patching. Service-type activities, such as paint striping, sweeping and litter removal, would be included in a "maintenance management system," but because they have little effect on pavement performance, they are not usually considered directly in the pavement management system.

In an administrative sense, maintenance may be separated from rehabilitation by budgetary identification and by being performed in discontinuous sections of no more than several hundred feet in length. For example, a repair followed by a 1 1/2-inch overlay of a 300-ft section of badly deteriorated pavement might be classed as maintenance, whereas the same thickness of overlay covering an entire project length might be classed as rehabilitation and become a capital budget item. Thus, the division between maintenance and rehabilitation is somewhat vague, but there is little need for a more detailed definition for the purposes of this section. It should be noted, however, that AASHTO has a series of procedures and reports relating to various areas of maintenance, including fairly specific definitions. NCHRP has defined a performance-based budgeting procedure appropriate to maintenance management.

This section deals primarily with the information associated with maintenance and rehabilitation and its use in pavement management, rather than with maintenance practices themselves. Although such practices are not considered in any comprehensive manner in this section, their importance to pavement performance should be emphasized, especially in relation to "quality" and frequency of work. There are various manuals available on the details of maintenance practices, such as those of The Asphalt Institute.

10.2.2 Maintenance Management Systems

A maintenance management system is a technique or operational methodology for managing or directing and controlling maintenance resources for optimum benefits. It has been pointed out that this involves the following major components [Butler 75a]:

1. An inventory of the physical elements of the system that can be maintained, plus the physical, operational, and environmental factors that can influence the amount of maintenance work generated.
2. Performance standards that define maintenance procedures, resources in terms of labor, equipment, and materials, and the average accomplishment production rate expected from following the standards.
3. Predictions of the workload generated in terms of maintenance accomplishment units, by physical element of the highway, such as a pavement of a given design subjected to specific traffic loadings in a given environment.
4. Allocations of available resources through objective budgeting mechanisms based on the specific requirements of the system and policy decisions related to the quality or level of maintenance desired.
5. Feedback reports to monitor and update the system.
6. Planning and scheduling procedures directed toward efficient use of resources.

A prime relationship of maintenance management to pavement management is to provide cost information associated with various levels of planning and programming, design, and construction. The type and degree of maintenance can also influence the rate of serviceability loss for a pavement.

The objectives of maintenance management may be more specifically stated as follows:

1. To plan, direct, and control maintenance activities so that an acceptable level of service, consistent with the class of pavement, is achieved.
2. To elevate the methods and materials used in maintenance so that economical and efficient practices are developed.
3. To acquire and report maintenance cost data so that unit costs for specific items for specific sections may be determined.

It has been emphasized that maintenance management requires careful planning and implementation, efficient reporting methods, easy information retrieval, and accurate assessment of maintenance practices and problems [RTAC 77].

Thus, the maintenance management system as a whole involves managing highway maintenance, which includes the pavement. The pavement management system involves managing the pavement system, including its maintenance. The two concepts are complementary. They do not interfere with each other; rather, they reinforce and supplement each other. In some organizations, pavement maintenance and rehabilitation will be handled through a pavement management concept. In others, the maintenance section will carry the prime responsibility, with input from the pavement management group. The pavement management system is a necessary tool for analyzing and predicting the effect of various maintenance and rehabilitation policies.

Further details about Maintenance Management Systems are given in Section 10.3.

10.2.3 Maintenance Policies

Policies for pavement rehabilitation and pavement maintenance vary widely from organization to organization, from place to place, and from time to time. Several factors usually govern these policies: (1) funds available, (2) historical precedent, and (3) organizational or political considerations. One or all of these items may be involved with a particular maintenance policy.

Funds Available

Pavement maintenance is a continuing process that applies to a wide range and amount of physical area. Both the time and geographic extent of its application mean that even low-unit-cost maintenance will accumulate rapidly. If a fixed maintenance budget is low, then available unit expenditure funds will be low. Safety policies usually dictate that available funds be applied to fill the most extreme needs, and these require more than a unit share of the budget. The remaining budget then proves inadequate to serve the total area involved. Consequently, routine maintenance usually suffers or is omitted altogether. The subsequent budget period usually shows that the pavement has deteriorated more rapidly than expected, as a result of the lack of routine maintenance; thus, more of the small budget is required for heavy remedial work, and the downward cycle of deterioration continues.

Budgeting for maintenance is an important part of pavement management. The reader will recall that the decision-making process in the original design of a new pavement or of rehabilitation involves an estimated maintenance input. It is difficult, however, to predict such fund availability for 20 or more years, the typical analysis period for a pavement.

A properly developed pavement management system is adaptable to this problem by reanalysis or updating at any time. In fact, with a working pavement management system such reanalysis should be done periodically to evaluate the real performance of the pavement under the conditions that actually exist in service, rather than the originally predicted design inputs.

Historical Precedent

Until recently, maintenance has not been a widely hailed and researched area of endeavor. As a result, many maintenance policies or activities have been tried and adopted without adequate proof of their general applicability or cost effectiveness. For example, a practice such as joint or crack sealing works well in wet-freeze zones and thus becomes standard policy in Ontario [Chong 87]. As another example, a precedent set by temporary budget restrictions such as no overlays shall exceed two inches may become accepted policy even when budget rules change and/or other, more critical situations come into existence.

Even though we often think of the highway system as being a homogeneous entity, it is certainly far from that and continually changing. For example, the interstate system in the United States was started in 1954, and a great deal of construction was completed in the early 1960s. At that time a very high-quality maintenance policy was adopted. The pavements were all new and the maintenance required was light. By the mid 1970's quite a different situation prevailed, and maintenance at the same high-quality level became extremely costly.

Organizational and Political Considerations

Over the years, road maintenance has suffered perhaps as much as any other part of the transportation field from political problems. In difficult financial times, maintenance is frequently one of the first programs sacrificed to resolve immediate budget needs. Organizationally, pavement maintenance is lumped with all right-of-way maintenance activities. Thus, if an unusual event occurs, such as a heavy snowfall winter, which requires a greater share of the maintenance funds than originally anticipated, the available budget for pavement maintenance may be substantially reduced. Adequate attention must be given to organizational considerations of maintenance, especially with respect to the overall problem of integrating the pavement management system, or for that matter, the maintenance management system. The organizational problems are two fold in that they are related to both time and space.

Time considerations relate to the fact that much of the activity and emphasis of the pavement management concept is applied at the planning and programming, design, and construction stages whereas the maintenance portion of the problem becomes critical and active at some later time in the process. This time delay tends to separate the earlier activities from the maintenance functions. It is important in pavement management to ensure against this separation and to assist in involving maintenance personnel in the pavement management team at an early stage.

Space considerations relate to the management function often being centralized in a planning or design office. Maintenance functions on the other hand are field-related. The traditional maintenance organization is decentralized, with maintenance foremen in charge of relatively small areas. This field decentralization is very

functional in ensuring that attention is given to all segments of the highway system. It also makes it considerably more difficult to ensure that there is central coordination of maintenance costs and practices.

Reanalysis at any Time

The concept of an analysis cycle can be expanded to show that it is possible to run a reanalysis at any time the pavement engineer chooses. Thus, if an unexpected increase in traffic or change in other factors becomes evident, it should be possible to rerun the analysis, particularly the deterioration and economic evaluation models, with input data including the projected traffic and the existing condition of the pavement as obtained from the appropriate monitoring methods. In some cases, unexpectedly good or poor performance of the pavement materials will make it desirable to run a reanalysis cycle even though the pavement may not have reached its minimum serviceability level.

To generalize, we may think of the pavement as a physical system that is functioning all the time. We may examine that system at any time with available evaluation techniques and the system analysis computer program. To expand the concept, the pavement evaluation techniques of all types become the monitoring methods for the physical system.

10.2.4 Effects of Policy Variations

It is one thing to establish rehabilitation and maintenance policies. It is quite another to ensure that these are followed in practice. The pavement management system has two concerns. The first is to provide information that will assist, if possible, in encouraging consistent handling of pavement maintenance with minimum variation from or in policy. The second, and perhaps more important, is to provide a realistic method of considering and evaluating the effect of variations on the life and economy of the pavement sections on an individual basis.

The controlling factor of which is the better maintenance policy to follow is usually money. Policies are set over the long term; budgets are usually developed annually. Thus, a policy may call for both cleaning culverts and fog sealing the pavement every year. If funds are not available for both, then the person in charge of the field level will likely make a personal preference choice in spite of policy.

Another important factor is the condition of the pavements within the maintenance sub-unit. In the best-devised budgeting techniques, "average" conditions will usually be considered. Local personnel often expend excessive funds on pavements in bad condition at the expense of possible preventive maintenance on other sections of pavement. Similarly, because a great deal of supervisor approval will be based on current inspections and how things look, there may be a tendency to overspend on visible maintenance needs such as mowing and trash pickup, at the expense of preventive pavement maintenance that has been programmed and considered in the life and performance of each pavement section.

Overall Policy Shifts

Another major aspect of maintenance is the result of some overall shift in policy by the central agency involved. Such a situation occurred for many state highway departments in the mid 1970's when the oil crisis and poor economic conditions combined to drastically increase costs and reduce available funds. This situation occurred again in 1983 and in 1990-91. It is even more pronounced in foreign countries. For example, in Brazil in 1975 the available maintenance budgets shrank to near zero because more than 70 percent of all oil products are imported, and increased oil prices created a major financial crisis. The decision was made that continued development of new roads is more desirable than maintenance, with the hope that there would be more benefit to the overall economy.

It is highly desirable to apply a pavement management system to assist in making these decisions rather than to make them ad hoc.

Analysis of Effects of Policy Changes

An important function of a pavement management system is to assist with the problem of analyzing policy shifts or required variations from policies or plans. In such cases, it is mandatory that each section of the pavement network

be reanalyzed to see predicted effects on the expected life. This can be done on a regular basis and can provide invaluable information for use in showing budgeting agencies the true effects of stringent budgets. In many cases, the investment in existing pavement must be protected with maintenance or rehabilitation, or the resulting long-term loss is immense.

Finally, it is essential to reanalyze any given pavement in terms of original programming of rehabilitation and maintenance. If budget constraints are going to interfere with programmed work, then studies can and should be made to evaluate the effects and identify possible alternative actions.

10.2.5 Costs, Economics and Decision Criteria

The entire pavement design and management concept is related to costs and economics. In the pavement management process, the costs that must be considered include not only initial cost, but also routine maintenance costs, and user costs. User costs are those costs that the pavement user pays, both directly and indirectly, in relation to the pavement facility or lack of it. User costs are related primarily to a pavement in poor condition that results in excessive roughness. A second major user cost is related to the detour and delay cost that the user suffers with relation to time required for maintenance and rehabilitation of a given facility. Considering these costs, along with the initial construction costs and the time value of money, makes it possible to evaluate true relative costs of various pavement maintenance and rehabilitation strategies and to select those that are optimal.

Timing

The second major aspect of economics requires that there be good coordination between the various types of maintenance and rehabilitation. Thus, if a pavement has inadequate friction and a maintenance treatment, such as a seal coat, seems warranted it should not be applied until a structural evaluation determines that a structural overlay is not required. Without this coordination, the seal coat might be applied one year, and then a major reconstruction or overlay required the next year would obviate the value of the prior seal coat.

Excessive Costs Criterion

A final important aspect of economy is the concept of excessive maintenance costs. It is possible to extend the life of a severely distressed pavement by providing extensive, heavy maintenance. This is sometimes done when a major highway is suffering damage but available funds are inadequate for rehabilitation. Figure 10.1 illustrates such a concept where a pavement is at, or very near, it's unacceptable level but remains slightly above it because of the amount of maintenance expended. Keeping the deteriorated pavement above the minimum acceptable serviceability requires acceleration in maintenance expenditures to the point where the costs become unacceptable.

Thus, a pavement might be considered "failed" when it reaches an unacceptable level of serviceability or when the cost of maintenance becomes excessively high. This might be controlled in an actual field situation by programming maintenance funds according to a predetermined formula. When it becomes impossible to maintain the road adequately for the formula funds, then a detailed evaluation should be performed to determine the problem.

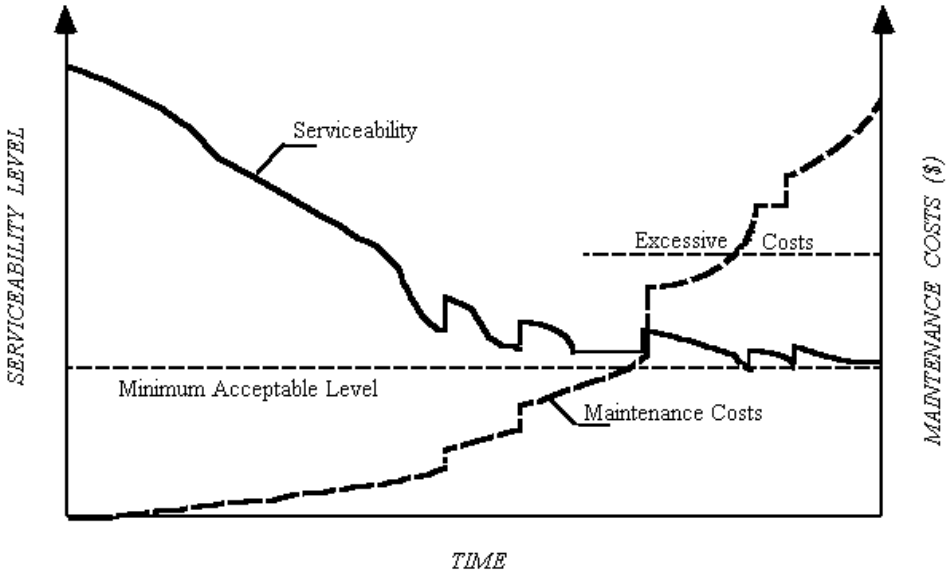


Figure 10.1 Sample Pavement Record Illustrating Failure or unacceptability due to Excessive Maintenance Costs

Evaluation Decision Criteria

The pavement evaluation process includes decision criteria for threshold levels (minimum or maximum) of each of the items being monitored or evaluated. In some cases the threshold levels act directly as trigger values, providing information on which the required action can be based. A minimum acceptable level of serviceability is an example of a trigger value where an agency policy might establish a minimal level of 2.5 for interstate highways. Pavement sections dropping below this level would then require a specific type of rehabilitation or maintenance.

Other types of decision criteria may be more flexible, either to serve as indicators to the engineer or to be varied with a particular pavement design as needed. Thus, in the first instance, the amount of cracking in a particular pavement section may have no absolute limit, but as cracking progresses it can be of significant concern to the engineer and indicate that rapid changes are taking place and that structural evaluation is needed. In the case of adjustable criteria, a particular pavement design method might predict an acceptable deflection of 0.015 inch for a particular section. Assume that subsequent evaluation then indicates a deflection significantly above 0.015 inch., say 0.020 inch. This indicates to the pavement engineer that the pavement is behaving differently than expected, and thus a more complete evaluation and/or analysis seems justified. In each case, three possible paths may by followed: (1) continue maintenance for predicted life as programmed; (2) perform a more complete evaluation and analysis of the pavement section to provide a better basis for decision; or (3) take immediate appropriate rehabilitation or maintenance action as required based on the violated criteria.

Decision criteria for selecting a maintenance treatment can vary with highway class. Figure 10.2 [Jorgensen 72] describes example criteria, used in actual agencies, for rutting and waves, sags and lumps (Agency C), and rutting (Agency F).

Combinational Decision

Certain highway agencies combine two or more of the measures previously outlined into a single index. Such combined indexes can prove very useful in overall analysis of a pavement network and in providing a simple method for ranking pavements into relative condition categories. But combining the information is not very useful for evaluating and analyzing a specific pavement section for maintenance. In that case, specific disaggregated information (i.e., distress, roughness, etc.) is needed.

10.2.6 Maintenance Information Needs

Maintenance feedback information is vital in any pavement management system. It invariably finds its use in pavement rehabilitation and maintenance. The original analysis of a given pavement section incorporates a particular expected maintenance schedule.

If there is any variation from the mean or expected value, then the serviceability history of the pavement section may vary from the predicted value; that is, the performance curve may drop more rapidly or more slowly than predicted. When this happens, as shown in Figure 10.3, it is essential to have a reanalysis of the project needs for maintenance and/or rehabilitation.

1.200 FLEXIBLE AND RIGID PAVEMENT MAINTENANCE

The purpose of this standard is to establish the guidelines by which the roadway surface of the interstate and other functional classes of highways shall be maintained and to establish the degree each type of distress can be tolerated before remedial physical maintenance measures must be taken.

1.210 FLEXIBLE PAVEMENTS

1. Rutting

Rutting will be tolerated to the degree specified in the table below. Where rutting occurs in excess of that specified, the deficiency shall be corrected at the earliest opportunity.

	Interstate	Principal	Major	Collector	Other
Maximum allowable depth of rut on multilane or 70 mph highways	½ in.	½ in.	½ in.	½ in.	½ in.
Maximum allowable depth of rut on two-lane or 60 mph highways	½ in.	½ in.	½ in.	¾ in.	1 in.
Maximum allowable depth of rut on bituminous treated roadway surfaces	½ in.	¾ in.	¾ in.	1 in.	1 in.

2. Waves, Sags, and Humps

These types of flexible pavement distresses add to the discomfort of the road user and can become a hazard if allowed to become severe.

On all functional classes of highways where 70 mph speed limits are allowed, repairs will be made wherever 50 percent or more of any given 100 feet of roadway is observed to exhibit characteristics of waves, sags, or humps in excess of one inch in height per 10-foot section.

All other roadways with posted speed limits of 60 mph or less will be considered for repair when observed to exhibit these characteristics in excess of two inches in height per 10-foot section except that waves, sags, or humps four inches in height or over will be corrected as soon as practicable after reported or observed.

(a) Agency C

Figure 10.2 Sample Codified Maintenance Levels: (a) Agency C

ROAD CONDITIONS

- Rutting at driveways, mailbox turnouts, rural intersections, etc.
- Edge ruts at curves and other isolated locations.

DESCRIPTIONS

Rutting at Driveways, Mailboxes, Rural Intersections, etc.

These are specific places where traffic goes into the shoulder often and where the shoulder can't take it.

Edge Ruts at Curves and Other Isolated Locations

These are places where traffic goes off the actual surface and beats the shoulder material away leaving an edge rut.

NEED FOR REPAIR

This work can be done on an inclement weather schedule. It should be done whenever conditions show a problem. On edge ruts, let them get at least one inch deep before repair, and catch them before they get about two inches deep.

HOW TO REPAIR

Spot dump and hand spread material. If there is very much to be done, it may be wise to schedule a reshaping at the same time.

Try to compact gravel material into place using the truck wheels or a portable roller.

HOW MUCH TO DO

Patch only the specific locations that show a need. Do not try to rebuild a shoulder by this method; you are usually working with places that repeatedly have trouble, so put out a little extra gravel when you do this job.

(b) Agency F

(Figure 10.2. Continued.)

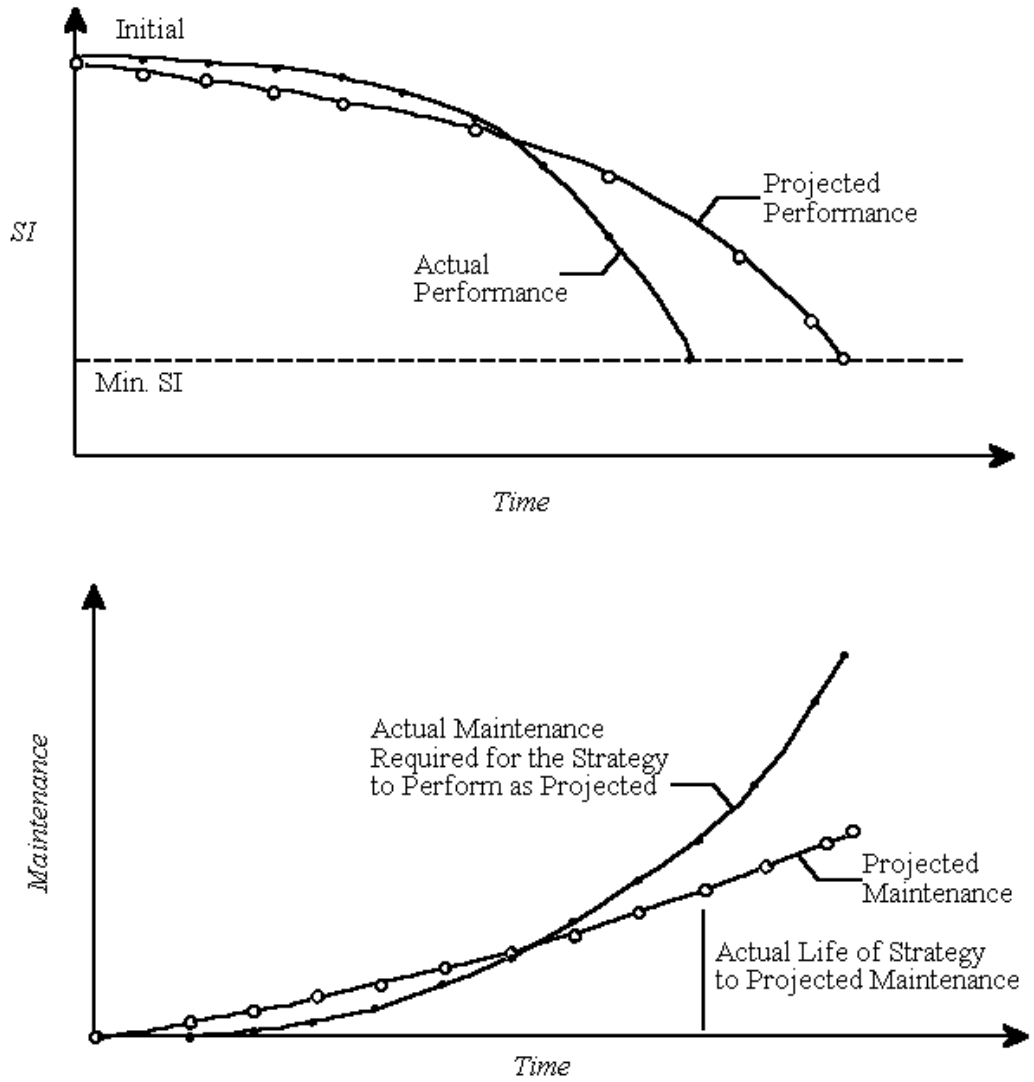


Figure 10.3 Plots of Projected and Observed Histories

The availability of feedback data from the pavement database is important to maintenance for the following reasons:

1. It provides the maintenance administrator with important data and information on trends applicable to programming and planning maintenance expenditures.
2. It permits evaluation of the validity of existing maintenance models.

Figure 10.3 illustrates the use of a maintenance data system to evaluate models by plotting deviations in predicted histories. Such information is extremely important to a system designer in determining how the formulated models are behaving.

Some discussion of maintenance data requirements is useful at this point. As a first step, pavement damage prior to performing maintenance should be recorded. This can be done on a sample basis or on a complete section basis depending on time and personnel available. It has been pointed out that the sampling technique is usually adequate, considerably cheaper, and more practical than a complete survey [Butler 74, 75a, 75b]. This is a feasible approach where sections are homogeneous with respect to design, year of construction, traffic, subgrade soil, and general environment. Such designated homogeneous sections should be the same for all pavement management activities,

and be given a control or reference number (actual locational referencing may be for example by milepost, or Geographic Information System, GIS).

The length allocation should be referenced and recorded in such a way that any point within the section can be located with reasonable accuracy.

The second important step involves actual units of the various type of maintenance performed, and the related costs, as subsequently discussed in more detail. The actual maintenance performed can be obtained from file records in the maintenance residencies if the maintenance foremen are encouraged to provide accurate records of the work accomplished.

Unfortunately, there are many pressures on maintenance personnel that may inadvertently encourage them to "generalize" records to average values rather than actually to record the work required on a bad section. Human nature does not, for some reason, like to report extreme conditions within a set of controls.

The costs of the performed maintenance are even more difficult to obtain. Actual expenditures may vary over quite wide ranges depending on material costs, unit prices, work schedules, efficiency, and so on. But it is not sufficient, for example, to record average patching costs over a five-mile section when most of the total cost may be incurred over only a small portion of the section. Where it is extremely difficult or impossible to obtain true costs from existing records, it may be necessary to apply average unit prices to the work units performed and recorded in the field records.

In every case it is essential to work toward true records rather than to accept unrealistic averages because it is actually these variations from the averages that are important in the maintenance management and pavement management systems.

Work Activities and Accomplishment Units

This section began by considering maintenance information needs in general terms. In a more specific sense, the component work activities, and the accomplishment units in which they are measured, have to be identified and properly recorded in order to fulfill the information needs. It is, of course, essential that both the work activity and the accomplishment unit be clearly understood by the field-level people (i.e., those assigned to handle specific tasks).

Table 10.1 [Jorgensen 72] contains a typical listing of major work activities and associated accomplishment or work units. It covers the overall highway maintenance function, and therefore more than just pavement maintenance activities are included.

Each agency would have a set of forms for recording daily, weekly, biweekly, etc. the activities listed in Table 10.1. A typical set of such forms appropriate to the pavement part of maintenance is contained in [RTAC 77].

Guidelines for performance standards (i.e., accomplishment units expected) previously noted in this section should be established for each work activity. Analysis of performance achieved can be used to update existing standards.

Uses of Information by Other Areas of Pavement Management

It is perhaps self-evident that maintenance data may be used to varying degrees by other phases of pavement management. In order to do this effectively, the separation of maintenance and construction costs must be clearly identified, because the designation of various items and quantities as construction or maintenance can vary from agency to agency.

When a maintenance management system is functioning effectively, and proper records have been kept, analysis of the data can point out inadequate designs, inaccurate traffic estimates, inaccurate evaluation of materials, construction problems, and so on. A summary of how maintenance information can be used for such other pavement management activities is given in Figure 10.4 [RTAC 77]. For example, in addition to the forgoing analysis, maintenance data may be used in planning and programming and in design for future cost estimates.

Table 10.1 Illustrative List of Major Work Activities [Jorgensen 72]

	Maintenance Activity	Work Unit
Roadway surface	• Patch with premix	• Tons of premix
	Level with premix	Tons of premix
	Deep patch with premix or full depth replacement of surface course	Tons of premix
	Skin/spray/surface treatment patch	Cu. yd. of aggregate
	Fill or seal cracks with joints	Gal. of sealant
	Seal coat or surface treat full lane widths and in continuous form	Lane mile of surface
	Patch with P.C.C.	Cu. yd. of P.C.C.
	Blade or reshape unpaved surface	Road mile bladed
	Patch unpaved surface with gravel	Cu. yd. of material added
Shoulders and approaches	• Blade or reshape unpaved shoulders	• Shoulder mile bladed
	Patch with aggregate or other stabilized material	Cu. yd. of material added
Drainage	• Clean and reshape ditches	• 100 lineal feet of ditch cleaned
	Clean and repair drainage structures	Each installation
	Replace minor structures	Lineal foot of structure
Roadside	• Mowing	• Acre mowed
	Chemical control of vegetation	Gallon of spray
	Cut brush	Acre mowed
	Cut/trim trees	Each tree
	Rest area maintenance	Each rest area
	Erosion Control	None
	Litter pickup	Mile of R/W
	Fence repair	Lineal foot of fence
Major structures	• Bridge inspection	• Each bridge
	Structure painting	Gallon of paint
	Seal or repair expansion joint	Lineal foot of joint
	Other structure repair	None
Snow and ice	• Snowplowing with/without sanding	• Lane mile plowed
	Snow removal loader/blower	Cu. yd. of snow removed
	Spot and/or continuous sanding	Cu. yd. of sand
	Spot and/or continuous salting	Cu. yd. of salt
	Stockpiling sand or salt	Cu. yd. handled
	Erect, remove, repair snow fence	Roll of fence (50’)
	Clean up after storm	None
Traffic services	• Sign maintenance of any kind	• Each sign
	Lighting maintenance of any kind	Each installation
	Painting guideline	Pass mile striped
	Paint pavement messages	Each site
	Guardrail maintenance	Rail section worked on
	Ferry operation	Mile of operation
	Movable bridge operation	Each opening
	Road patrol	Odometer mile
Extraordinary maintenance	• Unusual or disaster maintenance	• None
Service functions and overheads	• Crushing/mixing	• Cu. yd. of material
	Stockpiling	Cu. yd. handled
	Equipment service and repair	Each vehicle
	Housekeeping	None
	Buildings and grounds maintenance	None

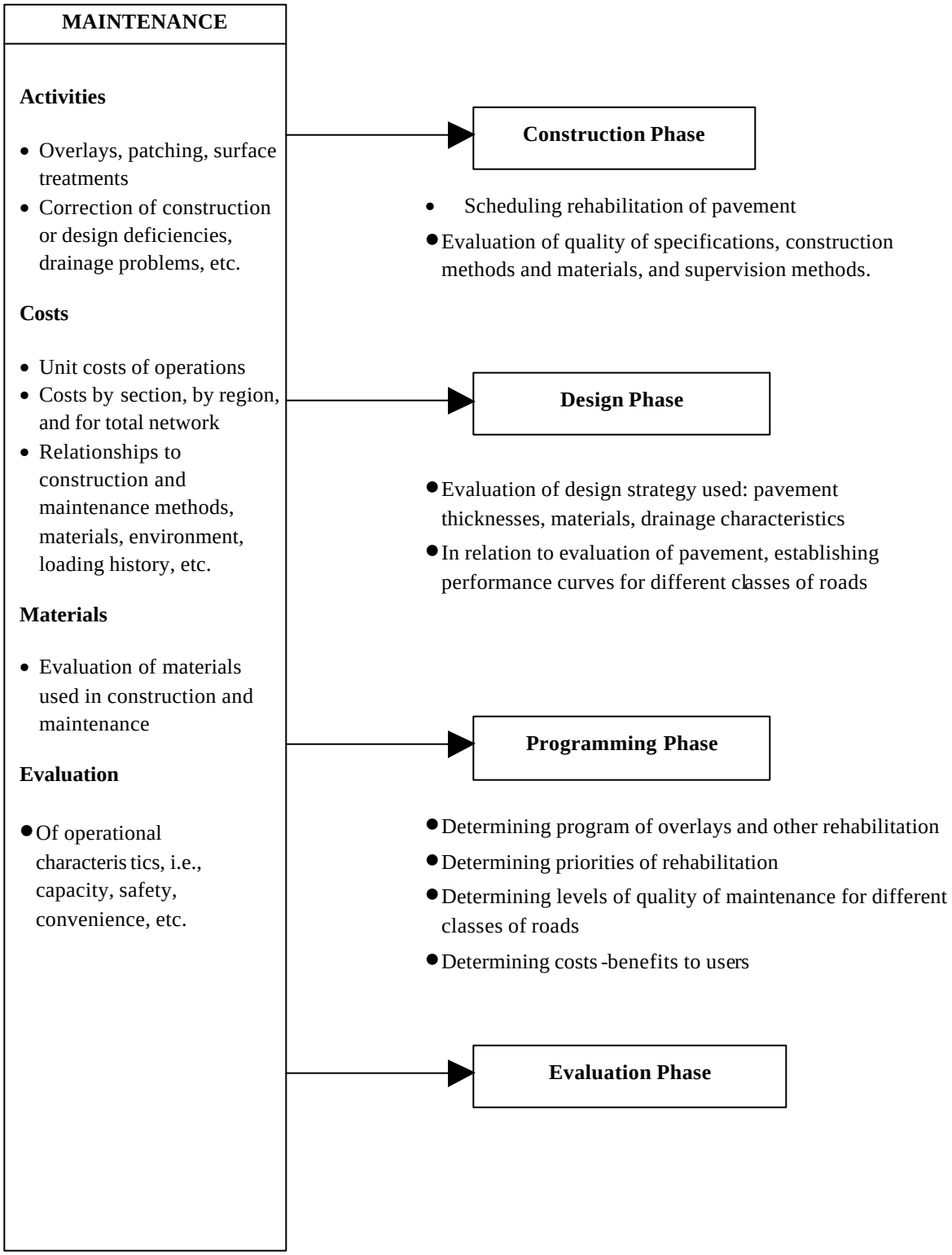


Figure 10.4 Relationship of Maintenance to other Phases of Pavement Management

10.2.7 Summary of Maintenance

This section reviews the major components of maintenance practices and management that relate to the pavement management system. Type and degree of maintenance can significantly influence pavement performance and the time and type of any major rehabilitation required.

Maintenance management, if conducted properly, should be capable of determining if the maintenance activities are being carried out in the most economical and efficient manner. In addition, the records kept should be capable of providing information to the other phases of pavement management.

The basic components and functions of maintenance management systems have been identified and discussed in the section. Particular attention has been given to the levels of maintenance management, to maintenance policies, to the effects of policy variations, and to costs, economics, and decision criteria associated with maintenance management.

Information needs associated with maintenance have also been given particular attention. In addition to the use of the data generated for maintenance management purposes, it is necessary and sometimes essential for other phases of pavement management. Although there are relatively well-developed methods available for collecting and reporting maintenance data, considerable work is still required for proper use in these other pavement management phases.

10.3 OVERVIEW OF MAINTENANCE MANAGEMENT CONCEPTS AND PMS INTERFACE

A good Maintenance Management System provides a wide range of capabilities in a friendly, easy-to-use graphical interface. The system was designed to help users at all levels visualize the raw data and summarize information in an organized way. Graphic representations are presented where possible to give the user a birds eye view of all maintenance information with easy access to details as necessary. The system should operate in the latest graphical computer environments including Windows 95/98® and Windows NT®. Visual/MMS may be operated as an independent maintenance management system or within an integrated infrastructure asset management suite with other management systems.

The system is organized into several modules. The Network Planning module addresses the global needs of the full network or any sub-jurisdictional network of an agency. It may reside at the central office and district headquarters office levels. The Work Management module manages the day-to-day process of programming, scheduling, performing, monitoring, and completing all maintenance work performed under the jurisdiction of the agency. The Contract Management module manages the day-to-day process of monitoring and reporting contracted maintenance work. The Work Management and Contract Management modules may be used at the district level and the individual work area levels. The Labor, Equipment, and Materials modules are highly flexible providing for full management of these resources or when appropriate, offer an interface to other systems containing this information.

Network Level Planning and Analysis Overview

The network level planning analysis allows for an overview of whole network maintenance requirements accounting for multiple jurisdictions at multiple organizational levels. Sophisticated planning tools are incorporated to provide planners a prioritized distribution of available budgets by jurisdiction, functional class, maintenance activity, and other factors as desired by an agency. In addition, it provides a complete assessment of maintenance needs and backlog for the network.

The system allows the user to adjust maintenance service levels and evaluate the effects of various maintenance policies on overall budgetary needs. The analysis accommodates observed and predicted defects, frequency based activities, and accumulated backlog. The user has a wide array of controls over many important factors in the analysis to allow informative sensitivity and scenario analyses to be performed. The Network Planning module provides managers a valuable tool for summarizing all detailed maintenance information. This information can be used to model and analyze critical factors affecting decisions on allocating maintenance funds.

Project Work Management Overview

Project level work management involves all routine functions for comprehensive management of maintenance personnel, equipment, and materials required to perform force account maintenance. In addition, it has an advanced Contract Management module for administering maintenance work by private contractors. The system tracks individual defects and frequency activities from initial reporting to work completed to quality assessed. Quality assessment is a useful tool for both force account and contracted maintenance activities. Powerful analysis tools allow easy tracking of costs and quality with an assessment of the efficiency of each contractor relative to each maintenance activity. Summarized information from the Work Management and Contract Management modules provides timely feedback into the Network Planning module to update predictive models, unit costs, and performance standards.

Labor, Equipment, and Materials Overview

A good MMS offers a wide range of interface configurations regarding use and/or exchange of information concerning labor, equipment and materials resources. These different configurations range from an installation where MMS is used in a stand-alone configuration to other installations where data sources outside of MMS provide all Labor, Equipment and Materials basic information.
The capabilities and data interface extent for Labor, Equipment, and Materials information is configured for each agency based upon their needs, integration with other data sources, or system implementation plans.

When MMS is used in a stand-alone configuration, the Labor, Equipment and Material Modules are activated in their entirety. These modules allow direct data entry defining personnel and their skills, equipment and capabilities and materials types as well as maintaining their stock, ordering materials, etc. However when an agency has other databases and systems for managing all this basic information, the functionality of the Labor, Equipment and Material Modules changes (and from a users' perspective could possibly be replaced by a single Labor, Equipment and Materials Interface (LEM Interface) module or its equivalent). The LEM Interface extracts/exchanges information from the other computer systems and organizes it for display to the user. The user then has access to the LEM basic information to use and assist with project work management planning and reporting but does not have the privilege to edit the data.

10.3.1 Road Network Definition

Visual/MMS allows agencies to define their road network according to their organizations hierarchical structure, location referencing, and other attribute data. An inventory of maintainable elements can be defined and maintained by the user. A level of service or other similar survey can then be performed on the inventory of maintainable items. This inventory is then available for identifying work to be performed and establishing current condition distribution conducted by many agencies. The inventory and Level of Service (LOS) data can be used to assist in developing maintenance contract needs and identifying work for the agency's maintenance crews. The inventory and LOS data is also used in the network level analysis module for developing LOS/Cost models and performing network analysis.

Maintenance Control Section

A set of maintenance control sections may be defined in any manner desired by the agency, which completely defines the network under the jurisdiction of the entire agency. At the work management level, viewing is usually password-restricted to portions of the network within the user's jurisdiction. All information maintained at the maintenance control section level is defined for the agency during system configuration. Agency-specific classification, organization, functional, and other types of data may be incorporated. The specific maintenance organization location referencing method is displayed and cross-referenced throughout the system for ease of use and familiarity by all agency maintenance personnel.

Control sections provide a simple method to organize data on contiguous and similarly treated roadways. Figure 10.5 shows the form view of the maintenance control section definition window. The table view tab allows viewing multiple control sections in a spreadsheet-like view. Both views have convenient find, filter, sort, and query features to allow quick easy access to specific sections.

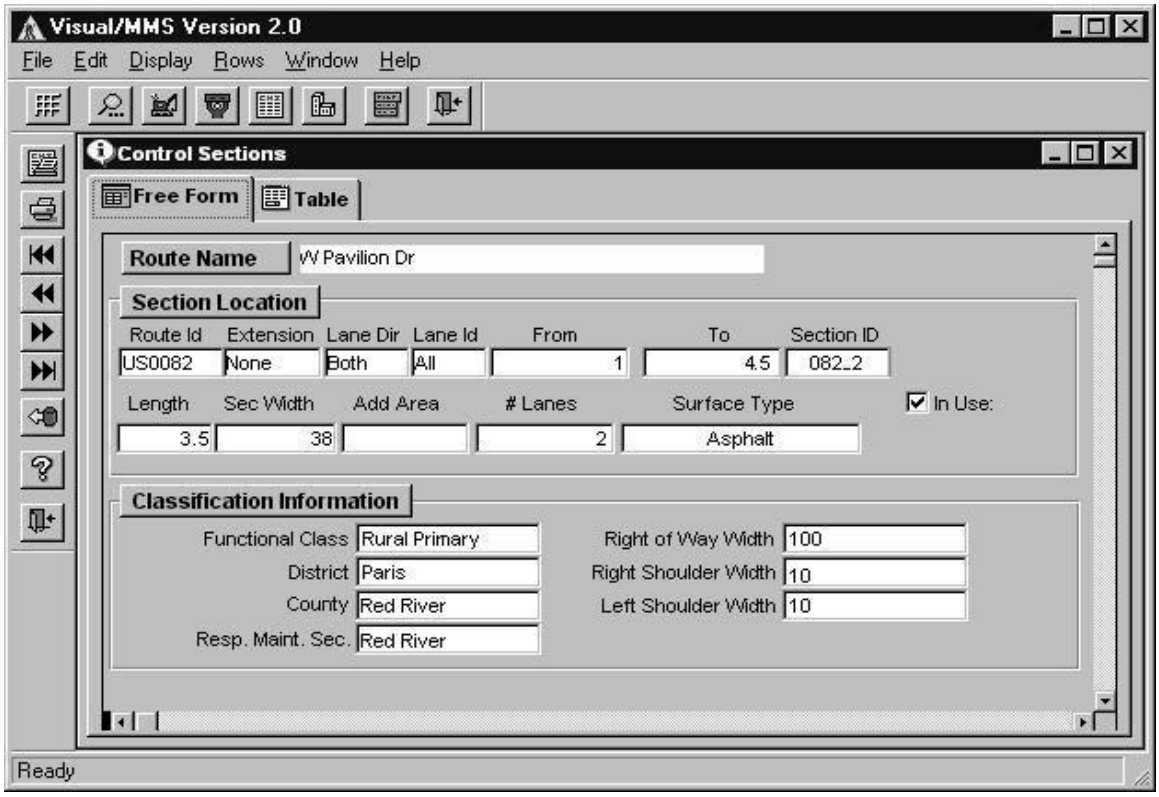


Figure 10.5 Form View of the Maintenance Control Section Definition

Condition Assessment of Maintainable Inventory

A good MMS allows the agency to define, establish and update the condition of the maintainable inventory elements. Condition of the maintainable inventory can be recorded along a route according to user defined defects. The agency can define as many defects as desired to report condition or assist in planning routine maintenance. Once all defects are defined, the Defect Survey window, is used to record and display defect survey information. The tabbed window allows the user to select a defect to review, enter the condition information using typical Levels of Service definitions Very Good through Very Poor, view the condition state distribution and get feedback from how much of the system has been surveyed.

Once defects have been recorded they are available to users at various levels for different applications. The field offices can use the defect survey to assist in planning routine maintenance activities and contracted maintenance. In network level analysis, the defect survey can be used for generation of annual level of service reports to the agency administration and elected officials, used for developing the Level of Service Cost (LOS/Cost) models in network analysis.

10.3.2 Project Level Management Modules

The Work Management and Contract Management modules provide maintenance organizations a tool for day-to-day use in assigning resources and managing their work operations for both force labor and contracted activities. The system provides details of labor, equipment, and material allocations and use on maintenance work for all force account maintenance across multiple organizational levels. It also has a full contract management module to manage maintenance work performed by private contractors. Information concerning the Labor, Equipment and Material modules that provide information for the labor equipment and materials are discussed in the Labor Equipment and Materials section of this document.

The key features in the Project Level Management modules include:

1) an inventory of maintainable elements,
2) a set of in-house and contract maintenance activities to perform on the maintainable elements,
3) a set of specific defects, by section, that are reported to the system in some manner,
4) the ability to optionally schedule maintenance work at any time in the future,
5) record work performed by force account labor, equipment and materials, and
6) the ability to assign contractors to road sections for contracted maintenance.

Scheduling, Managing and Recording Maintenance

A good MMS is flexible in how maintenance activities are managed and recorded. While it has all the tools necessary to identify the maintenance inventory, record maintenance surveys, schedule crews and issue work orders it can also function as a cost recording system where nothing more than the work performed by the force labor, including equipment and materials or contractors is recorded. This flexibility provides for varying levels of sophistication among the users. The need to perform maintenance is generated several ways, in-house through maintenance supervisors and engineers assigning work to crews or contractors and through the Public Complaints / Emergency maintenance process.

Maintenance by Work Orders

Equipment assignment, materials assignment, and maintenance performed by force account labor are handled through a sophisticated system of work order issuing, scheduling, and completion functions. Service requests can be organized and presented to the manager for issuing work orders. MMS facilitates updating work accomplished including the details of labor, equipment, and material quantities used each day. This information is tracked, summarized, and provided as feedback to update the predictive models and performance standards in the Network Planning module.

Network Analysis Component

Network analysis is a means of predicting costs for certain given threshold levels of service in the different asset categories, or conversely, to distribute a budget optimally between a number of asset categories to maximize level of service. MMS allows the user to calculate and review the result of many different network analyzes called

scenarios. The user can then develop a maintenance work program based upon output from one or more scenarios. There are several setup windows used to define parameters used in calculating the scenario work programs.

10.3.3 Network Level Planning and Analysis Module

The Network Planning module is intended for use at a central office and district office level to plan maintenance funding and distribution for a number of maintenance areas. The planning function requires less detailed information than work management and incorporates general predictive analysis techniques. It is a higher level module for assessing performance and levels of service (LOS) of specific assets over the entire network, for assessing past and future policy, and for planning and allocating budgets The Network Planning module is divided into two components Network Analysis and Performance Modeling.

The network analysis component helps maintenance managers to determine overall maintenance needs across network and to prioritize limited funding addressing needs most appropriately. Budget allocations by district, maintenance area, functional classification, and defect are made by the system based on numerous factors controlled by the user in the definition of analysis scenarios. Managers can predict costs for certain given threshold levels of service in the different defect categories, or conversely, to distribute a budget optimally between a number of defect categories to maximize level of service.

This analysis is made possible by use of models, which give the cost to attain specific levels of service in the network for specific defects (such as road surface roughness). These LOS/Cost models are generated in turn using current annual costs and the current distribution of the LOS over the network. The distribution may come either from an actual survey or from deterioration models.

In order to conduct a survey it is useful to know the location and amount of element types within the network. The network is thus defined by it's Element Types and Defects such that each Element Type may be subject to a number of possible defects. Given this structure, an inventory of the amount of each element type in user specified sections of road may be compiled and maintained.

Network Planning Database

Network maintenance needs are estimated based upon 1) the defect survey distribution and LOS/Cost model applied to the inventory of maintainable items or 2) the default defect models used to develop LOS/Cost models. To estimate network maintenance costs a model is expected for defect development rates for all defined defects. The models are used to estimate the quantity of each defect expected to occur over the network in the coming year.

Inventory data may be entered, displayed, and tracked in a graphical interface. The data displayed in this window is used in the Analysis module for identifying the quantities of maintainable elements analyzed in network scenario analysis.

Defect surveys are used to calculate current condition state distribution. The condition state distribution and LOS/Cost models provide the information necessary to estimate future defect development rates for network planning. Defect surveys may be a snapshot of the detailed work management surveys or may be an independent survey performed at the network level specifically for planning and quality assurance purposes.

Network Planning Analysis

MMS provides a tool for maintenance planners to evaluate their network's overall maintenance needs and budget requirements. The system provides prioritization of limited budgets or the user may set maintenance service levels over a wide range of variable factors to estimate overall needs based on various maintenance service level policies. The system evaluates the effects on budget requirements, funds distributions, and relative maintenance index values.

The user defines and executes network analysis in a Scenario Defect Analysis Window. Detailed analysis scenarios may be defined by the user, retained by the system, and accessed through this window. The tabs along the top of the

window provide quick access to other parameters such as maintenance service level settings and results from analysis. After an analysis has been run, additional tabs show resulting recommended budget distribution and priority results including graphs to provide a quick overview of the effects of each network maintenance analysis scenario.

10.4 INTEGRATION OF MAINTENANCE IN MONTANA'S PMS

10.4.1 Why Maintenance?

Maintenance is a big concern in Montana as in other states. However, many of the highways in Montana are low volume and therefore, rank lower in priority than other high volume roads. Consequently, the maintenance division expends a large percentage of its budget for maintenance efforts in order to keep the roads in serviceable condition.

Despite the importance of maintenance, there has been a problem in gaining agency and upper management recognition and funding for maintenance efforts.

In the recent past, Montana's maintenance budget was extremely low at $2 million per year in comparison to the construction budget and the amount of miles needing maintenance. The maintenance division began applying systematic procedures following maintenance management and pavement management practices to help improve the understanding and reporting of maintenance information in order to show maintenance effectiveness. The result was an additional $13 million biennial appropriation by the state legislature specifically for maintenance of roads. Maintenance is now a key pivotal player for MDT's mission to preserve the aging existing infrastructure.

Because maintenance is now a key player, a systematic analytical process must be applied based on high quality information. In addition, the quality of maintenance materials and workmanship should be high such that the money spent provides the most effectiveness and greatest extension of roadway life. Such application of systematic techniques and high quality procedures should show a valuable contribution of maintenance and the need for research and technology innovation in improving the overall maintenance process.

The PMS unit believes that primary consideration for maintenance should be given to keeping the roads that are on the verge of structural deterioration well maintained (i.e., preventive maintenance) in order to allow the construction division to concentrate their efforts where it is most useful (i.e., large projects on roads that have failed structurally). The PMS unit in Montana is operating with the understanding that the public wants good value for their tax dollar.

Preventive maintenance should be thought of as preventing a premature failure of the road. Preventive maintenance has the ability to retard the process of deterioration and correct flaws, which could cause untimely problems. The goal of preventive maintenance is to preclude further damage that completely destroys an entire pavement structure and with it the economic investment. [MTD 94]

It is important, in presenting these examples of maintenance as related to pavement management, to put maintenance in its overall context as well. Maintenance covers more than just pavements. And while we are primarily interested in maintenance as related to pavement management and the effects on pavement performance, it is also important to understand how pavement related maintenance activities can be affected by the other duties of maintenance crews and maintenance operations. For example, in Montana snowplowing takes a large portion of the maintenance division's budget. In addition, this is an unpredictable item that in some years could possibly take the entire maintenance budget and leave little available for even emergency response maintenance to items such as spring break-up after the frost leaves the road. This also indicates that the maintenance division and maintenance crews must be flexible in their approach to maintenance activities. The pavement management system and its information give MDT this ability.

In the update of the PMS construction records, maintenance reports are used as shown in Table 10.2. As the MDT continues to update PMS construction records with these maintenance records, they will be able to develop more reliable performance models. Maintenance work for overlay, chip seals, and other thin surface treatments will be

input into the PMS database as they occur. Any significant work to the pavement surface that is conducted as maintenance account work will be studied for future PMS construction record updates.

Maintenance information is recorded by direction and lane to get adequate detail for engineering evaluations. The data can then be correlated to the appropriate pavement management section's historical data including construction history as well as condition and traffic history currently available in the pavement management system.

A new maintenance management system is in the early planning stages. The PMS section is working closely with maintenance management to automate the insertion of this maintenance work into the PMS history file.

10.4.2 Understanding Maintenance Effectiveness

Montana is interested in evaluating maintenance effectiveness for all of their force account work, including crack sealing, fog sealing, chip seals and thin overlays. All maintenance work is identified as to where the work is done, its cost center, and the amount of improvement that the maintenance treatment had on an individual roadway. One of the engineers assigned to the pavement management unit does spot checks on maintenance activities to determine how well each treatment is performing. These are audit checks that are performed in conjunction with checking condition survey results each year.
Exception reports are generated when the overall pavement index (OPI) goes up but no maintenance or construction was recorded. Their objective is, for all maintenance activities, to record what, when, where, and how good.

The primary Montana example involves the use of maintenance history data and pavement management analysis capabilities to estimate the effectiveness of maintenance activities for use in improving maintenance planning and programming. This is an engineering application because it involves the organization and detailed analysis of section specific PMS and MMS data. It then involves the engineering analysis in terms of performance evaluation in a manner that considers the application and effects of each maintenance activity in the overall performance of individual pavement sections and on group models derived from pavement performance analysis.

The location of maintenance activities is now reported down to 0.1 mile, by route. This provides the level of engineering detail required to use maintenance data in conjunction with pavement management analyses. The level of maintenance currently in focus in the PMS unit of Montana DOT includes primarily substantial maintenance treatments such as medium and thin overlays, chip seals, blade patch/overlays, and crack seals. Clearly when maintenance forces are applying many thin overlays, throughout a pavement network, it is imperative that the pavement management system that does performance modeling know that this work is being done. If the data does not reflect these substantial maintenance actions, the performance modeling analysis using this information in conjunction with data from pavements that have not had multiple overlays will produce generally incorrect models.

Table 10.2 MDT Pavement Management Maintenance Report

ROUTE _____	DIVISION _____	PROJECT WIDTH _____
MMS COST CENTER _____	DATE SENT _____	DATE RETURNED _____

OVERLAY	CHIP SEAL	OTHER SEALS	CRACK SEAL
CONTRACTOR ? MAINTENANCE ?	CONTRACTOR ? MAINTENANCE ?	CONTRACTOR ? MAINTENANCE ?	CONTRACTOR ? MAINTENANCE ?
CONTRACTOR	CONTRACTOR	CONTRACTOR	CONTRACTOR
MIX DESIGN Yes ? No ?	MIX DESIGN? Yes ? No ?	ASPHALT TYPE	CRACKS ROUTED? Yes ? No ?
PLANT MIX ROAD MIX ? RAP ?	ASPHALT TYPE	APPLICATION RATE GAL/SY	CRACKS CLEANED? Yes ? No ?
LAYDOWN MACHINE ? BLADE ?	APPLICATION RATE GAL/SY	ASPHALT SOURCE	MATERIAL USED _____
ASPHALT TYPE	ASPHALT SOURCE	SEAL TYPE	QUANTITY Gal./Mi
ASPHALT SOURCE	CHIP GRADE		
QUANTITY	CHIP APPLICATION RATE LB/SY		
THICKNESS	CHIP SOURCE		
MILLING ? LEVELING ?	ESTIMATED CHIP LOSS (%) _____		
DATE(S) OF WORK _____	DATE(S) OF WORK _____	DATE(S) OF WORK _____	DATES OF WORK _____
WEATHER CONDITIONS (avg.) _____	WEATHER CONDITIONS (avg.) _____	WEATHER CONDITIONS (avg.) _____	WEATHER CONDITIONS (avg.) _____
TEMPERATURE (avg.) _____	TEMPERATURE (avg.) _____	TEMPERATURE (avg.) _____	TEMPERATURE (avg.) _____
BEG MP _____ END MP _____	BEG MP _____ END MP _____	BEG MP _____ END MP _____	BEG MP _____ END MP _____
LANE (L1, R1, ALL) _____	LANE (L1, R1, ALL) _____	LANE (L1, R1, ALL) _____	LANE (L1, R1, ALL) _____

Was material testing performed? Yes ? No ?

Marshall Stability – Design _____ Marshall Stability – Constructed _____ Voids – Design _____	Voids – Constructed _____ Density (lb/ft^2) – Design _____ Density (lb/ft^2) – Constructed _____	% Asphalt – Design _____ % Asphalt – Constructed _____ Flow _____	Additive (Lime or Other) _____ Additive % _____

10.4.3 Three Aspects of Maintenance Effects - Illustration

To adequately define the effects of maintenance for planning cost effective treatments, there are three aspects of the maintenance treatments that must be considered as illustrated in Figure 10.6. The first involves changes to the pavement structure and the keeping of adequate records of maintenance treatments applied. The second is the immediate change in the pavement condition due to maintenance application. The third facet of the problem is the effect of the maintenance treatment on the pavement's deterioration rate.

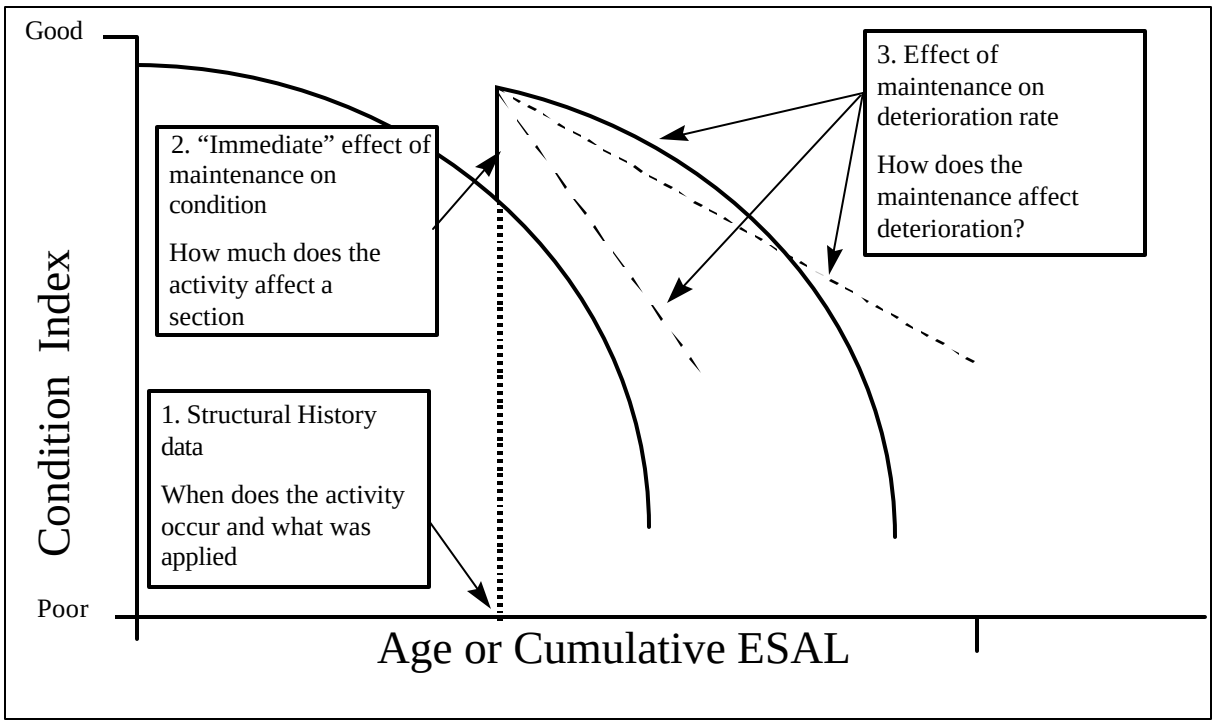


Figure 10.6 Illustration of Maintenance Effects on Pavement Performance

Changes to Structural History

When maintenance treatments are applied it is imperative to record the activity so the effect on pavement condition can be quantified. Unfortunately these data are not always available at the current time. The goal of the Montana PMS section is to eventually have every maintenance treatment recorded within their structural history records. To begin this process the state is reviewing on a yearly basis changes in pavement condition especially when a pavement has improved without a corresponding construction or maintenance record. This situation can occur when activities have been completed and no records were entered in the database or when the condition survey is erroneous. In either case this is a critical step in data quality assurance. When sections are found that are missing construction or maintenance data, the PMS staff queries the maintenance and construction divisions to determine what work was done and other aspects such as cost and timing of the work. This is the initial step in beginning to understand maintenance effectiveness. In this example we show how the interaction of condition data from year to year can be used to identify missing maintenance data and some of the tools available to the Montana PMS unit for accomplishing this important task.

Figure 10.7 shows an illustration of the effects of incorrect or missing structural history on the pavement performance model development process. It is expected that reasonable data should fall within the area labeled "Typical Performance Region." If data points fall outside of this region, they have failed a very general "test of reasonableness" and are therefore candidates for scrutiny as to the reason they are not within the expected region.

Potential reasons to be investigated depend on the location of a point relative to the region as identified by the bullet points on the figure.

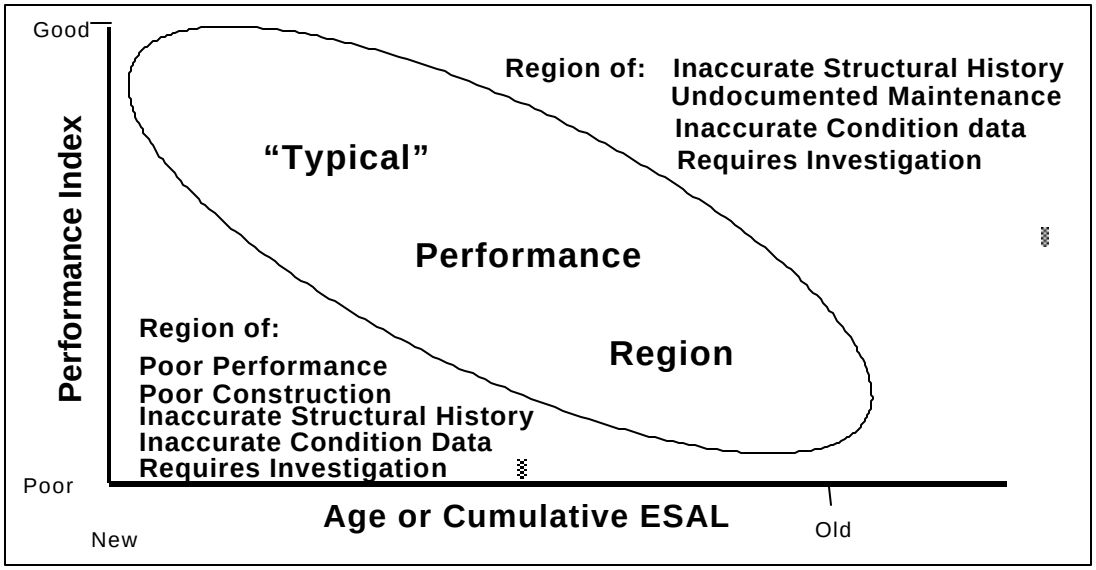


Figure 10.7 Typical Effects of Missing or Erroneous Pavement Structural/Maintenance History Data on Performance Model Development

Condition Changes

Once adequate maintenance data is accumulated in the database it is possible to begin quantifying the effects of maintenance on the condition measures the PMS uses. In the Montana system performance indexes range from distress specific indexes such as a fatigue index to an overall condition index that includes many distresses. In this example we show how different maintenance activities affect each index and how these improvements are used in programming maintenance in the Montana PMS.

10.4.4 Evaluating Maintenance Effects on Condition and Deterioration

A lot of consideration has been given to how best to quantify the effects of maintenance in the Montana pavement management system. Different estimates and models have been made showing the possible effects of maintenance on the service life of a roadway.

The typical approach to quantifying the effects of maintenance is divided into two parts; 1) estimate the condition improvement by evaluating condition immediately before and after maintenance, and 2) to try to quantify the effect of maintenance on the subsequent deterioration rate of the pavement.

Testing differences in pavement life due to maintenance, however, is difficult, by and large because maintenance is demand responsive and is applied based upon need or some underlying policy. Consequently, segregating road sections into two categories, those that receive much maintenance and those that receive little maintenance, and estimating their respective road lives does not indicate the true effect of maintenance. Rather, such an analysis would probably indicate that initially well-built roads (i.e., those that receive less maintenance) last longer than poorly built roads (those which demand much more or sooner maintenance). A simple "maintenance" interpretation of this phenomenon would say that the effect of maintenance is to increase the deterioration of the roadway but this is untrue and merely reflects a biased sampling process.

Figures 10.8 and 10.9 illustrate the extent to which the effect of maintenance is quantified for a pavement section. By intensive data collection, points A and B can be estimated and consequently the immediate effect of maintenance on condition can be quantified. However, this "immediate" effect is not sufficient to define the real beneficial effect. Minimally, you would need to know the pavement condition with and without maintenance at one other point in time (after the maintenance activity) to get a crude linear estimate of maintenance effect (the hatched areas under the curve in Figures 10.8 and 10.9). This requires pairs of similar pavement sections. One pavement from the pair would receive maintenance (the solid line B-D-G and B-D-C) while the other would not and would produce plot AFG or AFE. The effects of maintenance are then represented by the hatched area under the curve. The shape of the maintenance effect will depend on the maintenance treatment applied. In the case of the treatment modeled in Figure 10.8, the treatment temporarily increases condition but as the treatment nears the end of its life, the pavement deterioration returns to the same path as the original pavement structure.

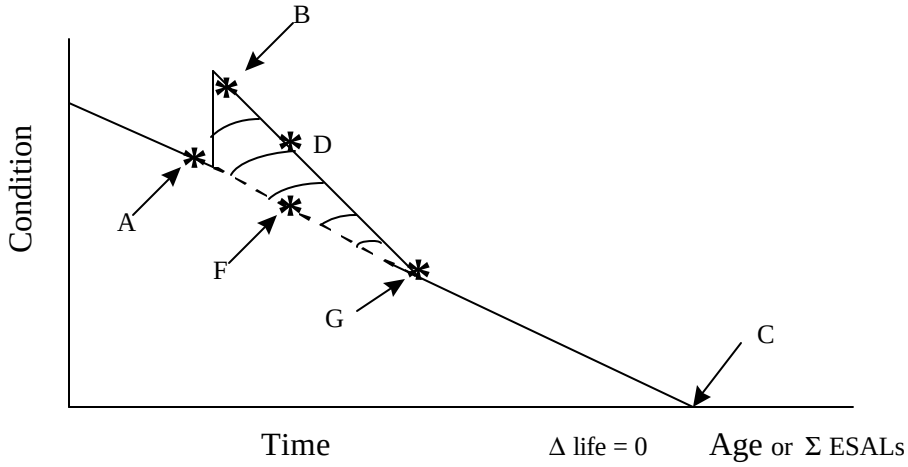


Figure 10.8 Temporary Improvement to Condition due Maintenance

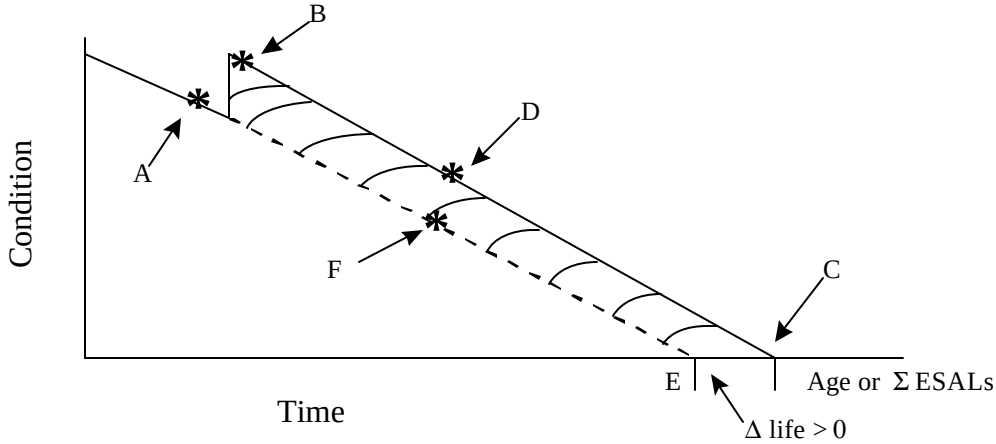
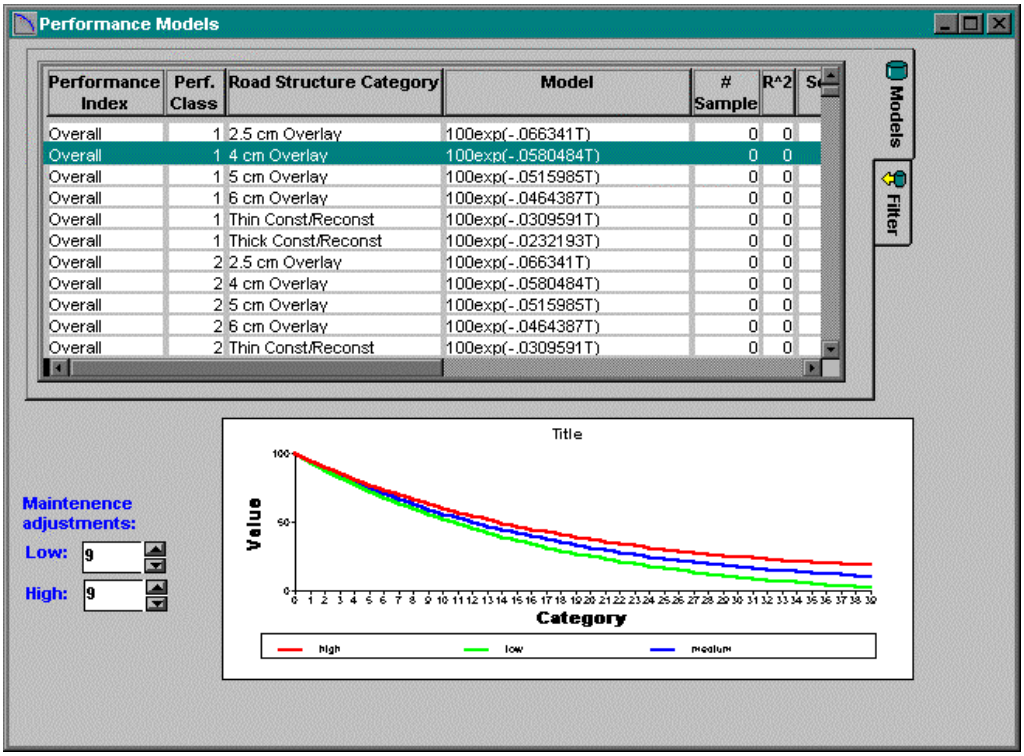


Figure 10.9 Life Extension due Maintenance

This temporary effect (see Figure 10.8) does not prolong road life and so increases the road's life-cycle cost. The benefit of such an effect is to reduce user costs and its implementation is only justified through simulation when benefit can be analyzed in proportion to the increased area under the curve. Such treatments may include crack sealing or fog seals. The current Montana PMS incorporates the effects of these maintenance treatments using this model. [OMB 92]

The effects of a treatment, as simulated in Figure 10.9, both improve condition and prolong roadway life. The benefits of treatments modeled in this way are greater as indicated by a larger increase of area under the curve and the potential to reduce life cycle costs because overall pavement life is being extended. Examples of these types of treatments may include thin overlays and possibly chip seals. Although not currently included, the updated Montana PMS will have the ability to incorporate the effects of these maintenance treatments (or lack of "designed-in maintenance") by using the user controlled performance modeling screen shown in Figure 10.10. This update is in the planning stages by MDT.

In order to estimate the effects of maintenance on road life in a normal highway agency, one would have to look for two sets of similar sections, each maintained by disparate maintenance policies in the same agency. Having two different maintenance policies simultaneously employed by one agency may occur across districts or can possibly be set up as a research experiment.



Performance Index	Perf. Class	Road Structure Category	Model	# Sample	R^2	Se
Overall	1	2.5 cm Overlay	100exp(-.066341T)	0	0	
Overall	1	4 cm Overlay	100exp(-.0580484T)	0	0	
Overall	1	5 cm Overlay	100exp(-.0515985T)	0	0	
Overall	1	6 cm Overlay	100exp(-.0464387T)	0	0	
Overall	1	Thin Const/Reconst	100exp(-.0309591T)	0	0	
Overall	1	Thick Const/Reconst	100exp(-.0232193T)	0	0	
Overall	2	2.5 cm Overlay	100exp(-.066341T)	0	0	
Overall	2	4 cm Overlay	100exp(-.0580484T)	0	0	
Overall	2	5 cm Overlay	100exp(-.0515985T)	0	0	
Overall	2	6 cm Overlay	100exp(-.0464387T)	0	0	
Overall	2	Thin Const/Reconst	100exp(-.0309591T)	0	0	

Figure 10.10 Performance Modeling Screen

One possible way to test the effects of maintenance would be to look at roads constructed to similar design standards but operated and maintained by two different divisions within an agency at different maintenance standards. For example, comparison of low volume state-maintained roads versus typical county-maintained roads has potential. In this case the roads selected would be built under approximately the same standards and operated in the same physical environment. All physical factors being the same, the maintenance policy of the state agency might be at a higher service level than in the county agency. In such a case, the difference in road life could be attributed to the difference in maintenance policy. Because of statistical error in the comparisons, an adequate number of pavement sections must be included in each test set.

10.4.5 Other Uses of Minor Maintenance Historical Data

The level of maintenance required to keep a road operating properly throughout its life may be a good indicator of the quality of or the appropriateness of its original construction and design. Consequently, investigating roadways

that require unusually high maintenance or surprisingly low maintenance, can often times result in verifying and identifying good design or construction practices, and identifying the source of poor design or construction methods with appropriate suggestions for remediation. This is a valuable purpose.

To find these low and high cost maintenance locations, maintenance work must be well quantified with respect to location on the road. Subsequent remediation in those local areas (be it maintenance work or rehabilitation work) could provide a long-term solution to those maintenance problems thus improving the overall serviceability of the entire road. This not only provides better service at small additional cost but also would lower the annual maintenance costs of the road.

There are several other important uses of maintenance data and analysis being considered for the updated Montana PMS. These include:

- Justifying requests for more maintenance budget
- Improving estimates of road life which are currently biased due to a lack of maintenance work accomplished information
- Improving construction and maintenance practices
- Deciding what maintenance work may be deferred when budgets are cut and minimize adverse impacts on the pavement.

The following methodologies are being considered for incorporation in the Montana PMS in order to better collect, analyze, and use maintenance information for the above purposes.

1. Perform defect surveys and estimate costs of current maintenance needs and presenting them to the legislature. Requirements include the staff and facilities to collect and process such a survey and corresponding estimated costs to maintain.

2. Include major maintenance historical work along with construction/rehabilitation history. Consequently, in analysis, what used to be a 30-year road in fair condition is now analyzed as a five-year road in fair condition. The requirement for this analysis is the storage of major maintenance historical work including compatible location referencing.

3. Store estimated annual cost summaries of all maintenance historical work activities on small sections of roadway. These are then plotted down the road to see exceptionally high cost road lengths and exceptionally low cost road lengths. Then perform a detailed on-site investigation of these "exceptional" cases to identify and solve a major structural problem or identify and proliferate an excellent maintenance or rehabilitation technique.

4. Based on a priority mechanism, set and estimate costs for a minimum level of service. With remaining money, estimate the costs for the highest priority until the budget is exhausted. This will provide a logical and easily understandable approach applying lower budgets to reducing maintenance service levels and identifying what services will be reduced. This plan can then be presented to the legislature where they will understand the consequences of budget cuts and then either accept these consequences or increase the budget.

In addition to these methodologies software is being formulated that can simulate expert opinion as to the condition effect consequences of maintenance policy changes.

- For major maintenance activities the software will be able to establish the needed budget to support different application cycles (e.g., surface treatments applied every ten years instead of every eight). This may mean the application of entire life cycles of treatments being activated where one run will simulate life cycles with eight-year surface treatments in between overlays and reconstruction, and another run would use a ten-year surface treatment frequency.

- For minor maintenance activities the software will allow changes in overall condition deterioration rate based upon the average maintenance dollars per mile expected to be applied to the roadway in the future.

The difficulty is in "feedback," the quantitative assessment based upon historical data of the expert opinion provided above. Specifically, at this time there is no statistical data analysis that will definitively answer the following related questions:

- If I spend 20% less on minor maintenance, how much faster will my roads deteriorate?

- If I stop cutting grass except for once a year, how much faster will my roads deteriorate?
- If instead of cutting grass once a month, I put this money into an extra pothole patching crew, how much slower will the roads deteriorate?

If these are the right questions to ask, then much effort will be required before they are answered adequately. Perhaps there are other questions whose answers are more easily attained while fulfilling one or more of the goals mentioned above. An adequate pavement management database, designed with maintenance considerations, will help to improve our understanding and incorporation of maintenance effects.

10.5 CONCLUSIONS

Maintenance activities can have a major effect on:

- The functional condition of the pavement,
- The rate of structural deterioration of the pavement, and
- The pavement life.

A properly executed and maintained Maintenance Management System (MMS) can help in quantifying these effects.

Full benefits will occur when MMS and PMS are integrated where needed.

10.6 REFERENCES TO MODULE 10

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[Chong 87] Chong, G.J. and W.A. Phang, "Improved Preventative Maintenance: Sealing Cracks in Flexible Pavements in Cold Regions," Ontario Ministry of Transportation Rept. PAV-87-01, Dec., 1987

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[Jorgensen 72] Roy Jorgensen Associates, "Performance Budgeting System For Highway Maintenance Management," NCHRP Report 131, 1972.

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[OMB 92] Office of Management and Budget, “Guidelines and Discount Rates for Benefit – Cost Analysis of Federal Programs,” Circular No. A-94, Office of Management and Budget, Executive Office of the President, Washington, D.C., 1992.

[RTAC 77] Roads and Transportation Association of Canada, "Pavement Management Guide," 1977.

GLOSSARY OF TERMS

AADT
The average 24-hour traffic volume counts collected over a number of days greater than 1 but less than a year, at a given location. AADT can also be approximated by adjusting the ADT count for daily (weekday versus weekend) and seasonal (summer versus winter) variations.

AASHTO
American Association of State Highway and Transportation Officials

ADT
The average 24-hour traffic volume counts collected over a number of days greater than 1 but less than a year, at a given location.

ADTT
The average 24-hour truck traffic volume counts collected over a number of days greater than 1 but less than a year, at a given location. ADTT may be expressed as a percentage of ADT.

Algorithm
A prescribed set of well-defined rules or processes for the solution of a problem in a finite number of steps.

Annual Costs
Any costs associated with the annual maintenance and repair of the facility.

Asphalt Emulsion Mix
A mixture of emulsified asphalt materials and mineral aggregate usually prepared in a conventional hot-mix plant or drum mixer at a temperature of not more than 127 °C (260 °F). It is spread and compacted at the job site at a temperature above 93 °C (200 °F).

Benefit-Cost Ratio
The ratio of the dollars of discounted benefits achievable to the given outlay of discounted costs.

Cape Seal
A surface treatment that involves the application of a slurry seal to a newly-constructed surface treatment or chip seal. Cape seals are used to provide a dense, waterproof surface with improved skid resistance.

Capital Costs
Non-recurring or infrequently recurring costs of long-term assets (including depreciation and property taxes).

CESAL
Cumulative Equivalent Standard Axle Loads, e.g. the summation of ESALs over a year (see ESAL).

Chip Seal
A surface treatment in which a pavement surface is sprayed with asphalt (generally emulsified) and then immediately covered with aggregate and rolled. Chip seals are used primarily to seal the surface of a pavement with non load-associated cracks and to improve surface friction, although they also are commonly used as a wearing course on low-volume roads.

Cold In-Place Recycling (CIR)
A process in which a portion of an existing bituminous pavement is pulverized or milled, the reclaimed material is mixed with new binder and virgin materials, and the resultant blend is placed as a base for a subsequent overlay. Emulsified asphalt is especially suited for cold in-place recycling. Although not necessarily required, a softening agent may be used along with the emulsified asphalt.

Cold Milling
A process of removing pavement material from the surface of the pavement either to prepare the surface (by removing rutting and surface irregularities) to receive overlays, to restore pavement cross slopes and profile, or even to re-establish the pavement's surface friction characteristics.

Combined Performance Indexes
Combinations of PI's, examples are Structural Index (STI), Surface Distress Index (SDI), Safety Index (SFI), Overall Pavement Index (OPI), Present Serviceability Index (PSI).

Crack Filling
A maintenance procedure that involves placement of materials into non-working cracks to substantially reduce infiltration of water and to reinforce the adjacent pavement. Working cracks are defined as those that experience significant horizontal movements, generally greater than about 2 mm (0.1 in). Crack filling should be distinguished from crack sealing.

Crack Sealing
A maintenance procedure that involves placement of specialized materials, either above or into working cracks, using unique configurations to reduce the intrusion of incompressibles into the crack and to prevent intrusion of water into the underlying pavement layers. Working cracks are defined as those that experience significant horizontal movements, generally greater than about 2 mm (0.1 in).

CRCP
Continuously Reinforced (Cement) Concrete Pavement

Data Mining
Reviewing existing data, data bases and subsets of data, to determine what information might be available that had previously been overlooked.

Default Value
A design value that is based on experience or on studied conclusions and that is used as a substitute value when an actual value is not available.

Dense-Graded Asphalt Overlay
An overlay course consisting of a mix of asphalt cement and a well-graded (also called dense-graded) aggregate. A well-graded aggregate is uniformly distributed throughout the full range of sieve sizes.

Deterministic Model
A model that expresses the interaction of system elements with complete certainty, that is, as absolute values.

Diamond Grinding
A maintenance procedure for concrete pavements that involves the removal of a thin layer of concrete (generally no more than 6.4 mm [0.25 in]) from the surface of the pavement to remove surface irregularities (most commonly joint faulting), to restore a smooth riding surface, and to increase pavement surface friction.

Diamond Grooving
The establishment of discrete grooves in the concrete pavement surface using diamond saw blades to provide a drainage channel for water and thereby reduce the potential for hydroplaning and wet weather accidents.

Discount Rate
In cost-benefit analysis, an interest rate used to reduce the value of benefits or costs accruing in future years back to their current worth (present value). If the discount rate is 4 percent, $1.04 a year from now is of equal value as $1.00 today.

Empirical Model – Prediction based on experience only.

Emulsified Asphalt
An emulsion of asphalt cement and water, which contains a small amount of an emulsifying agent. Emulsified asphalt droplets, which are suspended in water, may be either the anionic (negative charge) or cationic (positive charge) type, depending upon the emulsifying agent.

ESAL
Equivalent Standard Axle Load, used to transform loads by different vehicles into standard units.

Equivalent Uniform Annual Cost (EUAC)
The net present value of all discounted cost and benefits of an alternative as if they were to occur uniformly throughout the analysis period. Net Present Value (NPV) is the discounted monetary value of expected benefits (i.e., benefits minus costs).

FHWA
Federal Highway Authority

Fog Seal
A light application of slow setting asphalt emulsion diluted with water. It is used to renew old asphalt surfaces and to seal small cracks and surface voids.

FWD
Falling Weight Deflectometer, used to assess the structural pavement properties.

GIS
Geographic Information System

Heater Scarification
A form of Hot In-Place Recycling in which the surface of the old pavement is heated, scarified with a set of scarifying teeth, mixed with a recycling agent, and then leveled and compacted.

Hot In-Place Recycling (HIR)
A process which consists of softening the existing asphalt surface with heat, mechanically removing the surface material, mixing the material with a recycling agent, adding (if required) virgin asphalt or aggregate to the material, and then replacing the material back on the pavement.

Hot Mix Asphalt (HMA)
High quality, thoroughly controlled hot mixture of asphalt cement and well-graded, high-quality aggregate thoroughly compacted into a uniform dense mass.

Hot Surface Recycling
See hot in-place recycling.

Inflation rate
The rate of increase in the general price levels, caused usually by an increase in the volume of money and credit relative to available goods. The inflation rate is also reflective of the rate of decline in the general purchasing power of a currency.

Initial Costs
All costs associated with the initial design and construction of a facility, placement of a treatment, or any other activity with a cost component.

International Roughness Index (IRI)
A ratio of the accumulated suspension motion to the distance traveled obtained from a mathematical model of a standard quarter car transversing a measured profile at a speed of 80 km/h (50 mph). Expressed in units of meters per kilometer (inches per mile), the IRI summarizes the longitudinal surface profile in the wheelpath.

JCP
Jointed (Cement) Concrete Pavement.

Joint Resealing
The resealing of transverse joints in concrete pavements to minimize the infiltration of surface water into the underlying pavement structure and to prevent the intrusion of incompressibles into the joint.

Joint Sealant Reservoir
The channel sawed or formed at a joint that accommodates the joint sealant.

Level of Service
A qualitative rating of the effectiveness of a highway or highway facility in serving traffic (users), in terms of operating conditions (volume, speed, comfort, safety).

Load Transfer Restoration (LTR)
The placement of load transfer devices across joints or cracks in an existing jointed PCC pavement. LTR is used on existing jointed PCC pavements that were constructed without dowel bars at transverse joints.

Life Cycle Cost Analysis (LCCA)
An economic assessment of an item, system, or facility and competing design alternatives considering all significant costs of ownership over the economic life, expressed in terms of equivalent dollars.

Maintenance Management System (MMS)
Rational procedures that provide optimum maintenance strategies for pavements and related elements like ROW, culverts, guardrails, signs, traffic lights, etc. These procedures are based on predicted maintenance effectiveness and cost for a desired Level Of Service, and the optimal employment of labor, equipment and materials, whilst incorporating feedback regarding the various attributes, criteria and constraints involved.

Mechanistic Model
Prediction based on known mechanistic properties of uniform materials, such as Youngs Modulus, strain and stress at break.

Microsurfacing
Microsurfacing is a mixture of polymer modified asphalt emulsion, mineral aggregate, mineral filler, water, and other additives, properly proportioned, mixed and spread on a paved surface.

Mineral Filler
A finely divided mineral product, at least 70 percent of which will pass a 0.075 mm (No. 200) sieve. Pulverized limestone is the most commonly manufactured filler, although other stone dust, hydrated lime, portland cement, and certain natural deposits of finely divided mineral matter are also used.

Model
A mathematical description of a real-life situation that uses data on past and present conditions to make a projection about the future.

Nominal Dollars
Dollars of purchasing power in which actual prices are stated, including inflation or deflation. Hence, nominal dollars are dollars whose purchasing power fluctuates over time.

NOVACHIP™
A maintenance treatment for AC pavements, sometimes called an ultrathin friction course: it consists of a layer of hot-mix material placed over a heavy, polymer modified emulsified asphalt tack coat; the total thickness of the application being typically between 10 and 20 mm (0.40 and 0.80 in). It can be used to reduce deterioration caused by weathering, raveling, and oxidation, and can be used to fill ruts and to smooth corrugations and other surface irregularities.

Open-Graded Friction Course (OGFC)
An overlay course consisting of a mix of asphalt cement and open-graded (also called uniformly-graded) aggregate. An open-graded aggregate consists of particles of predominantly a single size.

Overall Pavement Index (OPI)
A combination of various weighted pavement performance indices (e.g. ride, rutting, cracking, etc).

Partial-Depth Recycling
See cold in-place recycling.

Pavement Distress Index (PDI)
A combination of several distress ratings.

Pavement Evaluation
A technique to measure a range of pavement characteristics such as roughness, rutting, friction and various distresses such as cracking.

Pavement Management
A coordinated systematic process for carrying out all activities related to providing pavements.

Pavement Management Software
A set of tools to assist decision makers in preserving a pavement network.

Pavement Management Section
A location-defined part of the pavement network with a homogeneous cross section, uniform construction history, traffic loading characteristics and history, and uniform drainage and climatic conditions.

Pavement Management System (PMS)
A systematic process that collects and analyzes pavement information with rational procedures that provide optimum pavement strategies based on predicted pavement attributes incorporating feedback regarding the various attributes, criteria and constraints involved. PMS is also called PMIS (Pavement Management Information System) or RMS (Roadway Management System).

Pavement Performance Group
Grouping of pavement sections for performance models based on Traffic Levels (e.g. AADT), Functional Class (e.g. Interstates, State Highways, etc) and Road Structure Category (RSC)

Pavement Performance Model
An empirical or mathematical representation of predicted pavement performance and behavior.

Pavement Preservation
The sum of all activities undertaken to provide and maintain serviceable roadways; this includes corrective maintenance and preventive maintenance, as well as minor rehabilitation projects.

Pavement Preventive Maintenance
Planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retard future deterioration, and maintains or improves the functional condition of the system (without increasing the structural capacity).

Pavement Reconstruction
Construction of the equivalent of a new pavement structure which usually involves complete removal and replacement of the existing pavement structure including new and/or recycled materials.

Pavement Rehabilitation
Work undertaken to extend the service life of an existing pavement. This includes the restoration, placing an overlay, and/or other work required to return an existing roadway to a condition of structural and functional adequacy.

Pavement Serviceability Index (PSI)
A subjective rating of the pavement condition made by a group of individuals riding over the pavement.

Performance Index (PI)
A combination of weighted performance ratings taken from ride, rutting and other pavement evaluation data.

Periodic Costs
Costs associated with rehabilitation activities that must be applied periodically over the life of the facility.

Poisson's Ratio
The ratio of the reduced (increased) cross-section under tensile (compressive) stress, to the original cross-section. For an elastic material the ratio is 0.5.

Present Worth Method
Economic method that requires conversion of costs and benefits by discounting all present and future costs to a single point in time, usually at or around the time of the first expenditure.

Probabilistic Model
Quantification of a future condition using probabilities in an algorithm, where any one of several outputs, each of known probability, can occur for each alternative.

Quality Assurance (QA)
The systematic use of performance requirements, design criteria, specifications, production control procedures, and acceptance plans for materials, processes, or products to ensure prescribed properties or characteristics.

Quality Control (QC)
The system of collection, analysis, and interpretation of measurements and other data concerning prescribed characteristics of a material, process, or product, for determining the degree of conformance with specified requirements.

Real Dollars
Dollars of uniform purchasing power exclusive of general inflation or deflation. Real dollars have a constant purchasing power over time.

Recycling Agents
Organic materials with chemical and physical characteristics selected to address any binder deficiencies and to restore aged asphalt material to desired specifications.

Rejuvenating Agent
Similar to recycling agents in material composition, these products are added to existing aged or oxidized AC pavements in order to restore flexibility and retard cracking.

Retrofitted Load Transfer
See Load Transfer Restoration.

Road Structure Category (RSC)
Categorization of pavement structures based on surface material and thickness, underlying pavement structure, and rehabilitation type.

Rubberized Asphalt Chip Seal
A variation on conventional chip seals in which the asphalt binder is replaced with a blend of ground tire rubber (or latex rubber) and asphalt cement to enhance the elasticity and adhesion characteristics of the binder. Commonly used in conjunction with an overlay to retard reflection cracking.

Rubblization
Technique where existing concrete pavement is hammered into small pieces or chunks of loose material that will act as a granular base.

Salvage Value
The remaining worth of the pavement at the end of the analysis period. There are generally two components of salvage value: residual value – the net value from recycling the pavement and serviceable life – the remaining life of the pavement at the end of the analysis period.

Sand Seal
An application of asphalt material covered with fine aggregate. It may be used to improve the skid resistance of slippery pavements and to seal against air and water intrusion.

Seal Coat
See Chip Seal

Sandwich Seal
A surface treatment that consists of application of a large aggregate, followed by a spray of asphalt emulsion that is in turn covered with an application of smaller aggregate. Sandwich seals are used to seal the surface and improve skid resistance.

Scrub Seal
Application of a polymer modified asphalt to the pavement surface followed by the broom-scrubbing of the asphalt into cracks and voids, then the application of an even coat of sand or small aggregate, and finally a second brooming of the aggregate and asphalt mixture. This seal is then rolled with a pneumatic tire roller.

Shape Factor
The width to depth ratio of a joint sealant reservoir. A proper shape factor is required to allow the sealant to effectively withstand repeated extension and compression as the temperature and moisture in the slab changes. Most commonly available sealants require a shape factor between 1 and 2.

Slurry Seal
A mixture of slow-setting emulsified asphalt, well-graded fine aggregate, mineral filler, and water. It is used to fill cracks and seal areas of old pavements, to restore a uniform surface texture, to seal the surface to prevent moisture and air intrusion into the pavement, and to provide skid resistance.

Stiffness Modulus
Comparable to Youngs Modulus, but used for non-elastic materials. The Stiffness is defined as the ratio of applied stress to the resulting strain for a certain temperature and time of loading.

Stockpiled Cold Mix
An asphalt maintenance mix consisting of aggregate and emulsified asphalt, which once prepared can be stored and readily used for a period up to six months depending on the formulation of the emulsion used and the aggregate characteristics.

Stone Mastic Asphalt Overlay
An overlay course consisting of a mix of asphalt cement, stabilizer material, mineral filler, and gap-graded aggregate. A gap-graded aggregate is similar to an open-graded material but is not quite as open.

Surface Texture
The characteristics of the concrete pavement surface that contribute to both surface fiction and noise.

Undersealing
Also called sub-sealing, pressure grouting, or slab stabilization: this process consists of the pressure insertion of a flow able material beneath a PCC slab used to fill cavities beneath PCC slabs and occasionally to correct the vertical alignment by raising individual slabs.

User Costs
Costs incurred by highway users traveling on the facility and the excess costs incurred by those who cannot use the facility because of either agency or self-imposed detour requirements. User costs typically are comprised of vehicle operating costs (VOC), accident costs, and user delay.

Young's Modulus or Elastic Modulus
The ratio of the stress (force per unit of surface area) to the resulting strain (relative increase in length) in a tensile test.