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Foreword

Novak and Kuo discuss the role of a pavement management system (PMS) in the Michigan Department of Transportation—which is to enable policy makers to control long-term network condition and funding requirements, to reduce the total cost of pavement preservation, and to have decisions flow from the top down. Sachs and Smith describe how the Metropolitan Transportation Commission supports the development and use of a PMS by cities and counties in the San Francisco Bay Area. Smith and Fallaha present information on a project-level PMS that was developed to interface with the existing network-level PMS. Johnson and Cation present the development aspects of three pavement performance indicators (overall distress index, structural index, roughness index) used in the North Dakota PMS. Queiroz et al. summarize the PMS implemented in Brazil, discuss the special limitations and standardization requirements for proper use of pavement management in developing countries, and present recommendations for PMS in developing countries.

Saraf and Majidzadeh describe the development of distress prediction models for a network-level PMS. Sharaf and Abdul-Hai describe the development of an expert system for the management of pavement maintenance in developing countries. Geoffroy and Shufon present New York State's network-level PMS, which is goal-driven and is designed to operate in a decentralized decision-making environment. In a second paper, Novak and Kuo illustrate how the use of project life-cycle cost analysis in pavement management can increase the total cost of network preservation. They propose that policy-makers use network life-cycle cost analysis to minimize the total cost of network preservation.

Grivas et al. present a methodology for determining a pavement distress index; the index formulation is based on individual distress ratings along nominal lengths of pavement and a set of weighting values associated with distress types and severity-extent combinations. Mouaket et al. present a life-cycle costing analysis of seal coating, taking into account agency and user costs. A seal coating decision tree for various types of pavement surface distress was also developed. Zoltan et al. present a rational method for selecting maintenance treatment solutions on the basis of pavement performance, structural capacity, and roughness. Paterson and Attoh-Okine developed two generalized roughness progression models for flexible pavements. They are summary models intended for use in pavement management applications and as a performance model for design. Humplick identifies the types of error that affect the results of infrastructure surface inspection and presents hypotheses derived from theoretical expectations of the effects of various factors on the accuracy of inspection systems.

Gibby and Kitamura identify several factors that affect the condition of pavements owned by local governments. Lu et al. present an adaptive filter forecasting system that forecasts pavement roughness conditions by means of an adaptive filter using roughness history. In the next paper, Lu et al. present the methodology used in developing a rut index based on data collected by the ARAN unit. The methodology and index can be applied to any rut-depth instrument that collects and presents rut data in a similar fashion. Chen et al. describe a mixed integer programming model to minimize the total cost of pavement structures while meeting the constraints of the AASHTO flexible pavement design equations and user-defined criteria. Maser and Scullion present the results from four radar tests on asphalt thickness and conclude that ground-penetrating radar can accurately measure pavement layer properties.

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Contents

Role of Pavement Management System Analysis in Preservation Program Development

EDWIN C. NOVAK, JR., AND WEN-HOU KUO

The traditional role of pavement management system (PMS) analysis is as an integral part of the preservation program and project development process. The Michigan Department of Transportation already has a well-developed process that does not include a PMS analysis method. To avoid the disruptive effects of injecting or mixing one into this complex but well-defined process, a new role for PMS analysis was created: an application software system that analyzes and processes data from the PMS data base for use by policy makers who are then able to do such things as control long-term network condition and funding requirements [via maintenance, rehabilitation, and reconstruction (MR&R) program development constraints] to reduce the total cost of pavement preservation, and to have decisions flow from the top down. The complete preservation program development process is divided into generic processes: data storage or data base, pre-MR&R program, MR&R program, and post-MR&R program. The pre-MR&R program is conducted at the policy-making level. Policy makers currently must make decisions on the basis of incomplete data of poor technical quality. In addition, no analysis tools are available to enable them to accomplish objectives such as reducing the total cost of pavement preservation, using technology to improve funding efficiency, reducing the cost of overhead, and managing pavements actively. It is proposed that PMS analysis correct these problems by providing policy-level management with complete, high-quality, processed data and analysis tools essential for making rational decisions.

The primary concern of highway officials is the cost-effective preservation of highway networks. Acting on this concern, the *Federal-Aid Highway Program Manual* (*1*) was revised in 1989 to set forth policy to select, design, and manage federal-aid highway pavements in a cost-effective manner and to identify pavement work eligible for federal-aid funding. The new policy requires that each state highway agency (SHA) have a PMS that is acceptable to FHWA and based on concepts described in AASHTO publications including *AASHTO Guidelines for Pavement Management Systems* (*2*). The AASHTO guidelines, in a schematic representation of the various modules of a PMS, indicate that the PMS consists of three major modules: the data base, the analysis method, and the feedback process (Figure 1). This implies that the PMS analysis method is an integral part of the agency's maintenance, rehabilitation, and reconstruction (MR&R) program development process.

It is proposed that the pavement preservation process consist of four independent processes: data storage or data base, pre-MR&R program development, MR&R program development, and post-MR&R program development, as shown in Figure 2. The PMS analysis is proposed to be an automated application software system that links the pre-MR&R process directly to the data base, so utility software is necessary to process PMS analysis data into the forms of information that users need. Such a utility software system can be thought of as an intraagency communication system that serves each of the four preservation processes. A schematic representation of a PMS designed as proposed is shown in Figure 3.

All agencies have always used the processes shown in Figure 2. However, studies of these processes indicated pre-MR&R program development (policy level) is the least well informed agency activity. More and better data for decision making exist at lower levels, but they are neither in proper form nor accessible to the policy-making level. As a result, the policy level has operated on incomplete information of poor technical quality. Policies developed on such information are too general to be practicable, except for funding allocation. Furthermore, the policy level has no means of controlling future funding requirements or network condition, no means of using the department's technical capability to improve the efficiency of available funds, and no means of judging the worth of proposed MR&R programs. In this environment, reactive management is necessary, whereas a management system that controls long-term network condition and funding requirements is more effective and desirable.

Another area of concern was the general ineffectiveness of technology—specifically pavement research—to bring on-line cost-saving methodology. For this reason, it appears that a direct communication link between applied pavement research and policy makers is essential if using technology is to be a way of improving funding efficiency. The primary problem appears to be the lack of a way for policy and technical activities to communicate. For such communication to be possible, all levels must describe projects, programs, and networks using the same terms—terms that are mutually understood and meaningful.

Policy makers are accustomed to using subjective terminology and making decisions about subjective issues. Technical activities deal primarily with objective terms that have specific definitions that apply to analytical problems. A mutually essential set of objective terms with specific definitions common to all agency activities was found to be a must for pavement management. Another essential is the ability to relate the performance of MR&R projects to MR&R programs, the performance of MR&R programs to networks, and vice versa. Because technical activities deal primarily with

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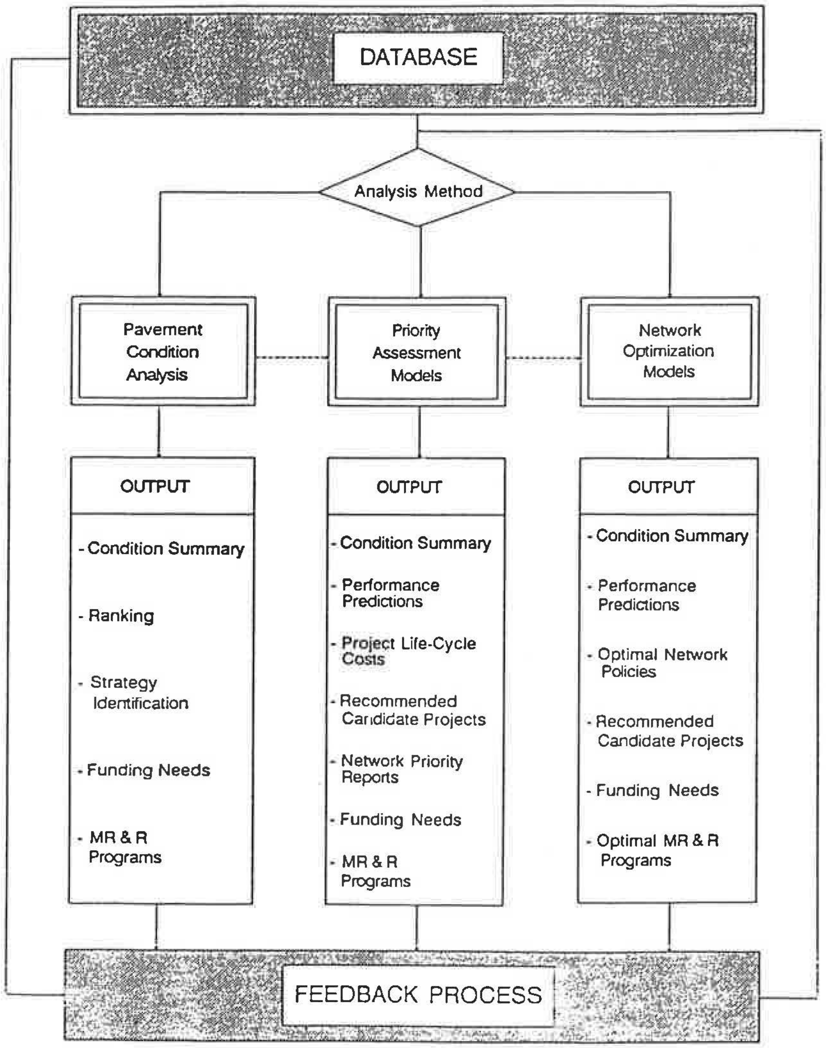

FIGURE 1 AASHTO's representation of a PMS.

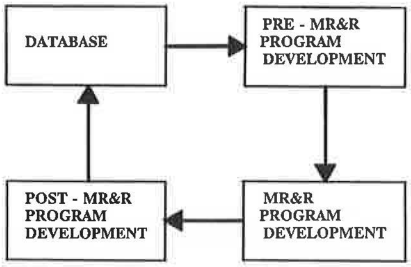

FIGURE 2 Components of complete pavement preservation process.

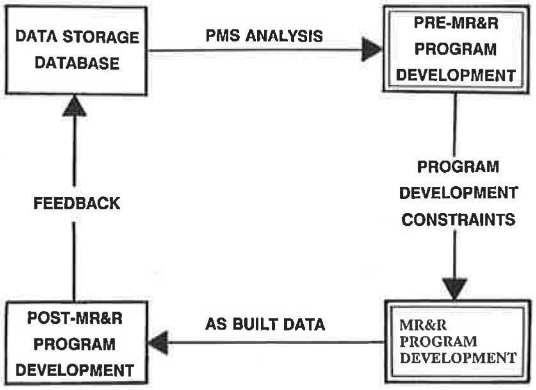

FIGURE 3 Pavement preservation components linked by PMS application and utility software.

projects, the policy level deals primarily with networks, and both deal with programs, all levels must use the same terms to describe them. These terms then become the interface between the project and network levels of analysis.

BACKGROUND

The Michigan Department of Transportation (DOT) formed a pavement management committee in 1980 and assigned it the following tasks:

1. Review the department's current procedures.
2. Determine means for better integrating functions.
3. Identify areas of needed improvement.
4. Make recommendations to upper management.

The committee's preliminary recommendation was that "the department should start development of a simplified PMS that eventually will address many of the needs identified." This recommendation was accepted, and the same committee was asked to develop proposals for improving the existing management system. During this time, a small research staff was directed to investigate technical aspects of pavement condition survey procedures and PMS analysis methodology. One problem for the committee was that department managers and committee membership changed often, producing a fluctuating environment of diverse opinions from which consensus could be reached on what should not be done but not on what should be done.

The research group, in the meantime, determined that a purely analytical PMS analysis method is feasible if it is based on a pool of data representing the lane-mile length, remaining service life (RSL), design service life (DSL), cost, and all benefits of each of all feasible MR&R treatments, of all uniform sections that make up all networks. The RSL comes from PMS pavement conditions surveys (*3*), the DSL from PMS project design analysis, and the cost from PMS cost analysis. Accurate project cost estimates are possible if the available data include a detailed research-level inventory of pavement condition for 100 percent of all networks and a complete physical inventory.

After several years of meetings, the PMS committee could not develop or find a suitable documented PMS to adopt. The analytical method developed as a research study was considered to be a baseline PMS method that the committee proposed to develop further. The research team that developed the method was converted to a full-time PMS development group. The PMS committee never made recommendations for further development and implementation, and its responsibility was changed to its current status of a PMS users group.

From 1980 to 1986, interviews with a wide cross section of key staff, opinions expressed by a majority of committee members, and results of studies of current practice all indicated that Michigan's current pavement management practices are well accepted and should not be modified by a formal PMS. Nevertheless, key staff repeatedly indicate the need for PMS to provide

1. Easier access to historical pavement information.
2. Better pavement performance data.
3. Analysis methods to provide information for decision making.
4. Simplified methods of developing and presenting pavement policy, funding allocations, and network priorities.

PAVEMENT PRESERVATION PROCESS

The basic activities in the pavement preservation processes have been conducted and institutionalized by all SHAs, and it is reported that the information needed to establish pavement preservation policy, allocate funds, and set network priorities should be the same for all SHAs (*4*). But it is understood that the activities and methodology used, although frequently similar, are specific to an agency. The reason that all SHAs do the same thing differently is attributed to operational and organizational differences from one SHA to the other, to the gap between revenues and needs, and to the subjective nature of pavement management associated with the quality and the completeness of information used to make policy decisions.

From the beginning all SHAs received revenues, planned where and how these funds would be allocated, designed projects, let contracts, supervised and monitored construction, and stored records of what was done. Improvements followed each year, and independent activities such as design, materials, testing, research, traffic and safety, and planning became separate activities with specific program and project development duties. All agencies are composed of the same basic activities, but each agency has different organizational and operational characteristics. Nevertheless, pavement preservation for all agencies consists of the four processes shown in Figure 2. An explanation of the activities basic to each process and its products is presented in the following sections.

Pre-MR&R Program Development

Developing the pre-MR&R program entails

1. Allocating funds to programs such as capacity improvement, network expansion, safety, bridge, and maintenance, as well as pavement preservation;
2. Establishing MR&R program development policies and constraints; and
3. Setting priorities for benefits to be provided by the MR&R program.

MR&R Program Development

Developing an MR&R program consists of project identification, programming, scheduling, design, cost estimating, traffic and safety, letting, and construction. When allocated funds are adequate to enable funding for all or most proposed MR&R projects, few program or project development problems occur. It is when the total cost of proposed projects exceeds available funds that problems mount. This process has been well established and is the most complex, organizationally and operationally.

Post-MR&R Program Development

All SHAs conduct some form of postprocessing of annually collected MR&R or as-built data. However, the process usually does not include improving methodology, identifying means to improve funding efficiency, or processing as-built data and information into forms most useful for developing new policies and making decisions. Hence, the post-MR&R process has not been useful to the policy level. The result has been that the policy level has little opportunity to learn from past programs and does not have the means to develop cost-saving policies. Post-MR&R also includes condition survey and pavement evaluation activities.

Data Base

The data base is the repository for all the agency's historical data and information about each annual preservation program. All agencies have had a data storage system that can be transformed to a data base: racks full of plan drawings, file folders full of test reports, boxes full of file folders, and warehouses full of boxes. Only recently have data and information storage been computerized.

PROPOSED ROLE OF PMS IN PAVEMENT PRESERVATION PROCESS

The new FHWA pavement policy (*1*) presents the type of information a PMS should deal with. The AASHTO PMS guide (*2*) describes the characteristics and parts of a PMS and its products. Both of these references are written as though every agency will have a different PMS analysis method. If all agencies have different systems for doing the same thing (maximize every available highway dollar), it follows that these systems are subjective. Then, should it be reasonable to declare that, based on our subjective system, we are maximizing the effectiveness of every available dollar? The point is that the PMS analysis method should not be a subjective system that attempts to take into account all the concerns and nitty gritty details that are necessary to MR&R program development. Instead, the PMS analysis method should be an accurate analysis method designed to provide reliable long-term outcomes of any feasible funding scheme. With complete and accurate data, the methodology is simple, direct, and essential for rational decision making. To meet these requirements, the PMS analysis method must provide

1. The answer to any feasible objective question about network preservation.
2. The ability to establish MR&R program development constraints that will
    - Control long-term network condition and funding requirements,
    - Guide the technical staff through MR&R program development, and
    - Minimize the total cost of network preservation.
3. The means to improve funding efficiency by way of improved technical practices.
4. A measure of the effectiveness of proposed MR&R programs and technical MR&R program development activities.
5. A quantitative measure of the benefits of alternative programs.

A PMS analysis method is analogous to an accounting method; that is, neither method makes decisions, and both methods' output data are only as reliable and accurate as the input data are complete and correct. The purpose of software systems that are the PMS, and that link the four preservation program processes as shown in Figure 3, follows.

PMS Analysis Application Software System

The purpose of PMS analysis is to provide the policy level with all the high-quality data and information needed to determine the pavement preservation MR&R strategy that will, at the lowest total long-term cost of preservation, result in the desired long-term network condition and funding requirements. This means that the traditional network-level analysis must be based on the following application software requirements:

1. The performance of projects, networks, and MR&R programs must all be characterized by the same parameters.
2. The data base contains cost-estimating data, an inventory of the highway infrastructure, basic data necessary to compute the benefits derived from any feasible MR&R treatment for any uniform section, and pavement condition data.
3. The pavement condition surveys provide the basic data needed to estimate cost of MR&R treatments and reactive maintenance, identify cause of deterioration, estimate rate of deterioration, and estimate the DSL of alternative MR&R treatments for 100 percent of each network.
4. The software system must have the ability to identify boundaries of contiguous segments of pavement having uniform condition and RSL.
5. Application software to automate project-level analysis of all uniform sections that make up any designated network.
6. Strategy analysis software for developing MR&R program development constraints and to conduct network life-cycle costing.

The basic methodology for these software requirements is explained by Kuo et al. (*4,5*). An interface among performance of projects, networks, and MR&R programs is created when their performance is characterized by RSL and lane-mile length. The DSL of a project is a constant that becomes its RSL at the time of construction. The means used to estimate RSL are illustrated in Figure 4. The performance of projects, MR&R programs, and networks is characterized in terms of RSL and lane-mile length. This automatically provides an interface between project- and network-level analysis and enables policy and technical levels to communicate with the same terms and the same definitions for the same things.

A detailed inventory of the pavement infrastructure and pavement distress is needed so that the PMS analysis software system can provide policy makers with

- Accurate cost estimates of MR&R, preventive maintenance, and repair treatments and reactive (routine) main-

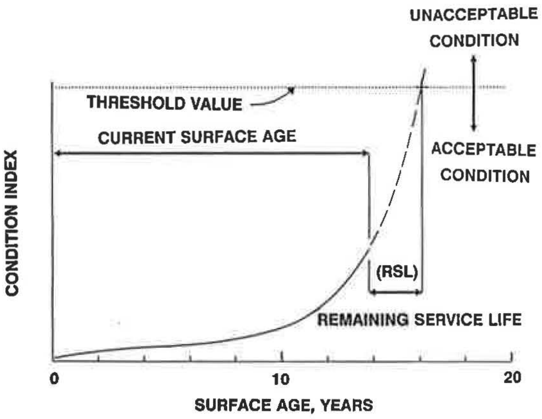

FIGURE 4 Pavement performance curve illustrating RSL concept.

tenance cost for all uniform sections that make up each network.

- Accurate assessments of the performance of each uniform section that, when combined, provide an accurate assessment of network performance.
- Accurate assessment of cost effectiveness and benefits derived from each of all feasible MR&R treatments.
- An accurate measure of funding efficiency and quantified benefits of proposed MR&R strategies and programs.
- Accurate estimates of cause and rate of deterioration.

For the PMS analysis software to provide this, it must consist of application software systems for analyzing pavement condition data, projects, networks, and strategies. The left half of Figure 5 illustrates the activities in and the flow of processed data through the PMS analysis software system starting from the data base and ending at the policy-planning level.

This PMS analysis method gives policy makers a way to conduct economic analysis to minimize the total cost of pavement preservation (network life-cycle cost). The methodology for network life-cycle cost analysis is simple (*5*). And a comparison of network and project life-cycle cost methods in another paper by Novak and Kuo in this Record illustrates the many advantages of network life-cycle cost analysis. A manual form of this PMS analysis method (*6*) illustrates how it provides the PMS products listed in the AASHTO guidelines for PMS (*2*,p.3). The proposed role for the PMS analysis enables policy makers to evaluate alternative funding schemes. Such a study was conducted for FHWA of three Michigan DOT highway districts (*7*). The results illustrate that when PMS analysis is designed to serve the pre-MR&R program process, there is much freedom for creatively allocating funds to other programs, for reducing the total cost of pavement preservation, and for reducing administrative overhead cost.

MR&R Program Development Constraints

The right half of Figure 5 indicates the MR&R program development process. The promulgated constraints (MR&R strategy, funding level, and benefit priorities) are the starting point for MR&R program development. Subordinate staff select projects and match their lengths and DSLs to those of the MR&R strategy. This approach may bother those who think in terms of doing what is best for the project. However, matching project length and DSL to an MR&R strategy pro-

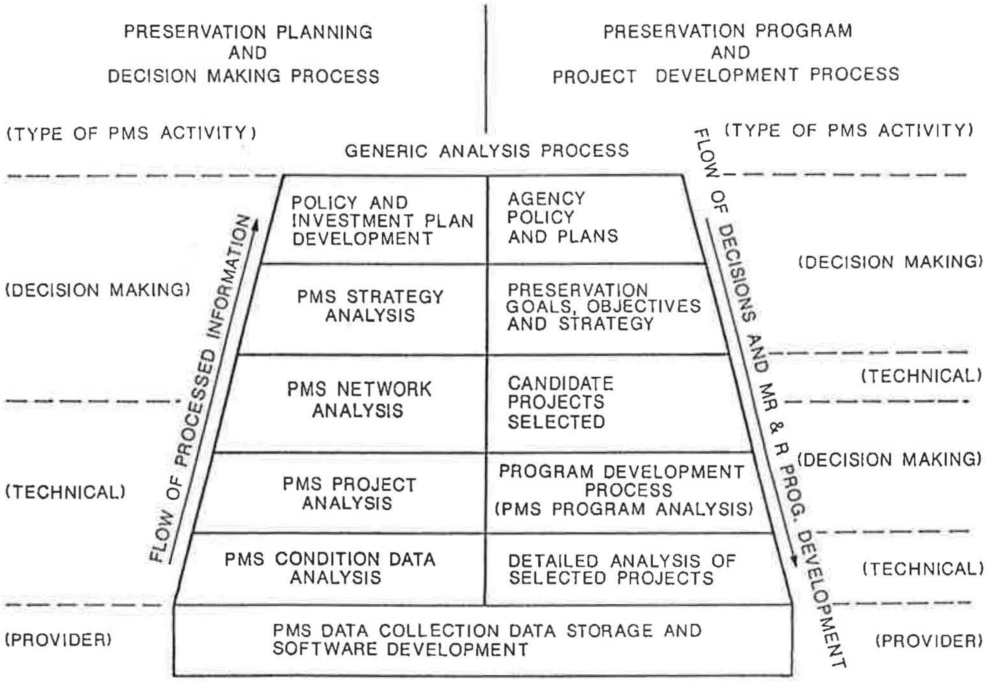

FIGURE 5 Relationship between PMS analysis method and pre-MR&R and actual MR&R program development processes. Also shown are flow of processed information and decisions and type of activities involved.

vides greater freedom to select projects, to select MR&R treatments, and to use engineering analysis to reduce project cost compared with systems that use project life-cycle cost methods to select treatments. As Figure 5 illustrates, a program analysis software system is available to assist with, and should be a necessary part of, finding alternative combinations of projects and treatments that maximize MR&R program benefits. Alternative programs are listed in order of benefit/cost ratio. The policy level knows, for its PMS analysis method, the minimum MR&R program cost and the maximum benefits that are theoretically possible for each network. A comparison of the theoretically best possible program and the proposed MR&R program provides a yardstick measure (efficiency) of its acceptability.

As-Built Data

The as-built data that flow from the MR&R program development activities are the final MR&R strategy, DSL, cost, location, lane-mile length, materials, layer thickness, and physical inventory types of data that are needed for analysis and processing before transfer to the data base. As-built data consist only of data and information that PMS users have asked to access via the PMS and data that are required as input data for the PMS analysis application software.

Feedback

Feedback consists of the data, information, and software improvements that are the products of post-MR&R program development.

DISADVANTAGES OF PROPOSED ROLE FOR PMS ANALYSIS

A PMS whose role is that of a data processing and communication link provides agencies with advantages, but it has its problems as well. The first problem is that of getting MR&R program development activities to give their data products to the PMS. Pavement design, cost estimation, and project programming and scheduling are examples of activities whose data products should be communicated to the post-MR&R program development process via PMS utility software. This means that whereas the PMS is not a part of the operational procedures for MR&R program development, all activities involved in program development must provide their key products to the post-MR&R process. Getting organizational units (such as research) to be a part of the feedback process and to supply key data to the data base are serious problems. However, it makes sense that the primary purpose for storing as-built data and for applied pavement research should be to reduce the future lane-mile cost of pavements per year of DSL. The PMS should be designed to have all the research-level condition and physical inventory data and the analysis software systems needed for applied research to serve the policy makers directly. Likewise, the accuracy of PMS cost estimates should be the responsibility of the activity that makes the agency's cost estimates. Other problems include formalizing pre- and post-MR&R program development and making adjustments needed to change to an active (as opposed to reactive) management style.

SUMMARY

In the absence of a PMS with a role and capabilities as outlined in this paper, the policy level does not have information that is complete enough or of sufficient quality to enable making good rational decisions, to implement cost-saving measures, to control the effectiveness of MR&R programs, to quantify the benefits of alternative MR&R programs, to minimize the cost of pavement preservation, to use technology to effect cost savings, to evaluate technical staff performance, to reduce the cost of overhead, or to move from a reactive to an active management style.

To correct such a situation, the proposed role for the PMS analysis method is to be a data processing and analysis link between the PMS data base and the policy makers. This role requires that the data base contain all the raw data and information of the highest possible quality so that the PMS analysis method can be programmed to answer any conceivable question about any conceivable funding scheme.

The proposed role requires use of RSL (to keep track of the rate of deterioration) and lane miles of pavement as measures of the quantity of pavement in each RSL category. These two terms enable all levels to communicate with each other and provide a simple interface between project and network analysis. For this to work, it is necessary that at least materials, pavement research, cost estimating, and pavement design activities be responsible for their respective areas of the post-MR&R program development (feedback) process. These activities are then in a position to serve the policy level directly by providing the most complete and accurate data the agency's technical staff can produce and to ensure that the policy level has complete, accurate, and reliable data and state-of-the-art tools necessary to maximize benefits from every available highway dollar.

CONCLUSIONS

1. The policy level currently must operate on incomplete information generally of poor quality. The proposed role of the PMS analysis method is to correct this problem by providing

- Complete and accurate pavement preservation and cost data;
- Economic analysis tools (network life-cycle cost analysis) to minimize the cost of pavement preservation for any given network condition;
- Network analysis tools (strategy analysis) that provide the ability to set MR&R program development constraints that will control long-term network condition and funding requirements;
- Analysis tools (based on the economic and network tools) to evaluate alternative funding schemes;
- Dedicated technical staff for the post-MR&R program development process whose ultimate responsibility is to reduce the cost of network preservation;

- Top-down decision making, which is made possible by having the policy level set the constraints for MR&R program development and by providing (via the PMS analysis method and utility software systems) monitoring capability that measures the efficiency of proposed MR&R programs and evaluates the cost effectiveness of technical staffs;
- Common terms (RSL, DSL, and lane-mile length) for describing the performance of projects, MR&R programs, and networks, which in turn provides an automatic interface between project- and network-level analysis and enables policy and technical levels to communicate using terms of mutual significance; and
- Means to quantify the benefits of alternative MR&R programs.

2. The agency's existing MR&R program development process is too complex and institutionalized, and it is unnecessary to insert or mix a PMS analysis method into it. However, PMS utility software is a necessary communication link between MR&R program development and the other three components of pavement preservation.

3. The proposed role for PMS analysis requires that the post-MR&R program consist of technical (applied research) staff. Typical technical skills include computer programming, pavement research, cost estimating, and pavement design. This provides for the policy level's need to have complete, accurate, and reliable data, information, answers, and state-of-the-art analysis capability at their immediate disposal.

GLOSSARY

Design service life (DSL): estimated number of years pavement is expected to be in acceptable condition.

Remaining service life (RSL): estimated number of years from the current year that pavement condition is expected to remain acceptable (RSL is a linear form of rate of deterioration, so PMS analyses based on RSL are simplified).

DSL and RSL categories: time is divided into 5-year categories so that the 0, 5, 10, 15, 20, 25, 30, 35, and 40 categories represent the periods 0–2, 3–7, 8–12, 13–17, 18–22, 23–27, 28–32, 33–37, and 38–42 years, respectively. For new projects, DSL and RSL are the same, and a project's RSL never exceeds it DSL.

Maintenance, rehabilitation, and reconstruction (MR&R): maintenance includes all preventive maintenance treatments that improve a pavement's condition and extend its RSL. All preventive maintenance treatments have a DSL; they are the bulk of projects that extend the RSL of currently acceptable pavements. Rehabilitation includes all project treatments that have a DSL and are not categorized as preventive maintenance or reconstruction. Reconstruction includes all project treatments that bury the original pavement or remove and replace one or more of its layers so that the reconstructed pavement has the same DSL and is in other respects equivalent to a newly constructed pavement.

MR&R treatment: any MR&R action that moves a section of pavement to a higher RSL category. All MR&R treatments are characterized by their DSLs.

MR&R projects: projects selected for, or proposed to be part of, future MR&R programs. They are identified by route title and other identifiers and by their begin and end location. They are characterized by lane-mile length and DSL.

MR&R program (also called *preservation program*): a list of MR&R projects selected for the annual improvement of performance of a designated network. MR&R programs are characterized by lane-mile length and the weighted average of the DSL (ADSL) of each of its projects. At time of construction, a project's ADSL equals its average RSL (ARSL).

MR&R strategy: a surrogate for an MR&R program, that is, MR&R strategies specify the lane-mile length of feasible MR&R projects and the percentage of network that is to be designed into each RSL category. A simple MR&R strategy would specify the lane-mile length (or percentage of network) and the ADSL of the MR&R program. MR&R strategies are used for planning and as development constraints.

Composite MR&R strategy: the planned use of a series of different MR&R strategies each to be applied for a specified time period, usually at least 5 years. Their purpose is to achieve incremental adjustment of network performance to reach ultimate network condition objectives at the least total network life-cycle cost.

MR&R strategy matrix: matrix that indicates the lane-mile length of pavement or percentage of network to be moved from each lower RSL category and to which higher RSL category it is moved.

Future MR&R requirements: MR&R strategy necessary to maintain or adjust the network's performance or condition.

Pavement condition: measured in terms of the pavement's longitudinal profile [roughness in international roughness index (IRI) inches per mile], transverse profile (rut depth to the nearest ⅛ in.), and an inventory of surface distress by type, severity, and extent expressed in terms of distress point accumulation. Each condition measure is summarized and reported for each contiguous 0.1-mi pavement segment. Two pavement condition categories are used, acceptable and unacceptable, on the basis of agency-established threshold values for each of the three measures of pavement condition. The condition of a pavement that is in acceptable condition is reported as the RSL of the condition measure having the shortest RSL.

Threshold value: value that defines each maximum acceptable measure of condition—IRI inches per mile, depth of rutting, and accumulation of distress. A pavement's condition is acceptable only if all three measures of condition are acceptable. Unacceptable condition occurs when one measure of condition reaches its threshold value.

Uniform sections: one or more contiguous 0.1-mi pavement segments whose condition or RSL may vary within specified limits. Uniform sections may be considered and treated the same as projects and are characterized by their lane-mile length and RSL or ARSL.

Network: state trunkline system or any designated portion thereof, consisting of contiguous uniform sections (projects)—the Interstate system, for example. Networks are characterized by lane-mile length, ARSL, percentage of network in unacceptable condition, and percentage of network in each RSL category.

Network condition: percentage of network in unacceptable condition.

Network performance: percentage of network in each RSL category. However, for planning and demonstration purposes, it is more convenient to indicate network performance in terms of the network's ARSL and the percentage of network in unacceptable condition.

Cost matrix: historical average cost per lane mile of construction for MR&R projects whose DSLs fall within each of the following DSL categories: 3–7, 8–12, 13–17, 18–22, 23–27, 28–32, 33–37 years, etc. More comprehensive cost matrices are developed on the basis of project analysis of 100 percent of the networks' uniform sections. Automated project analysis provides the cost, DSL, cost effectiveness, and benefits of all feasible MR&R treatments for all uniform sections in the network. From this project data and a designated range of cost effectiveness, the most cost-effective treatment for each uniform section provides the data for a cost matrix consisting of lane miles of pavement available in each RSL category and the lane-mile cost to move it to each higher RSL category.

Program development constraints: MR&R strategy and the funding level with which the MR&R program must comply to achieve the network performance and life-cycle cost required by policy makers.

Routine or reactive maintenance: maintenance conducted to provide reasonable pavement serviceability but not extend the pavement's RSL. The reactive maintenance workload is considered equal to the lane miles of pavement in unacceptable condition.

Life-cycle cost (LCC): total cost of ownership of a given section of pavement that occurs during the LCC analysis period. This ownership cost is considered to include the sum of the cost of annual MR&R programs plus the sum of the annual cost of reactive maintenance that is accumulated over the LCC analysis period. User costs are not included in the LCC analysis because it is assumed that current levels of network performance cannot be economically justified. That is, economic justification occurs when agency plus user cost is less than or equal to agency plus user savings that result from MR&R investments. It is believed that this is not the case, so minimum network LCC is based on the network performance specified by policy makers and the annual MR&R programs and reactive maintenance costs necessary to achieve and maintain that performance.

LCC analysis period: equal to the maximum DSL among all feasible MR&R treatments plus 5 years. In Michigan, where the maximum DSL is 35 years, a 40-year LCC analysis period is used.

Funding efficiency: ratio of cost of theoretically most cost-effective MR&R program to the cost of the proposed MR&R program.

PMS analysis method: application software system consisting of analysis methods for processing pavement condition and physical inventory data, automated project analysis, network analysis, and strategy analysis. It is thought that the PMS analysis method should not be agency-specific, but its products are not readily understood nor is it handy for novice personnel to use. Therefore, utility software is needed to make the PMS analysis method's products user-friendly and provide all the information users need in the form they desire. This utility software is agency-specific.

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Having a Regional Transportation Planning Agency Develop, Support, and Facilitate a PMS for Local Agencies

PAUL SACHS AND ROGER E. SMITH

A unique support relationship for a local agency pavement management system (PMS) has been established in the San Francisco Bay Area. Since 1984 the Metropolitan Transportation Commission (MTC), the regional transportation planning agency for the Bay Area, has supported the development and use of PMS by cities and counties in its region. During the past 7 years, 47 jurisdictions, representing more than half the street and road centerline mileage in the region, are at some stage of implementing and using the Bay Area PMS. MTC trains jurisdictions in PMS concepts, PMS computer applications, and PMS budget result interpretations. MTC conducts quarterly user meetings in which jurisdictions not only direct MTC staff on future modifications to the PMS but also work with one another to assist in PMS implementation. If requested, MTC PMS staff will present PMS budget results to participating jurisdictions to emphasize the importance of pavement management. MTC's continued support and facilitation have been major factors contributing to the success of pavement management at the local level in the Bay Area. This regional agency involvement is believed to be one of the most important innovations in support of local agency PMS.

In 1981 the Metropolitan Transportation Commission (MTC), a multicounty transportation planning agency in the San Francisco Bay Area, began to work with several local public works directors to help them document local agency needs and shortfalls in pavement maintenance within the Bay Area. This was needed to support the directors' requests for additional revenues for pavement maintenance from their locally elected officials. In 1982 MTC released *Determining Maintenance Needs of County Roads and City Streets* (*1*), which showed that Bay Area cities and counties were deferring pavement maintenance projects at a rate of $100 million a year. The report also documented that Bay Area cities and counties had an existing street and road pavement backlog of $300 million to $500 million. This report helped to convince the state legislature to increase the state gas tax from 7 to 9 cents. Of the 2-cent increase, 1 cent went to cities and counties for use on local streets and roads.

During the next 2 years MTC continued to work with a committee of local public works officials to assist them with evaluating and setting priorities for their road and street needs. A major recommendation from this study was that MTC adopt and support a pavement management system (PMS) for local agencies in the Bay Area.

In 1984, MTC began to develop a PMS (*2*). Six local jurisdictions (three cities and three counties) formed an advisory group that helped MTC monitor the PMS development. ERES Consulting, Inc., was retained by MTC to assist with this effort. The six pilot jurisdictions implemented the PMS in 1984, and by 1991 the PMS had been adopted by 47 Bay Area cities and counties. These jurisdictions are responsible for more than half of the 17,800 local street and road centerline miles in the Bay Area. The PMS has also been adopted by more than 75 other jurisdictions nationwide.

Besides supporting the development of the PMS software, MTC assists in every aspect of PMS implementation and operation. This includes training classes, presentations to directors of city and county public works departments explaining the PMS evaluation results, presentations to local elected boards and councils, and on-call (hotline) support. This support is one of the features that makes the MTC-supported Bay Area PMS successful.

The Bay Area PMS software, data collection, analysis procedures, and documentation were designed under the guidance of users. The programs have expanded, but the emphasis on user interaction has not changed. Users are surveyed to determine whether new procedures are desired and whether old procedures should be maintained or eliminated. MTC would never have embarked on the development of a project-level module or a mapping module without the support of its users. Through the user group meetings, which are held quarterly at MTC's offices, ideas are exchanged not only among MTC staff and personnel from participating jurisdictions, but also among users themselves. Because there are users at all stages of implementation, more-experienced users are willing to help others implement PMSs. It is a unique arrangement in PMS, and it takes place because a regional agency facilitates and promotes its use. This paper looks at three major areas where MTC has aided the implementation of the PMS in the Bay Area. It also looks at some instances in which MTC has assisted local jurisdictions in the continued use of the program after an agency has been through the entire process once. It is hoped that other regional agencies will gain from MTC's experience and adopt a similar program to help their local jurisdictions with PMS implementation.

MTC AS FACILITATOR

Although the PMS software provides the procedures needed by local agencies to implement network-level PMS (*3*) and

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project-level PMS (*4*), the major factor contributing to the success of the Bay Area PMS is the range of support activities provided to users by MTC. One of the reasons that many PMSs are discontinued or not fully used is that the PMS knowledge is developed in one person in each agency (*5*). When that person leaves the position, the expertise is lost and the system is no longer used. Another reason is the perceived complexity of the PMS process in general and the software programs in particular (*5*). To counter these problems, MTC has developed one of the most comprehensive support programs for local agency PMSs in current use. Three major elements of this are user meetings, user services, and budget analysis.

This support must address several types of organization. The nine counties in the Bay Area range from completely urbanized to primarily rural. Of the nearly 100 cities and towns, about a third are responsible for less than 50 mi of roads and streets, a third are responsible for 50 to 150 mi of roads and streets, and a third are responsible for more than 150 mi. Members from each group were among the pilot agencies, and MTC has continued to support agencies in all three groups.

User Meetings

One important support function is holding user meetings at the MTC offices. In the early PMS development stages, the six pilot agencies met monthly. Each agency shared experiences and identified problems with the PMS. This led to the realization that the user meetings should be an integral part of the support structure for the PMS. Training on the PMS elements was instituted as a regular part of these meetings.

The quarterly user meetings have become the focal point for identifying changes and enhancements needed for the PMS. All agencies using the Bay Area PMS are encouraged to attend the meetings. Early in system development, MTC asked the pilot users what they liked and didn't like about the system. If, for instance, two of the jurisdictions wanted the ability to change costs and treatments in the decision tree that was being developed, it was through the user meetings that they expressed their opinions. If they could convince a majority of the six users, the program would be developed accordingly. All decisions about the program were made in this manner.

In the user meetings, the users direct MTC on needed modules or enhancements for the PMS. About once a year the users are surveyed as to what they like most about the program and what features they would like to add. In the past, the development of a project-level module and the development of a mapping module have been rated high priorities. During the user meeting week, subcommittee meetings are often held on the development of major new modules

When the initial computer program was being developed, MTC also conducted training for the participants. The training sessions included (a) establishing a PMS work plan and steering committee, (b) network breaking, (c) identifying distress, (d) interpreting PMS budget results, and (e) PMS computer training.

After the initial release of the PMS program, new users were added. In 1991 there were 47 local cities and counties at some stage of implementation—some into a third or fourth iteration of the program. The significance of the user meetings has not decreased. The meetings now include a wide area of activities. A general user meeting is held in which MTC staff present developments since the last meeting. MTC has established a method of tracking the progress of each user from the time that the user begins PMS implementation to the time he or she is using the PMS to develop the overall city or county budget.

The user meeting is a breeding ground for communication between old and new users. Since the program has been in operation, many of the more-experienced users have made presentations at the user meetings to new and potential users. The user meetings give each user an opportunity to discuss experiences. Those that develop new methods to accomplish a particular task are asked to present them to the other users. The meetings also offer ample time for one-on-one discussion. In effect, the user meeting has become a support group for users to share their thoughts and problems.

Other meetings also occur during the week of the user meeting. MTC provides training sessions on a cyclical basis. Each year there are usually 5 to 10 new users from within the Bay Area. These users need to be trained on establishing a PMS work plan, goals, a steering committee, and such. In a quarterly meeting the basic PMS training sessions are provided in (new) user orientation, network breaking, distress identification, PMS budgeting, and so on. These training sessions, though primarily developed for new users, are also attended by new personnel in established user agencies and those desiring refresher training.

It is essential that this training be available for the experienced users. Many agencies experience staff turnover in the departments responsible for PMS and send their new staff to the meetings. Early on, MTC found that the quickest way for a PMS to be put on a shelf in a city or county was for the individual responsible for PMS to leave or to be promoted. MTC staff is in contact with each of its participating users in the Bay Area and works to get new staff trained in PMS concepts and the correct use of the PMS program.

As users began to expand beyond the Bay Area, MTC realized that not all users would be able to attend the user meetings or the training sessions. To assist users who cannot attend the training sessions, MTC has videotaped five of the training sessions and provides them to jurisdictions at cost. The tapes train users how to break the network, identify distress, and use the microcomputer as it relates to PMS. Agencies in the Bay Area often use the tapes to supplement the training that the MTC staff gives.

Another important service that was touched on earlier is the retraining of users who have a new person assigned to the project. This retraining is the most important step for a city or county to make if they want to continue using the PMS. MTC staff provides the service with either an on-site visit or invites the staff from the jurisdiction to the MTC for training. In some instances MTC has retrained four different people from the same jurisdiction over the past 3 to 5 years.

MTC has recently added technology transfer seminars to the quarterly meetings. They have covered a wide range of topics, from the correct application of slurry seals to overlay design. After each meeting individuals are asked what type of topics should be covered in the next seminar. These seminars have improved the understanding of PMS concepts among the various groups affected by PMS and have been well attended. MTC has also published a quarterly newsletter for the past 4 years that is distributed at the meeting. The newsletter includes computer tips, new maintenance strategies, potential funding sources for street and road programs, and other such topics. More recently, each newsletter includes an article written by a user; the article describes how the user has benefited from the PMS program. The articles provide impetus for newer users in the program as well as for older users who have not progressed as fast as others.

The general user meetings also provide an opportunity to respond to user questions and concerns about the computer program. MTC has adapted and modified the computer program many times through this process. Many suggestions and recommendations from the users that were not part of the original PMS have now been added. For instance, the program is now able to split and combine sections. This is necessary if a treatment is applied to part of an original section but not the remainder.

User Services

In MTC surveys of users, on-call and on-site assistance is always rated as the highest priority. When MTC developed the PMS computer program, it soon found out someone needed to handle computer hotline calls when users encountered problems. Nothing frustrates a user more than trying to get a report out for the boss but getting only an error message. A few of these incidents will lead to loss of credibility and discontinuance of use.

MTC has instituted a hotline for questions ranging from how to turn on a computer to how to interpret PMS results. MTC staff now tracks all calls to find common problem areas, which are then discussed at the general user meetings. The hotline also provides other information to MTC staff.

MTC originally believed that it was not its responsibility to train public works department staff in DOS or in RBase, the data base manager used in the computer program. Through the hotline questions MTC realized that if the PMS were going to be used properly in every jurisdiction, basic DOS and RBase instruction had to be offered. These classes are now given every 6 months at the MTC offices.

The hotline also helps MTC staff identify user agencies with personnel newly assigned to the PMS or otherwise inexperienced with PMS concepts. The hotlines are used to invite new personnel to the MTC offices for appropriate training in the normal training sessions or in one-on-one sessions.

The most important feature of the hotline is that it provides the answers to users' questions on the spot. In most cases the users can be coached through their problems and will be able to continue using the program. On occasion, when the problem cannot be solved on the phone, MTC staff will go the extra mile. If the problem is urgent, the user will be invited to MTC with the data base and the MTC staff person will "recreate" and correct the problem, or an MTC staff person will make an on-site visit to debug the data base and get the user back on-line. If time is not urgent the user will send the data base to MTC staff for review and debugging. When the data base has been fixed, it is sent back to the user.

Another important service that MTC provides for new Bay Area users is an on-site visit for software installation, distribution of the PMS user guide, and a walkthrough of the PMS program. The on-site visit helps MTC staff determine the experience level of the new user in computers and the type of computer being used, and—probably most important—the user gets to know the individual on the phone should a hotline call be required.

As mentioned previously, MTC has developed a method by which users are tracked through their PMS implementations. When users begin they are assigned an "F." As they move through the PMS process they move up in letters: "E" means they are breaking networks and doing distress surveys; "D" means they are developing budgets, and so on. A user who has reached "A" has made a budget presentation to his or her elected board or council and has begun to implement the maintenance program. MTC makes these ratings every quarter. The ratings help MTC determine if there are users who need special assistance. Most of the time, those jurisdictions needing special assistance are the smaller cities with less than 50 mi of roads. They often do not have the personnel for the PMS implementation. MTC has identified a number of consultants who can help these agencies implement the Bay Area PMS.

The user services component of the PMS has proved to be an invaluable tool in facilitation on the part of MTC. It has built a trust with the local jurisdictions, because they know they have someone to call if problems arise in implementation. Without the user service component, the PMS would have been discontinued in many jurisdictions because of staff turnover.

Budget Analysis

Main goals of network-level pavement management are to determine budget needs and to substantiate the impact of budget options on the future condition of the network, future funding needs, stopgap funding needs, and backlog of funds. This information is used at a regional level to help substantiate the need for funding at the regional and state levels. It is also used at the local level to justify budget requests. MTC has developed a program to assist at both levels.

Regional and State

As each user completes the budget portion of the PMS, MTC requests that their data base be sent to MTC. MTC then compares their 5-year budget need to expected revenues for pavement expenditures. A regional aggregate of 25 users shows that on average, San Francisco Bay Area jurisdictions are spending roughly $0.39 when they should be spending $1.00 for maintenance and rehabilitation of pavements.

An earlier version of this regional aggregate was used in 1988, when the California State Senate asked regional agencies statewide to develop a 10-year estimate of needs and expected revenues for streets and roads. Using the city and county data bases that it had at the time, MTC produced a 10-year needs assessment for Bay Area streets and roads; the assessment showed that the Bay Area needed $2.2 billion for pavement maintenance but could only expect just over $1 billion in revenues. These figures were used by the senate to develop the bills that became Proposition 111 and Proposition 108. These propositions, which were passed in June 1990, increased the gas tax from $0.09 to an eventual $0.18. The increase in the gas tax is expected to raise $15 billion over 10 years, $3 billion of which is to be directed to cities and counties for use on local streets and roads.

MTC staff continues to encourage its users to complete its PMS in order to refine and update its regional aggregate needs and shortfall chart. In this way MTC is able to act as an advocate for additional revenues from a regional perspective.

Local

When a city or county completes its budget portion of the PMS, MTC prepares a document for the jurisdiction that MTC calls a budget option report (BOR). This report

- Reviews historical revenue and expenditure levels for street and road purposes;
- Estimates, from historical spending levels, future revenues for street and roads purposes for a 5-year period;
- Estimates a percentage of future street and road revenues that will be used strictly for pavement maintenance;
- Compares estimated revenues for pavement maintenance against actual need, as derived from PMS estimates;
- Documents ensuing shortfalls and surpluses for a 5-year period on the basis of projected funding;
- Develops other options to compare with the estimated level of pavement maintenance expenditures; and
- Offers recommendations on how the jurisdiction might want to proceed with its pavement maintenance program.

MTC has prepared BORs for 25 jurisdictions. One of the first agencies to receive a BOR was the city of San Leandro, in Alameda County. In April 1986, the BOR was presented to the San Leandro City Council, which informed them that the city's 5-year need for pavement maintenance was $11.5 million. Revenues for pavement maintenance over that period were estimated to be only $5.5 million. Seven months later the council requested that the department of public works and MTC staff deliver a formal presentation on the needed pavement maintenance revenue.

In the meantime, a ballot measure was placed before Alameda County voters that would increase the county's sales tax by a half cent for transportation purposes. Almost 20 percent of the revenue generated from the proposed increase would go to the city and county public works departments for use on streets and roads. In San Leandro's case, the estimated percentage of revenue being returned to them was approximately equal to the $6 million pavement maintenance shortfall. The evening before the vote on the referendum, San Leandro public works and MTC staff went before the city council. The council, after hearing the presentation, determined that if the referendum passed the next day, the portion of funds to be returned to the city would be used for pavement maintenance. Voters passed the referendum, giving San Leandro a steady source of revenue for pavement maintenance.

In July 1988 a BOR was presented to the city council of Vallejo in Solano County, showing a $14 million need and estimated revenues of $6.7 million. The year before using the PMS, Vallejo had a pavement maintenance budget of $900,000. The first year after its use the council devoted $1.4 million to pavement maintenance. Each year since then the council has increased the pavement maintenance budget. For FY 1990–1991 the budget is close to $2 million.

The city of Benicia in Solano County completed its condition survey in late 1989 and received a BOR in early 1990. The BOR showed that it had a $7 million need over 5 years and revenues for pavement maintenance were estimated at $2 million. Using the executive summary of the BOR, public works officials were able to secure an increase for pavement maintenance from $200,000 to $300,000. The city spent this in the first half of FY 1990–1991. The public works department went back to the council to ask for more funds and was able to secure another $400,000. In total, the city of Benicia was able to increase its expenditures to pavement maintenance by 350 percent in 1 year.

MTC has found that, though this process is time-consuming, it remains one of its most important roles. One of MTC's major interests in developing and continued support in the PMS is for cities and counties to use the results from the PMS to improve their pavement networks. MTC offers its services to jurisdictions to help them interpret the PMS results and to make presentations to their public works directors and locally elected board or councils to assist with the process. This assistance helps build confidence in the PMS and helps develop competence within the budget development and justifications in the agencies.

Does Facilitation Promote the Use of PMS?

Last year MTC staff analyzed data from the state of California to determine if MTC PMS users were increasing revenues for pavement maintenance.

Each public works department in California is required by law to report the source of its street and road revenues and how and where they are spent. MTC analysis included the period from FY 1980–1981 to FY 1988–1989.

The data for the 9 years were broken down into two analysis periods: FY 1981–1984 and FY 1985–1989. The PMS became available to Bay Area cities in FY 1985. During FY 1981–1984 Bay Area PMS users spent 23.5 percent of total street- and road-related revenues on pavement maintenance, whereas from FY 1985–1989 users spent 37.8 percent: a 62.1 percent increase in expenditures for pavement maintenance. From 1980–1981 to 1983–1984, other Bay Area cities spent 35.5 percent of total street and road revenues on pavement maintenance; from 1984–1985 to 1988–1989 they spent 31.4 percent, an 11.5 percent decrease in expenditures for pavement maintenance.

Pavement maintenance expenditures per mile were also analyzed. Broken down into the same time periods mentioned previously, the data show that MTC PMS user cities spent more than nonuser cities. From 1980–1981 to 1983–1984 MTC PMS users spent an average of $5,294/mi on pavement maintenance. From 1984–1985 to 1988–1989 an average of $10,792/mi was spent, an average increase of 103.9 percent. Other Bay Area cities spent an average of $7,498/mi on pavement maintenance from 1980–1981 to 1983–1984. From 1984–1985 to 1988–1989 an average of $8,949/mi was spent, a 19.4 percent increase.

Resources Devoted to PMS

From July 1984 to February 1986, MTC devoted the equivalent of 5.5 person years to the project. This was the development period. The cost was about $300,000. Since then, MTC has maintained the program at between 4 and 4.5 person years for every fiscal year. This calculation includes all professional staff time as well as support staff time. The cost per year has ranged from $250,000 to $350,000. When MTC embarked on the PMS project, it believed it was important to assist the local agencies in the Bay Area to better maintain their streets and roads. MTC therefore juggled its priorities and, with existing funding, developed and has continued to maintain the PMS. On top of the staff time, MTC has hired consultants. In the development stage the cost to MTC was approximately $180,000. During the past 5 years, MTC has spent an average of $50,000/year on consultant services.

For FY 1990–1991 the cost to support the program was approximately $400,000. Divided between the 47 jurisdictions this amounts to roughly $8,500 each. The cost to develop a PMS at the local level varies from $100 to $300/centerline-mi (*6*). The 47 jurisdictions maintain roughly 9,500 centerline-mi of streets and roads. This would amount to between $950,000 to $2,850,000 if they were to do it on their own.

CONCLUSIONS

MTC provided the Bay Area jurisdictions with PMS software that reduced the cost of adopting a PMS, thereby making use more likely by local agencies. Early in the process, MTC found that training and long-term support were as necessary as the software capabilities in successful PMS application. MTC has developed a series of support services that have proved to be of great value to the successful implementation and use of pavement management at the local-agency level. These services—which include user meetings, user services, and budget analysis—help personnel in agencies get started in PMS with the training and support needed to begin pavement management. They also assist agencies that have been using the PMS with the training and support needed to train and retrain current and newly assigned personnel. The user meetings are a focal point from which MTC takes direction on improving and modifying the software, training, and other support functions. This unique relationship has proved successful and demonstrated that success of PMS at the local-agency level is as much a function of the support available as it is the software. It is hoped that the MTC's experience can be used as a model for PMS support in other regional transportation planning agency areas.

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Developing an Interface Between Network- and Project-Level Pavement Management Systems for Local Agencies

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A network-level pavement management system (PMS) has been used by cities and counties of the San Francisco Bay Area since 1984. Several agencies need to extend the analysis to the project level. Network-level PMS decision support systems use microcomputers to provide relatively simple support. However, project-level PMS elements currently require much more detailed analysis, which often requires the physical testing of materials. Much project-level analysis must take place outside the computerized decision support process. The Bay Area project-level PMS elements were developed to provide the needed support while maintaining an interface with appropriate analysis techniques used in the network-level decision support system. The project-level PMS programs use the existing data in the network-level PMS data base and allow the addition of information from analysis conducted outside the computer programs. A guide for conducting project-level evaluation to identify feasible alternatives and develop cost-effective treatments was prepared. The project-level programs allow the definition of contract and construction packages and manual intervention to adjust the date of construction when constraints not considered by the program are present. The calculation of the effects of maintenance and rehabilitation use the same general principles employed in the network-level analysis; however, the approach is modified to make use of the more-complete data collected.

A network-level pavement management system (PMS) has been used by cities and counties of the San Francisco Bay Area since 1984 (*1–3*). As the agencies used the network-level PMS elements, they found that they also needed assistance with the analysis at project level. Project-level PMS elements have been developed for the Bay Area PMS through the support of the Metropolitan Transportation Commission (MTC), Oakland, California (*4*). The project-level decision support system was developed to interface with the network-level elements to make it more adoptable and usable by Bay Area agencies.

BACKGROUND

The Bay Area network-level PMS elements were developed under the guidance of a group of Bay Area public works personnel employing the diffusion of innovation concepts to make the PMS easier for Bay Area public works agency personnel to use (*2*). To continue these concepts through developing the project-level PMS elements, a committee was formed of public works personnel who use the Bay Area PMS and were interested in the project-level system. This group reviewed the elements as they were developed and provided feedback to the developers. They are currently testing the procedures. Diffusion of innovation concepts was again employed to help make the system more adoptable and usable for public works agencies.

Pavement management is generally described and developed at two levels: network and project. The primary differences between network- and project-level decision support tools include the level for which the decisions are being made and the amount and type of data required (*5,6*).

The differences in decision level are normally found in the quantity of pavement being considered and in the purpose of the decision. In network-level analysis, agencies generally include all of the pavements under their jurisdiction; however, they may also break out subsets, such as primary arterials, that are managed separately from the remaining network. The quantity of pavement considered at the project level is normally a single management section, which also often corresponds to an original construction section, though sections may be combined or subdivided in analysis and design.

The purpose of the network-level system is usually related to the budget process of identifying funding needs for pavement maintenance and rehabilitation and determining the effects of various funding scenarios on the health of the pavement system and on the overall welfare of the community. The primary results of network-level analysis include fund needs, forecasted conditions for funding scenarios, and priority listings of pavement sections for programming maintenance and rehabilitation. At the project level, the purpose is to provide the best maintenance or rehabilitation strategy possible for the selected sections of pavement for the available funds. The primary results of the project-level PMS include an assessment of the cause of deterioration, identification of possible strategies, and selection of the "best" strategy, given the constraints.

Data collection is expensive and time-consuming. When engineering analysis and design are being conducted to determine the type of maintenance or rehabilitation to apply to a section of pavement, it is often difficult to determine exactly the type and amount of data required until some of the data have been collected. Excessive data collection is one of the problems causing PMSs to fail or be discontinued (*3*). To avoid excessive collection, the absolute minimum data nec-

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essary are normally collected at the network level. This allows the PMS to be implemented with less initial investment; however, the data collected for the network level are not adequate for making most project-level decisions. More-complete data must be collected on individual management sections of pavement identified as candidates for maintenance or rehabilitation when the project-level analysis is needed. If the project-level data are retained when collected, a more-complete data base can be developed without an excessive investment in data collection at any one time. The need to minimize initial data collection is a primary reason for developing separate network- and project-level PMS elements.

BAY AREA NETWORK-LEVEL PMS ELEMENTS

Network-level elements were developed first because they were the ones that the Bay Area public works personnel thought they needed most (*3*). Once the decisions with which the users needed assistance were identified, only the data needed to support those decisions were identified for collection. This approach produced a streamlined system requiring a minimum amount of data to manage the system at the network level.

The network-level system contains five general categories of components:

1. An inventory of the pavement network,
2. A method to determine the condition of the pavement segments,
3. A procedure to determine maintenance and rehabilitation fund needs,
4. A method to select candidate sections when funding is constrained, and
5. A method to show the impact of budget decisions on the health of the network, fund backlogs, and future fund needs.

The pavement network is divided into segments with relatively uniform characteristics. Each segment is expected to receive the same maintenance or rehabilitation treatment. These segments are used as both management and data collection sections.

The PAVER surface observable distress–based pavement condition index (PCI) was modified for use in the Bay Area. It is used as the network-level measure of condition (*7,8*). The number of distress types was reduced to seven, and the distress survey procedures were simplified. The PCI from the latest inspection, the PCIs from all prior inspections, and the distress type, severity, and quantity for each section from the last inspection are retained in the data base. The condition of each section, in terms of PCI, is projected over the analysis period using a family curve concept adjusted for the performance of individual management sections. The projected condition of each management section is then connected to a maintenance and rehabilitation cost at a designated period through a set of decision trees that assign a network-level funding category treatment to each management section identified as needing maintenance or rehabilitation. The development of treatments, costs, and decision trees is described elsewhere (*1,2,8*). The funds needed for each management unit are calculated and summed by treatment type for each year of the analysis period to determine network budget needs unconstrained by available funds.

When funds are limited, an analysis is used to designate those management units for funding that provide the best network value for the money. A cost-effectiveness analysis is used to rank pavement sections for fund allocation, which is similar to a benefit-cost analysis except that a surrogate effectiveness is used in place of a directly calculated benefit (*2*). The area under the PCI-versus-age curve for individual sections is defined as the effectiveness. The basic hypothesis is that user utility (noncosted benefit) is the mirror image of performance (*9*). The ratio of the expected effectiveness per year for the identified maintenance or rehabilitation treatment to equivalent uniform annual cost per square yard is calculated for each section of pavement. The ratio is then weighted for level of usage. The weighted cost-effectiveness ratio is used to rank the sections to determine which ones should be selected for funding.

To determine the area under the PCI-versus-age curve, the PCI is projected to a terminal condition. It is also adjusted to reflect the expected influence of the maintenance or rehabilitation treatment used to determine the cost and projected until it reaches the same terminal condition. The area between these two curves is calculated to determine the effectiveness of the treatment.

The pavement manager can allocate different percentages of the funding to preventive maintenance to help select the best division of funding between preventive maintenance and rehabilitation. Different total funding levels can also be analyzed in a series of funding scenarios. The network-level support elements provide information on the sections selected as candidates for maintenance and rehabilitation, condition of the network, deferred funds, and stopgap funds required. It is expected that the pavement manager will use a series of these analyses to compare funding scenarios in order to develop recommendations for required funding levels and select the best considering the funds allocated to pavement maintenance and rehabilitation. However, the treatments used in the network-level analysis were primarily developed to identify budget needs and fund-allocation effects; they were never intended to be applied to the pavements without a project-level analysis.

Engineering analysis and design are required to determine the specific treatment to be applied to any pavement management section identified for rehabilitation by the network-level analysis. In addition, whereas the equations and relations used in the network-level analysis to calculate the treatment's impact on the PCI are believed to be adequate for that decision level, the developers were concerned about their applicability to project-level decisions. The network-level methods for calculating effects of treatments on PCI are described in detail by Smith et al. (*10*).

PROJECT-LEVEL ELEMENTS

Although the Bay Area public works agencies have been responsible for maintenance and rehabilitation design for many years, the implementation of the network-level PMS introduced them to more treatments and more structured analysis concepts than previously used. They then requested that the Bay Area PMS be extended to provide assistance at the project level and offered to assist by guiding the development and trying the elements as they were developed. Although the project-level PMS elements constitute a new class of decision support for the public works personnel, the developers and the user committee were determined that it would not duplicate effort from the network-level system. The results of the network-level programs are used as the starting point for the project-level analysis.

The data used in the project-level analysis were selected to interface with the data collected at the network level. The future need for this interface was considered in the development of the network-level elements. This might not be a problem in most state PMS support systems, because many appear to have started at the project level and progressed to network-level elements. But in most local agencies, PMS started with network-level elements and progressed to project-level elements. A more-complete description of the data is provided later.

In early trials, the using agencies ran their network-level analysis and then put the resulting section information into the project-level analysis. If they didn't get similar results, they believed something was wrong with the project-level system. Once a user becomes comfortable with a network-level system, the project-level system should give similar results when the same data are used if the project-level system is to have any credibility. The same general analysis concepts used in the network-level decision support software were used in the project-level elements whenever appropriate.

The Bay Area network-level system uses a 5-year analysis period. In the project-level system the user can choose from 1 to 5 years for the analysis period. This allows the user to pick the years for which the more-detailed analysis will be completed. The project-level analysis begins with candidate sections identified by the network-level decision support system; however, other sections can be manually added to the analysis list, and those selected by the decision support system can be removed from further analysis.

Outside Constraints

Programmed and concurrent activities outside the pavement maintenance arena affect pavement maintenance and rehabilitation planning, especially at the project level. Programmed activities are constraints that affect the scheduling of the treatment, such as planned renovation of an underground utility. Concurrent activities are those that are traditionally completed, or required by policy to be completed, when pavement maintenance or rehabilitation is applied. Concurrent activities affect the cost associated with applying the treatment and include activities such as sidewalk repair, drainage repair, structure adjustments, and safety structure construction.

To address this, the Bay Area PMS data base was modified so that constraints could be entered. The pavement manager identifies whether the activity will constrain pavement maintenance and rehabilitation or whether the work will be performed concurrently. Dates for constraining activities are entered, if appropriate. For constraining activities, the project-level decision support program adjusts the affected management section treatment dates to no earlier than the constrained date. For instance, if a water line under the street is scheduled for replacement in 1993, the street work will not be scheduled before 1993.

The costs associated with constraints can be entered; however, if a cost is entered, the pavement manager must also identify whether the cost should be considered in the cost analysis. Costs associated with renovation of utilities are normally borne by the utility agency; these need not be entered because they are not associated with the pavement repair and would not affect the analysis. However, costs required to adjust the height of guard rails and utility structures (e.g., manholes and valve boxes) in the street for an overlay would be included and considered in the cost analysis, because they vary with alternatives. Costs to repair sidewalks may need to be tracked, because they are borne by the public works agency, but they should not be considered in the cost analysis to determine the best treatment, because they do not affect pavement performance nor are they associated with a specific treatment. Both types of cost are tracked by the program and reported in the final analysis.

Contract and Construction Package Development

Management sections that have uniform characteristics reflecting past construction and maintenance efforts in cities and counties often include relatively small pavement areas. Many times the network-level analysis will identify many sections for the same treatment spread over the network for each analysis year or several diverse treatments to small street sections with a small geographic area. To gain efficiencies of scale, most public works agencies prefer to apply the same treatment to several management sections within a geographic area at one time. This is often called a chip seal program or overlay program. Agencies seldom apply an overlay to two blocks, heater scarify and overlay one block, skip two blocks, apply a chip seal to one block, and skip two more blocks before reconstructing three blocks, all along a section of street; they generally try to find an appropriate treatment for all of the sections with minor changes in surface preparation, base modification, or overlay thickness. If two management sections need a treatment in 1 year and the management section connecting them is identified as needing a similar treatment soon, the agency often applies a treatment to all three sections in the same year. Thus, considerable modification in management section selection occurs in the development of final projects by grouping management sections into contract or construction packages based on geographic location, type of treatment, and date of treatment.

The Bay Area project-level programs allow the user to define these packages. The basic information is retained on individual management sections in the data base; however, the management sections are combined for final analysis and treatment development at the project-level. The programs allow the development of construction and contract packages during all phases of project-level evaluation. Once the package has been defined, the costs, PCI increase, and cost-effectiveness will still be calculated for each management section; however, the cost-effectiveness will be weighted on a square-yardage basis for the entire package and that value will be compared to the cost-effectiveness of other packages. Individual management sections not included in contract and construction packages are considered a single section package in the analysis.

Project-Level Evaluation

Currently, much of the project evaluation must be completed outside the actual computerized decision support programs. The program is set up to allow the engineer to begin analysis, reach some point at which information is not available to complete the next step, leave the program, return to the program later, and continue the analysis without losing any information or steps.

Pavement evaluation is a complex engineering problem that requires a systematic approach to quantify and analyze the many variables that influence identification and selection of appropriate maintenance and rehabilitation treatments. In new design, many design parameters are assumed or developed from laboratory tests. However, many of the materials to resist damage induced by traffic and the environment are in place when maintenance and rehabilitation are being planned, and the existing material properties can be determined along with the condition, traffic, and other constraints. Project-level analysis can be approached as a series of steps to determine the cause of deterioration and identify relevant constraints (*11*). The answers are then used to identify practicable treatments. However, it is essential that the process determine the cause and extent of deterioration to ensure that the solution addresses the cause of the problem rather than just a symptom.

The size of the project and importance of the street or road to the agency influence the amount of time and funds that will be expended in project-level evaluation. Pavements on high-volume major roads and streets should be subjected to more testing and evaluation than those on low-volume roads and streets. The concepts and evaluation procedure described are valid for a road or street with any volume of traffic; only the amount of testing and time spent in reaching the conclusions should vary.

A question-answer–oriented project-level evaluation should address the following questions (*11,12*):

1. Is the pavement structurally adequate for future traffic?
2. Is the pavement functionally adequate?
3. Is the rate of deterioration abnormal?
4. Are the pavement materials durable?
5. Is the drainage adequate?
6. Has previous maintenance been abnormal?
7. Does the condition vary substantially along the length of the project or between lanes?
8. Does the environment require special consideration?
9. What traffic control options are available?
10. What geometric factors will affect the design?
11. What is the condition of the shoulders?

Questions 1 through 6 address the cause of deterioration; Question 7 helps determine if there should be a change in the basic management section; and Questions 8 through 11 identify special constraints that must be considered. Detailed checklists have been presented by Smith and Darter (*11*) and AASHTO (*12*).

The Bay Area network-level PMS uses the distress-based PCI as the basic measure of condition, and it is a good tool for the network level. But at the project level, although PCI can be used to identify abnormal rates of deterioration and variances in the performance of subsections of a management section, it does not adequately define either the functional or structural condition. Information on the specific type, amount, and severity of the distresses is more important. Extrapolated distress data are stored for each management section of pavement in the network-level Bay Area PMS data base; however, it may be necessary to collect more-complete or more-recent distress data for project-level analysis, because the distress data are generally based on a sample of the section area in the network-level analysis and it may have been some time since the inspection was completed. The distress data often need to be supplemented with additional measures of condition to address the questions just described; however, sometimes the distress information alone is adequate.

To guide the Bay Area PMS user through this question-answer process, a manual was prepared that describes ways to ask the questions, data to be used to answer the questions, and alternatives to be considered. By adopting project-level PMS elements that complement the network-level system, the minimum required data can be collected during the network-level surveys and more-complete data can be developed and captured by the PMS over a long period through project-level elements when that data are necessary to support the decisions being made. The data used at project-level complement rather than duplicate the data collected at network level. Data collection is spread over a longer time, which makes the PMS more adoptable to an agency selecting a PMS to implement; unnecessary data, or data that become obsolete, are not retained to impede analysis or affect future decisions. This allows only the data to be collected only when they are needed and reduces implementation costs.

Decision trees were used to identify alternatives in the network-level analysis, so the same concept was applied to the project level. Decision tables were prepared for each of the seven individual distress types and the three severity levels for a reasonable range of densities. Practicable treatments were then identified for each category. In general, as density increases, the treatments change from localized repair to area coverage; as severity increases, the treatments change from light surface repairs to heavy rehabilitation. These decision trees generally provide more than one alternative; in some cases they provide several. They are meant to be advisory and used by the newer engineer to identify feasible treatments and strategies.

The distress-based decision trees were modified to show how the alternatives would be modified if the pavements also experienced structural and functional problems. Feasible maintenance and rehabilitation strategies are identified for a pavement section on the basis of the individual distress type, severity, density combination present when the section is structurally deficient, excessively rough, or has poor surface friction.

Structural adequacy indicates the ability of the pavement to withstand the expected traffic loadings. The presence of certain distress types—for example, alligator cracking and rutting—can be used to determine how the pavement has performed structurally to the present; however, it is difficult to use distress to predict structural performance, especially if traffic has changed recently or is expected to change significantly in the future. All rational overlay design procedures use some method to determine the additional thickness needed for future traffic loadings. To supplement the distress data available, the analyst is requested to conduct an overlay design for the existing pavement. It is assumed that the pavement is structurally adequate if an overlay is not required. No specific overlay design procedure is required by the program, but deflection testing or component analysis based on cores and borings is recommended in conjunction with traffic projections, at least for higher-traffic pavements. Most local agencies do not have deflection-testing equipment; however, several consulting firms provide deflection testing and overlay design services. The California Department of Transportation deflection-based overlay design procedure is generally recommended for the Bay Area agencies if other methods are not being used by the agency or consulting firm conducting the analysis. It is recommended that structural problems be considered first because if a structural overlay or other rehabilitation treatment is applied, it will generally correct roughness and surface-friction problems also.

Functional adequacy is normally used to describe how well the pavement meets its basic purpose of providing a smooth and safe riding surface. It is usually measured in terms of roughness and surface friction. The Bay Area network-level PMS does not address roughness or surface-friction problems directly. Some indications of such problems can be surmised from distress information, for example, that pavements with severe distortion problems are generally very rough and that pavements with severe rutting generally have surface-friction problems in wet weather. However, other measures may be advisable during project-level evaluation, but many agencies do not have the funds to quantify the measures mechanically. Particularly for lower-volume pavements, a quick ride over the section by the design engineer is normally used to determine if the pavement has roughness problems so severe that they be addressed specifically. However, more-quantifiable methods of measuring surface roughness are described in the manual for those agencies that have the resources and the need to measure roughness. While most agencies will not purchase roughness equipment, consulting firms can provide the measuring and analysis services. Roughness analysis should be completed before consideration of skid problems, because the feasible treatments for correcting roughness problems can also correct surface-friction problems.

Surface friction is not generally measured by cities and counties, but accident location maps—especially wet-weather–accident locations—can be used to find areas that have surface-friction problems. Methods to measure skid in localized areas are described for the agencies because several law enforcement agencies in the area use them in accident analysis. The agencies are encouraged to use these devices during project-level evaluation when they are available, especially on their high-volume roads and streets and at intersections at which skid-related accidents have been reported. Skid-measuring services are also available from a few consultants.

The rate of deterioration is often used to program the time at which maintenance or rehabilitation should be applied, as well as to assist in determining the cause of deterioration. Timing of the application is explicitly used in both the network and project-level programs of the Bay Area PMS through the projected PCI. High rates of deterioration are considered to be associated with structural deterioration of pavements, and environmentally caused deterioration is expected to have a slower rate. If the pavement life has exceeded the original design life and has recently reached a level at which rehabilitation is being considered, the pavement may be capable of being rehabilitated with a minimum-cost treatment if traffic is expected to be the same; this may be much less expensive than the network-level treatment identified by the PMS software. However, if the pavement requires rehabilitation in a period much shorter than its design life, or if the traffic is expected to increase dramatically, reconstruction or some other extensive rehabilitation technique might be necessary. Rate of pavement deterioration can be measured in terms of the PCI change per year or increase in the amount of a distress type. Past and projected rates of deterioration are available from the Bay Area PMS programs based on PCI.

The localized variation along a section or between lanes can be determined by plotting the PCI versus section length or across lanes; however, the network-level system stores only the average PCI for the management section, and that is normally based on a small sample of the section area. For at least the major arterial streets, a more-complete distress survey is recommended. The PCI calculation program provides the PCI values and distress information for the individual inspection units, which can then be used to determine if there is significant variance; if there is, the sections can be subdivided for further analysis.

Information on drainage and material durability is not available in the network-level data. The manual advises the engineer on how to consider drainage and material durability in developing alternatives to find those that will address the cause of deterioration; it also gives some guidance on how to identify problems associated with each. In general, the presence of either will reduce the alternatives. Special environmental constraints are often not considered important in city and county analysis; however, in the Bay Area, several agencies have pavements both near the bay and in the adjacent hills. The subgrade types and the natural drainage differences between these locations affect the performance and must be considered in the analysis, often leading to the selection of different alternatives. Finally, the other constraints such as geometric factors and traffic control options are used to develop a final set of alternatives.

The PMS decision support programs are then used to conduct a life-cycle cost and cost-effectiveness analysis to identify the treatment that provides the best return the least funds. The engineer can select one treatment for each package to be given priority or allow the program to select the one with the highest cost-effectiveness ratio. A number of iterations may be required for some sections to complete the process of identifying the best solution and developing construction and contract packages.

PRIORITY SETTING

The treatment costs and effectiveness values are entered into the project-level programs to adjust the network-level fund allocation. The same cost-effectiveness analysis used in network-level analysis is used to rank pavement sections identified for maintenance and rehabilitation from highest to lowest weighted effectiveness-cost ratio. In the network-level system, available funds were defined for each year of the analysis period. This same procedure is used in the first ranking by the project-level program. Sections are selected for funding from the ranked list. The following equation is used to calculate the weighted effectiveness ratio (*2*):

$$\text{WER} = \frac{(\text{AREA/YR})\ \text{WF}}{\text{EUAC/SY}} \tag{1}$$

where

WER = weighted effectiveness ratio,
AREA = area under PCI curve,
YR = years affected,
WF = weighting factor for usage,
EUAC = equivalent uniform annual cost, and
SY = square yards in management section.

Following this ranking process, the program user may again intervene manually and adjust the construction dates of selected construction and contract packages and require that they be completed in a designated year, regardless of the cost-effectiveness rating. This lets the PMS user intervene and move sections and packages within the ranking to account for conditions not fully accounted for in the ranking procedure. For instance, sections identified as being excessively rough or lacking adequate skid resistance may be designated for repairs in a given year, even if they are not selected by the ranking system. At this point, the sections and packages are reranked. A final listing will then be provided. If the required funding for mandatory sections exceeds the available funding, an error message is provided.

PROJECT-LEVEL CALCULATION OF EFFECTS OF MAINTENANCE AND REHABILITATION

The PCI-versus-age curve must reflect the influence of the maintenance and rehabilitation treatments being analyzed. All maintenance and rehabilitation treatments have two impacts on the PCI-versus-age curve: first, the PCI will be increased; second, the remaining life will increase. Those treatments that return the PCI to 100 are considered rehabilitation; those that improve the PCI to a value less than 100 are considered maintenance.

Currently for rehabilitation, only one curve is available for the Bay Area PMS for flexible overlays and one for reconstruction as asphalt concrete pavements for each surface type and functional classification grouping. Other curves will need to be developed for different thicknesses of overlays and possibly for overlays applied at different condition levels in the future when data are available. In the meantime, the project-level priority-ranking program uses the estimated life extension of the rehabilitation treatment defined by the engineer to account for the expected difference in overlay lives or other treatments.

The projection is adjusted to force it to go through a terminal PCI at the time of the treatment plus the life extension. This is the same concept used to adjust the projected PCI without maintenance or rehabilitation to reflect the difference between the predicted and observed PCI, as described elsewhere (*1,2,8,10*). If the life extension is 18 years, the PCI will be assumed to reach the terminal PCI value at 18 years after application of the overlay plus any remaining life of the existing pavement.

When a treatment is applied to an asphalt pavement that does not replace, rework, or completely cover the surface of the pavement (such as a seal coat, patching, or crack sealing), all distress is not necessarily repaired and the PCI is not increased to 100. Because the PCI is based on distress, the amount of increase in PCI depends on the distresses repaired. The amount that the PCI will increase can be calculated if the distress types being repaired and the effect of the repairs are known (*13*). Although only extrapolated distress is stored in the Bay Area PMS data base, that information can be used to reasonably estimate the PCI increase due to application of the maintenance for use with the analysis concepts, if an adequate percentage of the section area has been inspected.

Not all repair types completely eliminate a distress. For instance, crack sealing can change a medium-severity transverse crack into a low-severity crack, but it does not eliminate the crack. A patch is considered a distress, so patching changes the repaired distress into a low-severity patch. Some distress types cannot be repaired by certain treatments—for instance, a seal coat will not correct rutting. Some distresses can be eliminated by a treatment; low-severity weathering and raveling can be eliminated by a slurry seal. A default set of changes for different treatments is included, but the user has the option of modifying the expected consequence of maintenance treatments on specific distress types and severities.

As an example of how this process is applied, consider the distress information on the asphalt-surfaced pavement shown in Table 1. When the longitudinal and transverse cracking is sealed, full-depth patching is applied to the medium-severity alligator cracking, and a seal coat is applied, the PCI is changed, as shown in Table 2. This procedure is used at the project level to estimate the impact of the increase in PCI due to maintenance. The future PCI is then projected from the expected increased PCI using a curve-shifting procedure (*10*). This basically assumes that the pavement will deteriorate at the same rate that it did when the PCI of the original pavement condition was equal to the PCI after the maintenance. Thus, the pavement deterioration is expected to follow the original curve, but the deterioration begins at the improved condition level. The increase is applied to the curve adjusted for past performance, so the influence of that performance is reflected in the area under the PCI-versus-age curve. As more data on actual field-reported PCI changes become available, the projected deterioration rate will be verified and corrected.

FUTURE IMPROVEMENTS

Some improvements, primarily those that require more data to better define expected performance, where identified in the description of the program. Other improvements address the basic analysis concepts and supporting procedures.

The described procedure requires that the analyst select a treatment for each construction and contract package or allow the program to select one based on a ranking procedure.

TABLE 1 DISTRESS AND DEDUCT VALUES FOR SAMPLE PAVEMENT

ORIGINAL CONDITION			
DENSITY	DISTRESS	SEVERITY	DEDUCTS
2%	alligator cracking	low	16
5%	alligator cracking	medium	39
1%	longitudinal and transverse cracking	low	2
3%	longitudinal and transverse cracking	medium	17
100%	weathering and raveling	low	17
	total deducts		91
	corrected deducts		52
		PCI = 100 - 52 = 48	

TABLE 2 DISTRESS AND DEDUCT VALUES FOR SAMPLE PAVEMENT AFTER CRACKS ARE SEALED, MEDIUM-SEVERITY ALLIGATOR CRACKING IS PATCHED, AND SEAL COAT IS APPLIED

DENSITY	DISTRESS	SEVERITY	DEDUCTS
2%	alligator cracking	low	16
4%	longitudinal and transverse cracking	low	7
5%	patching	low	10
	total deducts		33
	corrected deducts		20
		PCI = 100 - 20 = 80	

The developers have investigated several decision support approaches and have selected an incremental cost-benefit analysis to use when the users become comfortable with the current process.

Most of the project-level evaluation must be conducted outside the program with the aid of decision tables. The use of expert system methodology to assist in this process is being considered for future implementation.

SUMMARY

The Bay Area network-level PMS system allows management sections to be defined and identified, several sorting keys to be established, and the condition to be defined using a distress-based PCI. The future condition can be predicted, network-level budget funding treatments can be assigned to pavement types and condition categories, the total funding needs for the network over a 5-year period can be identified, and different fund-allocation strategies can be tested.

Project-level PMS elements have been developed that complement the network-level system. The project-level programs use the data available in the network-level Bay Area PMS data base, enable the use of more-detailed project-level evaluation data, allow the addition of information from analysis conducted outside the computer programs, and consider nonpavement constraints such as expected utility work. Whereas the network-level analysis uses a 5-year analysis period, the project-level analysis allows maintenance and rehabilitation programs to be developed for periods of 1 to 5 years. Currently, some of the project-level analysis elements developed to assist in treatment selection must be conducted apart from the program software; a manual with decision tables was prepared to assist with these tasks. The project-level programs allow the user to establish contract and construction packages by combining several management sections to form a set that is considered as a unit for the rest of the analysis. The project-level priority-ranking program allows manual intervention to adjust the date of construction when there are constraints not covered in the program. The calculation of the effects of maintenance and rehabilitation uses the same general principles as the network-level analysis (*2*); however, the approach is modified to incorporate the more-complete data used in the development of cost-effective project-level treatments. Funding recommendations are based on a cost-effectiveness analysis that was also used in the network-level analysis.

The interface between network- and project-level PMS elements must be carefully established to avoid duplication of effort and to maintain credibility of both systems. The systems should complement each other and allow information from one system to support the other.

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Performance Prediction Development Using Three Indexes for North Dakota Pavement Management System

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The development aspects of three pavement performance indicators are presented. The prediction curves were developed for North Dakota using the most innovative methods available. The methodology adapted for use in the North Dakota pavement management system (NDPMS) is unique because three performance indicators are used in the system and the performance techniques were also used in seven city and county NDPMS installations. The three performance indicators developed were an overall distress index, a structural index, and a roughness index. NDPMS incorporates the three indexes in the performance prediction module of the analytical software that was developed. Pavement prediction curves were developed for each index. The final curves provide a reasonable method of predicting pavement performance in North Dakota. The development process provided many technological challenges that were channeled into the curve designs.

Performance prediction models are the most essential part of a pavement management system. They are essential to the management of pavements at the network and the project levels, both technically and economically.

Methods for predicting pavement performance should not be selected arbitrarily: they are too important. Mistakes or random selection of methodology for performance prediction can be costly to the highway system. Besides raising cost-allocation questions, poorly designed models make optimal pavement design and selection of optimal rehabilitation strategies and timing of projects impossible. But well-done performance models will secure for their users economy, technical efficiency, and equity. Developing performance prediction curves for North Dakota involved exploring the available methods for predicting pavement performance and selecting or developing the most appropriate one. The performance modeling techniques used for the state system were also used to customize performance models for each of the seven cities and counties in North Dakota that also implemented the pavement management system.

Many factors can influence the performance of pavements and their ability to serve the transportation facility satisfactorily. These include truck traffic, climate, pavement structure, and type of pavement. Another important part of the performance prediction development was the determination of the factors that effect pavement deterioration in North Dakota.

To develop curves that fit the North Dakota performance trends, it was necessary to determine what could be predicted with the most accuracy to also meet the local conditions and constraints on availability of historical inventory. A number of indexes were analyzed for their predictability, including those for structural deterioration, overall distress, climate- and environment-related distress, individual distress, and roughness.

DEVELOPMENT OF PERFORMANCE CURVES FOR NORTH DAKOTA

The first stage in developing performance curves for any pavement management system is that of identifying the pavement-related attributes to predict. At its initial meeting, the North Dakota PMS steering committee discussed to great length which attributes should be used to best model typical pavement performance. The suggestions ranged from using a composite condition index to represent overall condition to predicting individual distress types such as alligator cracking for the selection of rehabilitation strategies. Structural-versus-environmental pavement deterioration was discussed also. The environmental, or nonstructurally induced, deterioration issue arose from a committee discussion about some of the very low commercial volume roadways in the state that carry relatively light loadings and deteriorate primarily because of weather and age. The committee believed that some method of predicting either structural or nonstructural deterioration would be important for selecting appropriate rehabilitation strategies that address the cause of the deterioration. Thus, the individual parts of the state's condition survey forms were divided into structural and nonstructural distresses. The predictability of either group of distresses was discussed by the committee.

The committee decided to test the applicability of using multiple performance curves for the predictions. It was believed that doing so could improve the department's decision-making capabilities by providing more detail to the performance curves.

GROUPING OF HIGHWAY SECTIONS

To develop pavement performance curves for a pavement management system, a method of categorizing pavements must be chosen. Three methods of grouping were investigated.

The first and simplest way is to group pavements that have similar characteristics such as surface type, traffic, and struc-

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ture. This approach assumes that pavements with the same grouping will perform similarly throughout their lives. This method is easy to understand and modify in the future.

The second, and more complex, method is to place all variables in determining pavement condition on the right-hand side of the equation. Each and every pavement section then has its own performance pattern. This technique is an example of a multiple linear regression model. The performance of each pavement is a function of individual items relating to that section. Some of the individual items in the prediction equations could include commercial traffic levels, subgrade strength, maximum surface deflection, and climate. These items tend to become very complicated and usually require complex and comprehensive data.

The third method is also the most complex. It is a combination of the first two methods that groups pavements to represent similar performance patterns, and particular variables are predicted. Because the continuous interaction of variables on each side of the equation makes this approach so complex, it was not thought to be easily modifiable.

After discussing the grouping methods, the PMS steering committee decided to evaluate the effectiveness of the first. The pavement categories were identified by the committee for the initial groupings. These categories included 3 ranges of in situ structural strength, 13 pavement classes (type of cross section), and 4 ranges of traffic [equivalent single axle loads (ESALs)].

At the final meeting of the performance subcommittee, the pavement groups were made final. The subcommittee discussed in detail the difference between the performance of pavements in the asphalt/granular group and in the asphalt/stabilized group. The subcommittee decided that there was no measurable difference between the two groups as indicated by a plot comparison of the two types and recommended they be combined. The final number of groupings agreed upon by the committee resulted in 42 groups of performance classes or cells that had pavement sections in them.

Performance curves were also customized for each of the seven local jurisdictions involved in the implementation. The number of groups of pavement performance at the local level ranged from 5 to 12. This smaller number was due to a less variance in types of pavement cross sections that the cities had historically used in construction.

DATA BASE ANALYSIS AND MANIPULATION

The key ingredient in any pavement performance prediction is the data used in making the prediction. The quality of the historical North Dakota pavement management data base was such that analysis could begin directly without any changes or modifications to the actual raw data numbers. The pavement management data base was composed of 128 different pavement data attributes. The condition assessment method used by the state was a windshield survey recording three levels of severity and three levels of extent for each of the pavement distresses.

The North Dakota data base contained more than 1,000 pavement sections on the state highway system. The first step in the performance curve development was to assign each section a pavement category number. A computer program was developed to assist with categorizing the pavement sections. The program also groups sections according to any subsequent deletions, additions, or changes in category definitions throughout the development process.

A second program was developed to assist in identifying the structural and environmental components of the condition index for each year the condition assessment had been surveyed. The summarized pavement data base maintained by the state stores only 1 year of individual distress elements. In addition, there was no information in the summarized data base combining the individual distress elements into structural or environmental components.

Therefore, it was necessary to build a data file from the 5 years of individual mile-by-mile historical distress information available. A computer program was developed that accessed all the individual mile-by-mile distress data (8,500 mi/year) and built the historical pavement section files for all of the performance curves to be generated.

Quality control of the data was important to maintain the data integrity. Without quality control checks during the many data manipulations, the potential for erroneous performance prediction was imminent. Quality control was provided by manual verification of a sample set of all the data analysis.

SELECTION OF PERFORMANCE PREDICTION METHODOLOGY

Several prediction techniques were evaluated to develop performance curves for North Dakota that reasonably reflect actual deterioration patterns, that can be updated easily, and that can be adapted to local jurisdiction applications. The results of these evaluations follow.

The first method used in developing prediction curves was a linear regression analysis. This resulted in a straight-line least-squares prediction to the first degree of the index over time. The method used a single independent variable in making the prediction.

The first iteration using this procedure identified a significant problem in the state's data storage format. The graphical output of the scatter plots shown in Figure 1 indicated that many data points were stacked at years for which in actuality the age indicated there were only one or two pavement sections. After careful investigation of the data, it was found that there was an error in the methodology in which the data base had originally been set up by the state. As a result, all of a pavement section's current and historical data were plotting as one analysis year.

This required the development of a computer algorithm that back-distributed the historical data to the proper age of the pavement. This process simply involved proper determination of the age of the pavement and the corresponding condition. The state's data file contained only the current age with 5 years of historical distress data. The algorithm also had to take into account any major rehabilitation that had occurred in the past 5 years to develop a proper age-versus-condition distribution.

Subsequent runs using the linear regressions resulted in more complexities. The data plots indicated the trend in performance generally started as a downward trend. These same plots began to show an increase in the amount data scattered

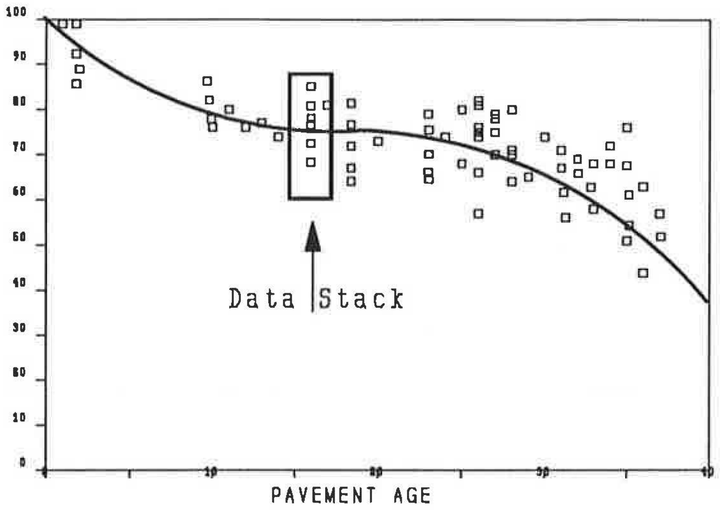

FIGURE 1 Data stacking.

as time progressed. Through critical evaluation, discussions with the state, and extensive reviews of the individual data elements, it was concluded that the maintenance effort was producing a significant effect on the performance curves shown in Figure 2 as a flattening or a rise in the performance.

It became necessary to address this complex issue as part of the pavement management system development. Surface seals were thought to be the major influence from the maintenance effort that was being reflected in the performance curve development. An analysis of maintenance-related performance curves developed by the state for chip seals resulted in the development of another computer program that eliminates records in the analytical pavement section data base that are under the influence of seals according to the seal performance curves (*1*).

Subsequent regression analysis indicated an improvement on the data scatter after the removal of the visual effect of the seals on the surface distress surveys; however, the results of the linear regression analysis were not satisfactory. Many groupings other than those initially identified were tried in an effort to improve the results of the analysis and feel out the data for what might work using other procedures. Modifications to the groupings that were attempted to improve the results included

- Asphalt/granular pavement class and fewer than 100 ESALs,
- Asphalt/granular pavement class and more than 100 ESALs,
- All pavements with an asphalt/granular pavement class,
- All flexible pavements with fewer than 100 ESALs,
- All flexible pavements with more than 100 ESALs, and
- All pavements with an asphalt/stabilized pavement class.

Even these broad groupings produced less-than-satisfactory results. Hundreds of different performance curves were gen-

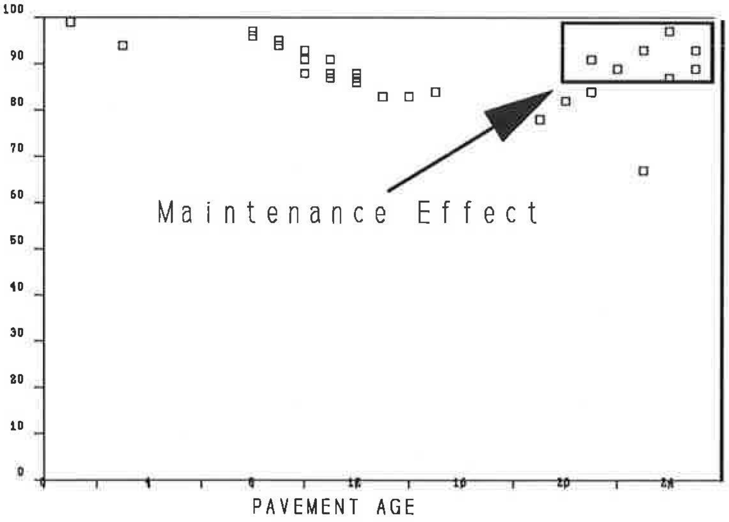

FIGURE 2 Maintenance effect.

erated for all types of condition indexes. The indexes tried were for (a) combined distress, (b) roughness, (c) sum of environmental distress deduct factors, (d) a sum of structural distress deducts, (e) alligator cracking, and (f) combination of factors. The overall output was not satisfactory enough that the prediction models could be used with any confidence. The r^2-values were too low, and the data scatter plots were unacceptable. The analysis did give insight into what combinations of data would work using other analysis techniques.

Several important benefits were gained from this analysis, the two most significant being the elimination of historical data stacking and the reduction of the maintenance effect. These were obstacles related to the data base and to performance, which made them critical to the success of the analysis. They had the potential to affect the predictability of the performance curves dramatically.

AASHTO ROAD TEST PERFORMANCE METHODOLOGY

The concepts and methodology developed in the AASHTO road test pavement performance were comprehensively investigated as to their application to this project. The road test presents a method that predicts serviceability from accumulated axle loadings and describes it as a loss in serviceability as a power function of axle load applications.

This concept was applicable with the data the state has available, but again the lack of data and their complexity at the county and city levels did not make it a desirable method. Many of the same complexities described with the multiple linear regression approach also hold true with the AASHTO road test methods.

EVALUATIONS OF PERFORMANCE MODELS DEVELOPED NATIONWIDE

Pavement performance models already developed from other sources were evaluated for possible application in the development of performance curves for North Dakota. Once again, the complexity of the data required was prohibitive for use by the cities and counties.

NONLINEAR ANALYSIS

The final approach investigated was the development of the performance curves using a nonlinear analysis (*2*). There were several reasons to look at this methodology. The multiple regression analysis indicated that (a) the performance of the pavements was indeed predictable, and (b) the scatter of the data was represented not by a straight line but by more of an S-shaped or a multifunctional line, as shown in Figure 3.

The nonlinear approach has several features that fit well with what the North Dakota system is trying to do. The first is the effect of maintenance. There are many low-volume surfaced roads in which maintenance had a great impact on the pavement performance by flattening out the curve with time as shown on Figure 4. The use of the processing technique called outlier analysis (*2*) allows the user to identify ranges of reasonable data over the life span of the pavement by identifying extreme observations. Another technique that sets boundaries of reasonable data ranges at a given point in time was used in adjusting the original curves only on the basis of historical data with some of the maintenance influence in them. This was an important feature for North Dakota because it allows the use of expert opinion to say what would happen to these pavements if this maintenance had not been performed, representing the Do Nothing condition shown in Figure 4.

Another feature that is applicable is the mathematical models that use either a normalized B-spline approximation or constrained least-squares estimations (*2*). These procedures result in a polynomial equation to the fourth degree. The use of these mathematics allows the multishaped curves to be described. The models are constrained if there is any shift upward on the mathematical curve.

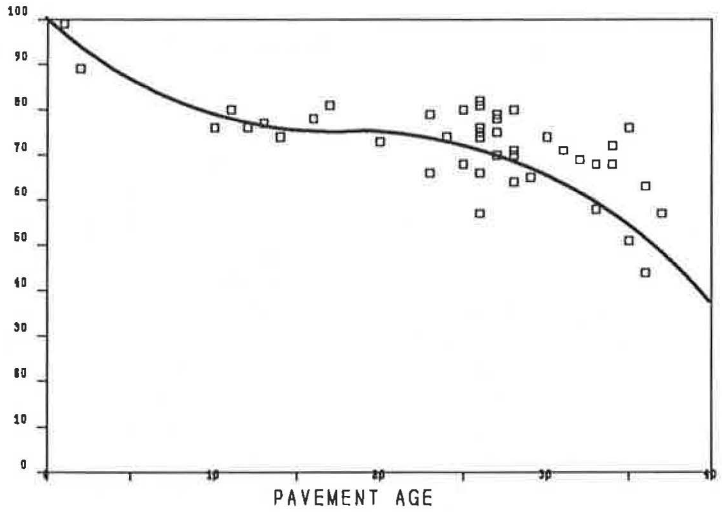

FIGURE 3 S-shaped curves.

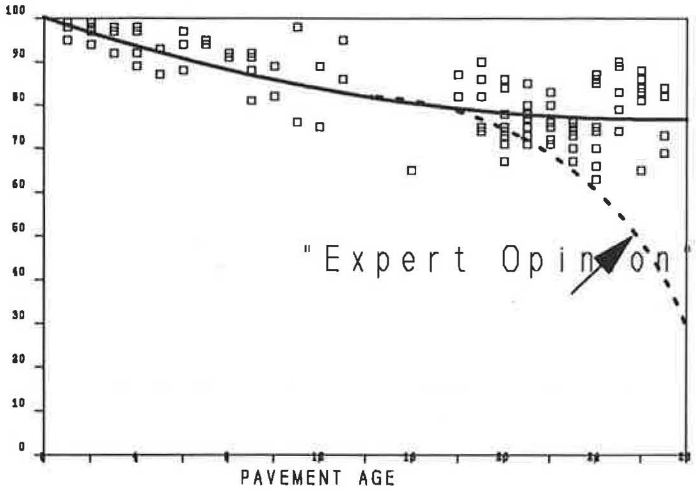

FIGURE 4 Expert augmentation of historical curve.

There are several primary benefits in using the nonlinear analysis, including

- The process is adaptive to incorporation of the expert system approach;
- Annual updates to the performance curves are readily accomplished;
- The software side is simplified as performance curves are easily added, changed, or subtracted; and
- The data-filtering procedure indicates the goodness of fit concerning the pavement category groupings that have been made and indicates whether any grouping changes should be made.

The use of the nonlinear analysis required some additional software manipulations. The use of a 100-to-0 condition index as the common scale for all the indexes required that some of the condition components be converted. A listing of the values represented in the data base that were converted for analysis follows; the asterisk denotes final curve predictions.

Type of Condition Factor	*Range of Values*
Distress index*	99 to 0
Structural distress deduct*	54 maximum deducts
Structural index*	54 to 0
Nonstructural index	45 to 0
Roughness index*	5 to 0
Alligator cracking	18 to 0

For each indicator, the software manipulated the possible values to a common scale indicating a value of 100 for the best condition and 0 for the worst condition. The resulting software converts any of the numeric North Dakota data, regardless of the variations mentioned, into a 100-to-0 scale through a simple algebraic ratio conversion. The software also converts the data from the original format into the proper data input file format for processing by the nonlinear analysis program. This allows the possibility of investigating performance curves for any data element in the historic data file.

The statistical properties of the different curves were evaluated using standard statistical methods. Initial investigations revealed r^2-values ranging from about .6 to 0. The lowest values were for the alligator cracking curves: there tended to be much scatter within the alligator cracking groupings. The values for combined distress and structural index varied from high to low but were within reason for the initial stages of development and indicated that work should proceed with this methodology. From the results of the final performance prediction curves, the distress index and the structural index tended to be the most predictable on the basis of the historic data and are the recommended indexes to be used by the state. The final r^2-values for each of the performance curves are shown in Table 1. The equations developed as a result of this task take the form of the constrained least-squares equation (*3*)

$$P_0 + P_1 * x + P_2 * x * x + P_3 * x * x * x$$
$$+ P_4 * x * x * x * x$$

The complete set of equations, data plots, and statistical checks was delivered to the state (*4–6*).

ROUGHNESS PERFORMANCE PREDICTION

The prediction of roughness proved to be the most difficult to model of all the indexes analyzed. Another complexity in the prediction development was that the state collected historical roughness data with a Mays Ride meter, but all future data would be collected with a profilometer producing the international roughness index (IRI). It was necessary to take this all into account during the development of methodology to predict roughness performance.

The investigation of a reasonable method to predict roughness on the basis of the quality of the existing historical data led to the method described in the following text. It was believed that the resulting method would satisfactorily approximate roughness and lend itself to incorporation of the profilometer data in future years.

The roughness was grouped according to the groupings shown in the following. The split of the flexible pavements by average

TABLE 1 R^2-VALUES FOR FINAL CURVES

Performance Curve #	Distress Index r^2	Structural Index r^2	Performance Curve #	Distress Index r^2	Structural Index r^2
1	0.789	0.731	22	0.940	0.987
2	0.922	0.894	23	0.776	0.769
3	0.778	0.828	24	0.931	0.914
4	0.877	0.720	25	100% expert	100% expert
5	0.482	0.458	26	0.856	0.522
6	0.913	0.845	27	0.649	0.849
7	0.857	0.834	28	0.463	0.401
8	0.13	0.951	29	0.655	0.340
9	0.834	0.938	30	0.872	0.894
10	0.782	0.466	31	0.848	1.00
11	0.860	0.377	32	0.972	1.00
12	0.552	0.303	33	0.902	1.00
13	0.586	0.190	34	0.971	1.00
14	0.799	0.958	35	0.893	1.00
15	0.898	0.456	36	0.945	1.00
16	0.797	0.877	37	0.462	1.00
17	0.450	0.882	38	0.701	0.678
18	0.784	0.903	39	0.719	1.00
19	0.592	0.014	40	0.641	1.00
20	0.999	0.991	41	0.974	1.00
21	0.658	0.832	42	0.850	1.00

daily traffic (ADT) was made in line with the levels set in the decision matrices. R_i is initial roughness; R_t is terminal roughness; R_c is current roughness; $R(p_n)$ is predicted roughness; A_i is age from present forward; A_t is terminal age defined by distress curves; and A_{tn} is terminal age of the corresponding distress index curve n.

Group	R_i	R_t	R^2
Continuously reinforced concrete pavement (CRCP)	3.99	3.1	.12
Rigid jointed	3.59	2.7	.19
Asphalt > 2,000 ADT	3.57	3.1	.11
Asphalt < 2,000 ADT	3.74	2.7	.36

The example in Figure 5 depicts how one of the roughness curves was used to predict roughness for all of their corresponding pavement performance categories by varying the slope of the curve. This approach to the roughness curves produced 4 initial roughness groups and 42 total roughness curves to match each of the pavement performance categories.

The rationale behind the approach to varying the slope of the roughness curve is as follows: take, for example, the flexible roughness curve for less than 2,001 ADT as shown in Figure 6. If only one curve were used, the amount of roughness would be less for shorter terminal age pavements than for longer ones. In other words, the shorter the pavement age, the smoother the pavement will be at its terminal serviceable age. This does not make sense. The method that was developed varies the slope of the curve to match the terminal age of the roughness to the terminal age of the distress. This follows a logical sequence of events in that as the severity and extent of surface distress increase, the pavement becomes rougher. This was also shown by plotting distress and roughness on the same graph in past reports (7).

The final step in the process was determining the amount that the roughness would deteriorate each year if nothing was to be done to improve it. The entire process comes down to a simple algebraic determination of a variable slope line. The equation derivation is shown in Figure 7.

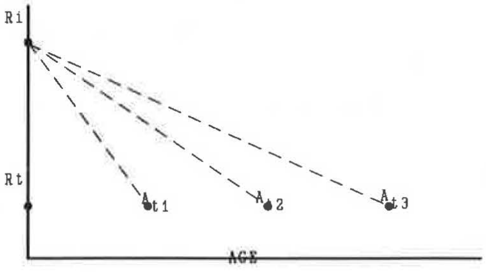

FIGURE 5 Flexible pavements ≤ 2,000 ADT.

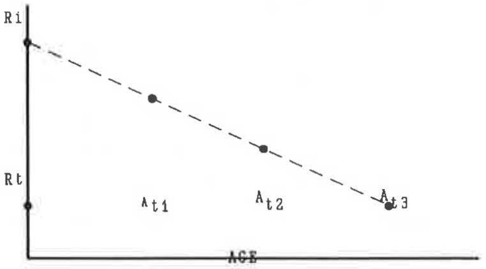

FIGURE 6 One curve.

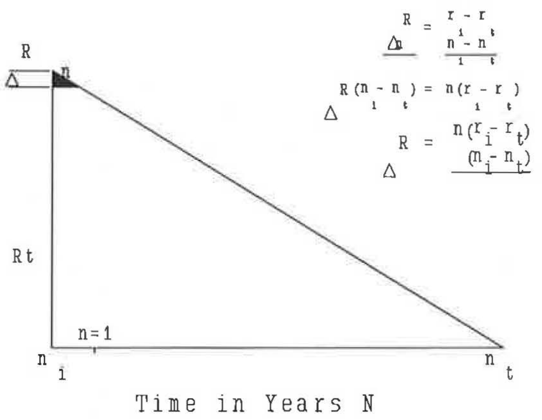

FIGURE 7 Slope equation.

The resulting equation was that for any given roughness (R), the predicted roughness $R(p_n) = R_c - \Delta R * n$. This equation is easily modified in the PMS software: the user can simply change the initial and terminal roughness levels on the straight-line curve through the software.

The four straight-line predictions were recommended considering the quality of the current roughness data available. The quality problems concerned annual comparisons of roughness primarily related to calibration difficulties of the equipment. The Mays Ride data indicated a variance between pavement sections within the year analyzed, but year-to-year comparisons were difficult. These predictions will improve when information from the profilometer becomes available to the department and new performance curves can be generated using the nonlinear approach. The historical Mays Ride meter data were not recommended to be useful in determining future rehabilitation projects based solely on roughness through the pavement management system.

EXTRAPOLATION OF CURVES IN PREDICTION MODELS

The pavement performance prediction models used in developing the overall pavement management system will individually predict the performance of every pavement section in the data base. Individual section predictions are made by using their relative positions to the prediction curves that represent them. This is based on the assumption that the decline in pavement condition is similar on all sections within the performance group represented by the group's performance curve. The future condition of each section is a function of its current condition relative to age. A curve is drawn through the index-age point for the section being predicted parallel to the representative prediction curve.

EXPERT SYSTEM AUGMENTING OF PERFORMANCE CURVES

Pavement management relies on predictions of performance on the basis of some historical information. This is the most critical link in the pavement management system. The predictions must make sense and follow the traditional line expected by the pavement engineer on the basis of past experience. If this basic principle is overlooked or not achieved through the development process, the pavement management system has failed.

Three areas needed the augmentation or concurrence of the expert opinions. The first area is that in which there were not enough historical data points. Several of the performance curves had points along the curves where data were missing or limited. Expert opinion was used to say whether the curve shown was reasonable or not and gave alternative or additional points where they should be. In other cases, performance curves cover too short a life span because there were no historical data to support the expected life performance of a particular rehabilitation alternative. One example of this is the recycled concrete pavement class: the state anticipated a life of 30 years, but there were historical performance data for only 5 years. Expert opinion was used to finish the curve. In another case there were not enough historical data; in other instances there were no data at all. An example of this was the performance of CRCPs that were to be rubblized and overlaid with asphalt. This technique is new to the department, so no historical condition information existed. The expert opinion was used to develop the entire curve until enough historical data are collected.

The second area that needed augmentation concerned the maintenance effect. Several of the performance curves, especially the low-volume roads, slope downward at first but flatten out. The performance curves based solely on the historical data are showing the effect of the maintenance effort. The condition of the highway sections is not allowed to fall below certain levels through maintenance effort. The expert judgment was used to answer the question, What would happen if this level of maintenance were not provided and the highway sections were allowed to deteriorate? The expert provided additional data points and specified ranges in which the performance curve reasonably should be at a certain time to represent the do-nothing condition.

The third area in which expert opinion was used was in augmenting the establishment of the terminal serviceability and life span of the performance curve. Performance prediction curves must have an ending point for the system to operate properly. The expert opinion was used to establish the terminal serviceability and terminal age of the pavements.

The people providing the expert data points were the state's design engineer, district maintenance engineer, district engineer, materials and research engineer, and pavement management coordinator, along with the consultant. The expert rules were to validate historical data, provide insight into do-nothing curves, set ranges of reasonable data, and provide additional points when historical data were unavailable.

The steering committee originally specified that a structural index be developed for each of the pavement groups. The resulting index was on a scale that was converted to be the same as the distress index (100 to 0). It became apparent at the first meeting of the pavement performance subcommittee that it was difficult to understand the converted structural index. The structural index was converted into a structural deduct that was easier to understand, and a subsequent meeting was set up for the expert augmentation of the structural deduct.

The performance subcommittee also determined the terminal serviceability level of 50 on the distress index. They believed that few if any pavements would ever reach this level of distress. They also set the relative age of the pavement when it would reach this level if nothing was done to stop it from deteriorating.

The final step was to regenerate the performance curves. The result was the curves representing the historical data and the expert opinions. The curves as they are shown in their final form (*8*) represent the reasonable expected pavement performance for the department's use in generating pavement management outputs.

BENEFITS AND CONCLUSIONS OF SELECTED PERFORMANCE PREDICTION METHODOLOGY

The development of performance curves for the North Dakota pavement system presented many unique and interesting challenges. The objective was to find the methods and the performance curves that best fit the situation.

The North Dakota system is unique because of its installation locations: besides being used by the state, it was installed at seven city and county jurisdictions. All jurisdictions had to be able to use the performance curves. The local jurisdictions also had gravel-surfaced roads for which performance curves were developed. The local jurisdictions were a challenge to develop reasonable performance prediction.

The amount of historical data also varies dramatically. The state has an excellent 5-year historical pavement management data base that stores a wide variety of data. The information that the cities and counties have varies from some historical distress information to none.

While identifying the most appropriate modeling technique for developing the performance curves, the following criteria were identified as being most important:

- New performance curves can be developed when needed,
- Existing curves can be developed readily,
- Curves can be developed with 1 year of data,
- Expert opinions can be used to set up initial curves for new designs or sections,
- Any number of performance curves are allowed,
- Curves can be modified readily, and
- Performance predictions realistically represent historical performance.

The use of the combined distress index and the structural index or deduct performance curves gave the opportunity to also represent nonstructural deterioration mechanisms. The area between structural deducts and the distress index is the nonstructural deficiency in most instances. This was mainly due to the low number of truck loadings in the state. On the higher-ESAL routes, there was some interaction between the distress types.

The nonlinear analysis approach recommended provides the methodology to satisfy all of the requirements established by the state. This approach also incorporates the expert system technique in a straightforward manner. The expert data points and boundary setting were structured into the outlier processing and the raw data base files.

The software developed and used to generate the curves was structured to allow the addition, modification, or deletion of the mathematical coefficients created by the nonlinear analysis. A change in a performance curve will not require a change in the software code. A sophisticated computer program for developing additional curves, and revising these performance curves, was available for modifications to the performance prediction curves.

The curves apply to both newly constructed as well as rehabilitated pavements. The pavement management system calculates the structural component by use of AASHTO layer coefficients for current and future pavement thicknesses. This makes up the structural component of the grouping. The traffic component is calculated at future dates by the use of an ESAL growth factor. The surface types are determined by the resulting pavement section.

The most critical parts of the pavement management system have been established with the development of flexible performance curves that are easy to understand. The result was the development of a pavement management system for the entire state of North Dakota that was flexible, adaptable, and understandable.

RECOMMENDATIONS

A substantial effort has been expended in developing the best predictive performance curves possible from the available historical data and expert augmentation. The curves will need updating as more historical data become available. Historical data are needed in areas that rely heavily on the expert opinion and also for the roughness.

The roughness is a unique situation because the state is converting to a profilometer from a Mays Ride Trailer. When historical profilometer roughness data for 1 or 2 years are available, the prediction curves should be reevaluated and updated.

An area that the state may wish to consider is the establishment of pavement performance sections to monitor. Representative sections for each of the pavement groups should be monitored closely on a yearly basis. Items to monitor may be a more detailed distress survey, recording of maintenance information and effect, close monitoring of truck traffic, and nondestructive testing.

Pavement performance prediction is the most technologically difficult portion of pavement management and the most influential on the system. It is critical that the integrity of the performance curves be maintained and updated over time.

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Standardization of Pavement Management Systems in Brazil and Other Developing Countries

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The deteriorating condition of paved road networks and the limited resources available for rehabilitating these roads in developing countries underscore the need for more rational approaches to select priority links on a road network. As a result, some developing countries have established pavement management systems (PMSs) to better manage their road infrastructure assets. A main objective of a PMS is to use reliable information and decision criteria in an organized framework to produce a cost-effective pavement program. Pavement management was developed in the United States and Canada and has been widely applied in North America, but there is a tremendous benefit to be gained by applying pavement management technology in developing countries. This has been proven in Brazil, a typical middle-income developing country, and can be applied to great benefit in other developing countries. The PMS implemented in Brazil is described, the special limitations and standardization requirements for the proper use of pavement management in a developing country are discussed. Such PMS must often be done at a technology level below that in the United States. Recommendations are presented for developing countries and for upgrading such technology.

The deteriorating condition of paved road networks and the limited resources available for rehabilitating these roads underscore the need for more rational approaches to select priority links on a road network. As a result, developing and developed countries have been establishing pavement management systems (PMSs) to better manage their road infrastructure assets.

A PMS consists of a comprehensive, coordinated set of activities associated with the planning, design, construction, maintenance, evaluation, and research of pavements. Its main objective is to use reliable information and decision criteria in an organized framework to produce a cost-effective pavement program. PMS activities are directed toward achieving the best value possible for the available funds in providing and operating pavements (*1*).

A PMS must be able to be updated; to consider alternative strategies; to identify the optimum alternative; to base decisions on quantified attributes, criteria, and constraints; and to use feedback information about the consequences of decisions.

Considering the needs of the network as a whole, a PMS can analyze alternative funding programs, making it possible to identify the program that will yield the greatest benefit over the selected analysis period. At the project level, detailed consideration is given to alternative design, construction, maintenance, and rehabilitation activities for a particular section or project within the network so that an optimum strategy can be identified (*2*).

Pavement management was developed in the United States and Canada and has been widely applied in North America, but there is a tremendous benefit to be gained by applying the technology in developing countries. This has been proved in Brazil, a typical middle-income developing country, and can be applied to great benefit in other developing countries.

This paper summarizes the PMS implemented in Brazil for the federal network and discusses the special limitations and requirements for the proper use of pavement management in developing countries. Such PMS must often be done at a technology level somewhat below that in large cities and states in the United States. Recommendations for standardization are presented for developing countries and for upgrading such technology. A brief description of PMS-related studies in Brazil is also presented.

STEPS FOR IMPLEMENTING PMS

Experience suggests that major factors in the successful implementation and improvement of PMS are staging, preimplementation planning, and strong top-management support; the establishment of a PMS steering committee has been useful in many cases (*3*). An important step is that of convincing top management of the value of a PMS. Teach them what a PMS can do and what is required to develop such a system. To this end, a formal 1-week seminar by Hudson and Haas was organized in Brazil in 1983; it included the participation of senior highway managers on the first day. A follow-up seminar was given by Hudson, Haas, and Queiroz at the University of Sao Paulo in 1985. Informal meetings, formal seminars, progress reports, and conference papers are important means of communication between technical staff and managers and across divisions that cooperate in PMS implementation. For Brazil, these activities were also crucial to the dissemination of PMS techniques to state highway authorities.

Figure 1 shows the main stages in developing and implementing a PMS, which were generally followed in Brazil.

Special constraints for implementing PMSs in developing countries include limited trained local human and material resources in several countries, and Brazil is no exception.

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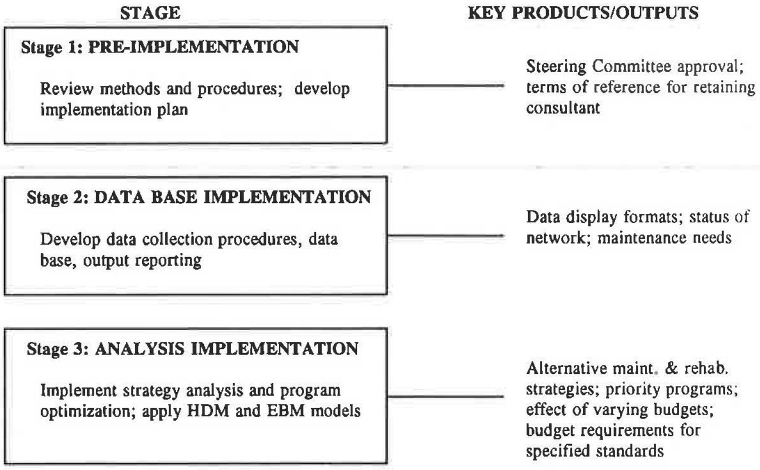

FIGURE 1 Major stages in development and implementation of PMS.

Adopting less-sophisticated methods and equipment for data collection can minimize the material needs. As the amount of modern equipment used to evaluate pavements has grown, so has the concern to choose the most-appropriate devices. An illustrative case is the measurement of pavement deflection, for which the traditional Benkelman beam is of particular interest; not only does it achieve adequate productivity at generally lower costs, but its use can lead to a high degree of accuracy in applying road investment analysis models such as HDM (*4*). Operation of Benkelman beams is labor-intensive, and the instruments are robust.

As for the human resource limitations, foreign consultants were used part time during the first 3 years of PMS implementation (about 1983–86). Consultants have also been useful in PMS improvement and can help to maintain a strong interest by agency managers through periodic visits and seminars. The consultants should work closely with committed local counterparts to insure PMS sustainability.

PMS STRUCTURE

The detailed structure of a PMS depends on the organization of the particular agency within which it is implemented. For Brazil it was considered important to include the following subsystems (*5*):

- *Information subsystem*, which includes data on road length, pavement type, roughness, distress, structural adequacy, traffic, and costs. A simple and realistic procedure for periodically collecting data on the road network that takes advantage of sampling techniques was designed to best fit the needs and capability of the federal roads agency;
- *Maintenance strategy subsystem*, which should be able to simulate total life-cycle conditions and costs for multiple road maintenance (and eventually design) alternatives for road links making up the network. This subsystem should also assist in determining current and future needs (i.e., those sections in the network that have reached or will reach their minimum acceptable or "trigger" level, depending on the criteria specified);
- *Optimization subsystem*, which is necessary whenever the needs exceed the available funds (a common situation in developing countries); and
- *Report subsystem*, which should be able to provide information on the current status of the paved road network, priority programs of rehabilitation, reconstruction and maintenance, and effects of different budget levels on these programs and on the state of the network.

For easier access, the computerized part of a PMS should operate on a personal computer workstation, which can eventually be linked to a mainframe system. A PMS should be flexible in the options provided to the user and in the graphical and tabular reporting functions ranging from detailed to summarized.

PAVEMENT EVALUATION

A systematic approach to pavement management started in Brazil in 1983 under the Brazilian National Highway Department (DNER) for application on the paved federal road network, and several states in Brazil have gradually adopted the developed methodology.

A specific pavement evaluation methodology has been developed as part of the DNER PMS (*5*). For evaluation, the paved road network is divided into homogeneous subsections. The following sequence of procedures is applied to define sample segments, where deflections are measured, and assessment areas, where detailed condition surveys are carried out (Figure 2):

1. Identification of homogeneous subsections within the unitary sections of the National Highways Plan (PNV). These subsections should be between 0.3 and 20 km long. The subsections are selected visually by the resident engineer, without using any equipment. The main factors considered are the type and condition of the surfacing and the homogeneity of traffic.
2. Measurement of roughness on homogeneous subsections. Roughness was adopted as the primary measure of

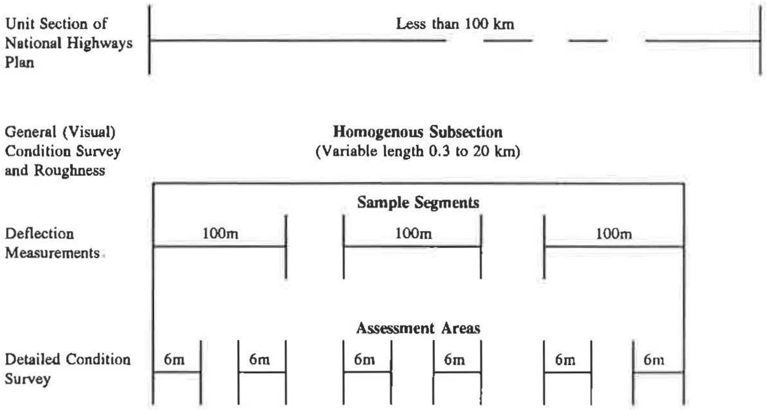

FIGURE 2 Sampling system for network survey.

pavement condition because it relates to safety, the overall economics of road transportation, and rider comfort and performance.

3. Identification in situ of sample segments considered representative of each homogeneous subsection. Three sample segments (each 100 m long) are identified at the beginning, middle, and end of each homogeneous subsection.

4. Measurement of deflection on the sample segments. Pavement deflection is an important parameter for predicting future pavement condition. The Benkelman beam has been adopted to measure deflection in external wheel tracks at 20-m intervals on alternate sides of each sample segment.

5. Survey of pavement condition. Determining the types and extent of pavement defects (such as cracking, potholes, and rutting) is of great importance for planning road maintenance and rehabilitation. Six assessment areas are marked out on each homogeneous subsection, that is, two on each of the extremes of the three 100-m sample segments. Pavement distress found in the assessment areas is duly recorded in both qualitative and quantitative terms.

In summary, under the DNER PMS, pavement evaluation includes a survey of pavement condition and deflection measurements on a sampling basis and of roughness measurements on the whole network.

Resources permitting, these measurements will continue to be taken annually and will be summarized in a format useful to senior management besides being used in economic and technical analyses.

ECONOMIC ANALYSIS OF ALTERNATIVE MAINTENANCE STRATEGIES

PMS implementation requires the use of a valid model to simulate total life-cycle performance and costs for several road maintenance and rehabilitation alternatives for each of the road links composing the network (*4*). For Brazil, the model of choice is the Highway Design and Maintenance Standards Model (HDM-III).

HDM is designed to make comparative cost estimates and economic evaluations of different construction and maintenance options, either for a given road project on a specific alignment or for groups of links on an entire network. A user can search for the alternative with the lowest discounted total cost. If HDM is used in conjunction with the Expenditure Budgeting Model (EBM), the set of design and maintenance options that would minimize total discounted transport costs of an entire road network under year-to-year budget constraints can be determined (*4*). Thus the EBM model assists the analysis team in identifying priority sections and the best maintenance alternative for each priority road section. Microcomputer versions of both HDM and EBM are now available, which makes the models more flexible for general use.

The models used to quantify the relationships between the costs of road construction and maintenance and vehicle operation in HDM resulted from data collected under a collaborative large-scale research program, most of which was carried out in Brazil under a wide range of environmental conditions (*4,6*). HDM relationships are thus directly applicable to Brazil and other tropical regions.

OPTIMAL REHABILITATION PROGRAM

An appropriate methodology for gathering data on the road infrastructure—along with data on the volume, composition, and weight of traffic on each homogeneous subsection—provides the basic information necessary for an economic analysis of alternative strategies to maintain and rehabilitate a road network. This analysis can be applied at the project and network levels using a program such as HDM.

The data necessary for running the HDM model refer to the structure and condition of the existing network, the various sets of maintenance, rehabilitation and reconstruction alternatives, maintenance standards, unit costs, traffic projections, and environmental parameters. Using these data, the model carries out the following series of calculations:

- Vehicle speeds and consumption of resources;
- Road deterioration and maintenance resources;
- Road construction resources;
- User, agency, and total financial and economic costs, calculated on the basis of physical quantities and unit costs; and
- Net present values, internal rates of return and first-year benefits.

Running the HDM-III is divided into the following phases (*4*):

- Data entry and generation of diagnoses;
- Simulation of traffic flows and variations in road condition year by year, taking account of deterioration, maintenance, and possible improvements; and
- Economic comparisons and analyses of alternative construction and maintenance options for selected groups of road links.

A fourth phase is executed by using the EBM model that selects the optimum combination of projects and maintenance alternatives in light of budget constraints.

An example of applying the HDM-EBM methodology is provided here by the economic analysis of alternative programs for rehabilitating the Brazilian federal paved road network, which took into account various annual budget levels for the period from 1986 to 1988. This was the first exercise of this type carried out for the Brazilian federal network, and it is to be followed by subsequent 3-year rolling programs. An analysis period of 12 years was used. The main objective of the analysis was to identify the sections of road that if rehabilitated would result in the maximum overall net present value for each level of investment.

Most of the benefits result from the reduction in vehicle operating costs over each year of the analysis period, a consequence of the improved road condition brought about by rehabilitation or maintenance, or both. The study was relatively conservative in that it did not take account of the reduction in accidents costs caused by safer pavements.

Data for the study were obtained on about 33 000 km of the federal paved network, which resulted in about 3,400 homogeneous subsections, or 10,200 sample segments (where Benkelman beam deflections were measured) and 20,400 assessment areas (where detailed condition survey was performed, including cracking, ravelling, pothole, patching, and rutting measurements).

Running HDM to analyze 3,400 sections would be too costly and time-consuming. To make the analysis manageable, the homogeneous subsections were classified by means of a factorial matrix with 108 cells, each of which represented a set of sections with a relatively narrow range of features. The following factors and respective levels were used to define the matrix (*7*):

- Average daily traffic (four levels): less than 1,000; 1,000 to 3,000; 3,000 to 5,000; and more than 5,000 vpd;
- Benkelman deflection (three levels): less than 0.5; 0.5 to 0.8; and more than 0.8 mm;
- Percentage of pavement area cracked: less than 20, 20 to 40, and more than 40 percent; and
- Roughness, in terms of the quarter-car index (QI): less than 40, 40 to 60, and more than 60 counts/km.

Ten sets of rehabilitation and maintenance alternatives were defined for possible application to each road section (or groups of road sections in a cell of the factorial matrix) during the analysis period, as given in Table 1.

The HDM model was then run for each cell of the factorial matrix, using as input data the weighted average of pavement condition and traffic applicable to each cell. The optimum maintenance and rehabilitation alternative selected from those shown in Table 1 was then identified for each group of road sections in a cell, that is the alternative that would maximize the net present value (*7*). Ideally, it would be desirable to implement physically the optimum option for all of the road sections on the network. However, it was found that the

TABLE 1 PAVEMENT REHABILITATION OPTIONS

Option	HDM Code	Maintenance/Rehab Alternative	Minimum Useful Life (years)	Conditions of Application	
				Roughness, QI (counts/km)	Cracking (%)
0	12	Routine maintenance	-	-	-
1	08	Deep Patching	-	-	-
	12	Routine maintenance	-	-	-
2	09	Slurry seal	3	-	30
	12	Routine maintenance	-	-	-
3	09	Double surface dressing	5	-	30
	12	Routine maintenance	-	-	-
4	10	Overlay (4 cm AC)	5	50	-
	12	Routine maintenance	-	-	-
5	10	Overlay (8 cm AC)	5	40	-
	12	Routine maintenance	-	-	-
6	10	Overlay (8 cm AC)	5	60	-
	12	Routine maintenance	-	-	-
7	10	Overlay (12 cm AC)	5	40	-
	12	Routine maintenance	-	-	-
8	10	Overlay (12 cm AC)	5	60	-
	12	Routine maintenance	-	-	-
9	11	Reconstruction (15 crushed stone + 5 AC)	10	80	-
	12	Routine maintenance	-	-	-

funds required were well over the available or plausible budget levels.

Using as input the HDM output for each cell, as well as the most plausible budget levels, an improved version of the EBM model was run to select priority cells to be rehabilitated. The criterion adopted was that of maximizing the overall net present value. The priority sections thus identified served as the basis for a major road rehabilitation program in 1986–88. Actual rehabilitation design for each priority road section was done by the application of an optimal design method (*8*). The improved EBM model, developed at the Brazilian Road Research Institute, excludes any restriction with regard to number of projects or of budgetary periods yet provides the same results as EBM when run with data within EBM limitations (*9*).

ACCEPTABILITY INDEX TO PRIORITY RANK REHABILITATION SECTIONS

The HDM-EBM analysis of a road network, as described, may become too costly and time-consuming for large road networks. To circumvent this problem, it was deemed worthwhile to develop an acceptability index (AI) that could be computed directly from the field parameters characterizing the homogeneous subsections of a network and yet enable rehabilitation priorities to be assigned to the subsections much more rapidly and simply than by means of a detailed economic analysis (*10*).

The main purpose of calculating AI is to classify the various sections of a road network in terms of rehabilitation priorities. The higher the value of AI, the greater the acceptability of the section and, therefore, the smaller the need for rehabilitation. The AI to be investigated would be allowed to range from 0 to 100. A section with an AI of 0 would have no acceptability and would therefore be assigned maximum priority, whereas a section with an AI of 100 would receive zero priority. The AI cannot replace the economic analysis, but it is helpful as a means to first screen the road network and select a subset of road sections in most need of rehabilitation. The HDM-EBM analysis would then be carried out on this subset and be of a much more manageable size.

The AI calculation algorithm was developed at the Brazilian Road Research Institute (*11*). Two sets of data are necessary for calculating the AI. The first concerns the road section itself, and the second is based on the averages and standard deviations for certain features of the road network. The values relating to the individual road section are as follows:

- Roughness, in terms of the quarter-car index in counts per kilometer (QI);
- Percentage of paved area affected by cracks rated as Class 2 or worse, plus patching and potholes (CR);
- Average Benkelman beam deflection in 0.01 mm (BD);
- Average daily volume of cars and light trucks in vehicles per day (CA);
- Average daily volume of buses in vehicles per day (BU);
- Average daily volume of trucks (medium, heavy, and semitrailers) in vehicles per day (TR);
- Annual rainfall in millimeters per year (RA); and
- Indicator denoting topographic relief, that is, 1 = flatland, 2 = hilly, 3 = mountainous (RE).

The values relating to the road network are expressed as the averages and standard deviations of these eight variables. As an example, a set of values for these parameters, calculated from the 1985 survey of the Brazilian federal paved road network, is given in Table 2 (*10*). Calculating AI for a road section requires the following steps:

1. Calculation of the reduced value—$R(x)$—of each of the parameters above:

$$R(x) = (x - \text{average})/\text{deviation}$$

where

$R(x)$ = reduced value,
x = parameter value for the section,
average = average of parameter (x) for the network, and
deviation = standard deviation of parameter (x) for the network.

2. Calculation of the standardized value—$S(x)$—of each of the parameters above: $S(x)$ is obtained as the area under the normal curve corresponding to $R(x)$. For example, if $R(\text{QI}) = 2.86$, then $S(\text{QI}) = 0.9979$. If the standardized value is negative, its absolute value should be used.

3. AI can then be computed by

$$\text{AI} = 100 - 0.03993 \times (\text{FQI} \times \text{FTR} \times \text{FST})^{2.278}$$

TABLE 2 SUMMARY CHARACTERISTICS OF BRAZILIAN FEDERAL PAVED ROADS

Item No.	Variable	Average	Std. Deviation
1	Roughness, QI (counts/km)	49.94	13.86
2	Cracking (%)	18.44	26.34
3	Deflection (mm)	57.38	37.99
4	Average daily traffic: cars	1,641.06	3,973.21
5	Average daily traffic: buses	279.08	1,003.48
6	Average daily traffic: trucks	1,391.29	2,271.21
7	Rainfall (mm/yr)	1,299.45	548.08
8	Relief	1.90	0.59

where

$$\begin{aligned} \mathrm{FQI} &= [10 \times S(\mathrm{QI})]^{0.0633}; \\ \mathrm{FTR} &= [10 \times S(\mathrm{CA})]^{0.141} + [10 \times S(\mathrm{BU})]^{0.164} \\ &\quad + [10 \times S(\mathrm{TR})]^{0.514}; \text{ and} \\ \mathrm{FST} &= [10 \times S(\mathrm{CR})]^{0.038} + [10 \times S(\mathrm{BD})]^{0.083} \\ &\quad + [10 \times S(\mathrm{RA})]^{0.024}\, \mathrm{n} + [10 \times S(\mathrm{RE})]^{0.014}. \end{aligned}$$

It is pertinent to note that the equation given to compute the acceptability index was developed through regression analysis with the results of the HDM-EBM analysis of the federal paved network (*11*). The objective was to obtain an AI that would approximate the priorities given by the economic analysis. The criterion chosen was that the AI should be correlated with the unit net present value (NPV/km) resulting from the optimum rehabilitation alternative of a road section (*10*). In fact, a comparison between priorities assigned by the HDM-EBM analysis and those computed by the AI showed very good agreement.

Unit net present value (NPV/km) and the acceptability index are well correlated (coefficient of determination of 0.96) by Coelho (*11*):

$$\mathrm{NPV/km} = 1564/(\mathrm{AI}^2 + 1) + 164 \exp(-0.098\ \mathrm{AI})$$

The AI is a simple ranking approach that estimates the relative rehabilitation need (and economic return) of road sections making up a network. It is a method that can be applied for large road networks when the number of sections is high and the HDM-EBM optimization method may not be practical. The AI not only has a direct connection with the optimization technique but also can be linked with the utility concept. In a road facility management system, utility is the level of overall effectiveness that can be achieved by undertaking a project (*12*). A road section with an AI of 0 has the highest priority to be rehabilitated, and therefore the utility value of rehabilitating this section should be maximum, that is, equal to 100. Conversely, if a section's AI is 100, this section is totally acceptable and does not require any rehabilitation; the utility value of rehabilitating it should be minimum, that is, zero. Therefore, the acceptability index and the utility value (UV) can be linked by

$$\mathrm{UV} = 100 - \mathrm{AI}$$

ROUGHNESS SCALE

Roughness plays an important role in pavement management because it is the most important measure of road condition influencing vehicle operating costs and because it affects the safety, comfort, and speed of travel. Roughness was expressed in this paper in terms of QI in counts per kilometer (*13*). However, roughness measurements are now generally expressed in terms of the international roughness index (IRI) (*14*). IRI is mathematically defined from relatively true profile to simulate the vertical motions induced in a moving quarter-car (i.e., one wheel, suspension, and sprung mass).

The basic concepts of QI and IRI come from simulation of vertical motion on a road profile. QI was the roughness standardized in the Brazil/UNDP study (*6*), which provided most of the data originating the HDM model. The relationship between QI (in counts per kilometer) and IRI (in meters per kilometer) scales is (*15*)

$$\mathrm{QI} = 13\ \mathrm{IRI}$$

Roughness measurements for the Brazilian PMS are obtained using a vehicle instrumented to produce a numeric proportional to the vehicle response to the road traversed. Two such systems, which are called response-type road roughness measuring systems (RTRRMSs), have been used in Brazil: Maysmeters and Linear Displacement Integrator (*6,16*). The roughness numeric obtained from a RTRRMS for a road section depends on the test speed, type and condition of host vehicle, and other factors. Therefore, any RTRRMS must be periodically calibrated to produce QI or IRI.

All RTRRMSs used in Brazil are calibrated by correlation, which is performed on control road sections. The QI (or IRI) for each control section is obtained by a rod-and-level survey (*17*). The RTRRMS to be calibrated measures 10 or more of their control sections, and the results are used with QI to determine a regression equation that is used to convert RTRRMS measurements into QI. As a result of this calibration, roughness measurements under the Brazilian PMS are stable over time and can be compared with careful measurements taken in any other region of the country or the world.

RECOMMENDATIONS FOR OTHER DEVELOPING COUNTRIES

Although developed for Brazil, the pavement management technology described herein can be applied directly, or with slight adjustments, to other developing countries with large road networks. For countries with smaller road networks (up to about 50 links or homogeneous subsections), it is recommended that the HDM-EBM optimization analysis be applied directly to the road links composing the network, without the need for grouping links into cells of a factorial matrix. Future improvements to the HDM-EBM software should make it feasible to apply the optimization technique directly to larger networks.

The periodic computation and display of acceptability indexes (or utility values) for a road network provides a simple means for senior management, and eventually the public user, to monitor performance of the networks they manage or use. It is recommended that an acceptability index similar to the one shown here be computed and used in developing countries in the future.

CONCLUSIONS

The successful development, implementation, improvement, and sustainability of a PMS in Brazil appeared to result from careful preimplementation planning, strong support by senior management, a sound data base, use of adequate models, and a commitment by those responsible for its operation. External resources, in terms of specific financing and consultant expertise, have played a key role in all phases of PMS development and implementation.

This paper described the main stages and activities in the development of a PMS for use in Brazil, including condition surveys, roughness measurements, a data base for pavement-related information, an analysis scheme (including the use of HDM and EBM models and an acceptability index), and implementation procedures.

The acceptability index presented is a simple ranking approach that provides the relative rehabilitation need (and economic return) of road sections comprising a network. It is a method that can be applied for large road networks for which the number of sections is high and the HDM-EBM optimization method may not be practical. The AI not only has a direct connection with the optimization technique used but also can be linked with utility concepts.

Although developed for Brazil, the pavement management technology described in this paper can be applied directly, or with slight adjustments, to other developing countries. Certainly the concepts are applicable everywhere.

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Distress Prediction Models for a Network-Level Pavement Management System

CHHOTE L. SARAF AND KAMRAN MAJIDZADEH

Distress prediction models play an important role in a pavement management system (PMS). These models are used to predict the condition of pavements treated with given maintenance and rehabilitation (M&R) action. They can also be used to compare the economics of various maintenance alternatives. The development of distress prediction models for a network-level PMS recently developed for the Ohio Department of Transportation is described. Five M&R actions or maintenance alternatives were included. Visual condition surveys of overlaid pavements (composite) currently include 14 distresses. These distresses were grouped into four distress groups each having its own severity and extent. Thus, 8 equations were developed for each M&R action, resulting in 40 equations for all five M&R actions and four distress groups. The models were used to predict distresses and pavement condition rating (PCR), which were compared with the corresponding distresses and PCR calculated from field observations. These comparisons indicated that the models were capable of predicting with reasonable accuracy the condition of a highway network as well as an individual pavement segment. Limited data for 5 years were available at the time of analysis; this should be kept in mind while the models are extrapolated for traffic loadings beyond these limits.

Highway engineers use a pavement management system (PMS) to develop information that can be applied to make cost-effective decisions about the type of maintenance needed as well as the place and time it is to be performed (*1*). Distress prediction models play an important role in this process. These models help in predicting performance of pavements treated with given maintenance and rehabilitation (M&R) actions. The effects of alternative M&R actions on the future performance of a pavement can also be assessed with the help of these models. Then engineers can use the information to select appropriate M&R strategies that satisfy their budget and performance constraints.

Pavement distress data collected from field surveys are generally used to develop distress prediction models. Visual and objective measurements have been used to collect pavement condition data from field surveys. The field data used in this study were collected by the Ohio Department of Transportation (ODOT), which employed a visual method called pavement condition rating that is described in the ODOT manual (*2*). The data collected from these surveys have been recorded annually since 1985 for all Interstate and other divided highways in Ohio. Therefore, 5 years of data were available when analysis was performed for this study.

The distress prediction models described in this paper were developed for the third-generation PMS (PMS-III) recently developed for ODOT. This is a network-level system that provides optimal maintenance strategies for the divided highway network of the state of Ohio. The system also includes features that allow users to estimate long-range budget allocations for a 6-year planning period and present assessment and forecasting of the network conditions and rehabilitation needs for the planning period (*3,4*).

This paper briefly describes the procedure to develop distress prediction models of composite pavements after jointed rigid pavements (JCPs) were overlaid with asphalt concrete overlays. Although these models were developed for the analysis of pavements at the network level, the estimated performance was also compared with the field observations of individual pavement's condition. This comparison indicated that the models can be used to predict with reasonable accuracy the distresses of individual pavements as well as groups of pavements.

DISTRESSES DEFINED FOR STUDY

ODOT engineers have defined 14 distresses for a composite pavement. A list of these distresses is presented in Table 1. All of these distresses are recorded visually by properly trained surveyors. The *ODOT Pavement Condition Rating Manual* (*2*) contains descriptions and photographs of each distress and its levels. Observations of the severity and extent of each distress are recorded in data forms as letter and number codes. These codes are then converted into numerical values with the help of distress weights and severity and extent weights as listed in Table 1. The following relationship is used for this purpose:

$$\text{deduct points} = \text{distress weight} \times \text{severity weight} \times \text{extent weight} \quad (1)$$

where deduct points represent the amount of damage caused by the distress present at the time of observation. The relative weights assigned to each distress and up to 100 when all 14 distresses are present in the pavement at their highest severity and extent levels. Therefore, the total deduct points vary from 0 to 100: 0 represents no visual damage and 100 represents total damage. The pavement condition rating (PCR) of a pavement is calculated from deduct points as follows:

$$\text{PCR} = 100 - \text{sum of deduct points of all visible distresses in the pavement} \quad (2)$$

The scale of PCR values is also from 100 to 0. A PCR of 100 represents a pavement with no visible damage, and a PCR

Resource International, Inc., 281 Enterprise Drive, Columbus, Ohio 43081.

TABLE 1 COMPOSITE PAVEMENT DISTRESSES AND THEIR WEIGHTS, INCLUDING SEVERITY AND EXTENT WEIGHTS

Distress	Distress Weight (di)	Severity Weight* (si)			Extent Weight** (ei)		
		L	M	H	O	F	E
Raveling	10.0	.3	.5	1.0	.3	.5	1.0
Bleeding	7.0	.4	.49	1.0	.4	.49	1.0
Patching	7.0	.3	.6	1.0	.3	.6	1.0
Surface Disintegration or Debonding	7.0	.38	.65	1.0	.38	.65	1.0
Rutting	8.0	.41	.68	1.0	.41	.68	1.0
Pumping	9.0	.3	.6	1.0	.3	.6	1.0
Shattered Slab	8.0	.3	.65	1.0	.3	.65	1.0
Settlement	5.0	.38	.74	1.0	.38	.74	1.0
Transverse Cracking							
Unjointed Base	20.0	.2	.6	1.0	.2	.6	1.0
Jointed Base							
(1) Joint Reflection Cracks	12.0	.2	.6	1.0	.2	.6	1.0
(2) Other	8.0	.2	.6	1.0	.2	.6	1.0
Longitudinal Cracking	7.5	.27	.51	1.0	.27	.51	1.0
Pressure Damage/Upheaval	6.0	.3	.67	1.0	.3	.67	1.0
Crack Sealing Deficiency	5.0	1.0	1.0	1.0	.6	.72	1.0

* L = LOW
M = MEDIUM
H = HIGH

** O = OCCASIONAL
F = FREQUENT
E = EXTENSIVE

of 0 represents a pavement with total damage. Although some of these original definitions of deduct points and PCR were retained, it was necessary to redefine the distresses of these pavements so that 14 distresses used for rating could be reduced. Thus, they were divided into four groups, as shown in Table 2, for which prediction models were developed. This reduced the number of equations from 28 (14 equations for severity and 14 for extent) to 8 for each M&R action.

A method was developed to estimate the severity and extent of each distress group. This method is summarized by the following relationships (see Table 1):

$$Ds = \left(\sum_{i=1}^{n} di\ si\right)/GW \tag{3}$$

$$De = \left(\sum_{i=1}^{n} di\ ei\right)/GW \tag{4}$$

TABLE 2 DISTRESS GROUPS CREATED FOR STUDY, WITH THEIR COMPONENT DISTRESSES AND GROUP WEIGHTS

Distress Groups	Weight (GW)	Component Distresses (See Table 1)
1. Structural 1	30	Raveling Bleeding Longitudinal Cracking Crack Sealing Deficiency
2. Structural 2	20	Rutting Surface Disintegration or Debonding Settlement
3. Joint 1	30	Pressure Damage/Upheaval Patching Pumping Shattered Slab
4. Joint 2	20	Unjointed Base Transverse Cracking or Jointed Base Joint Reflection Cracks Other Reflection Cracks

where

Ds = distress group severity,
De = distress group extent,
di = weight of ith component distress,
si = severity weight of ith component distress,
ei = extent weight of ith component distress, and
GW = total weight of distress group (see Table 2).

The following example illustrates the use of Equations 3 and 4 to calculate the severity and extent of each distress group from the field observations of severity and extent of component distresses.

The following distresses were recorded during the field survey of a hypothetical pavement:

Distress	*Severity*	*Extent*
Longitudinal cracking	*L*	*F*
Crack sealing deficiency	*L*	*O*

Because both distresses belong to Distress Group 1, the severity and extent of Distress Group 1 is calculated from Equations 3 and 4 using the numerical values associated with each observation (see Table 1 for the numerical values of *L*, *F*, and *O* of respective distresses and Table 2 for *GW*:

$$\text{severity} - Ds = (0 + 0 + 0.27 \times 7.5 + 1.0 \times 5)/30$$
$$= 0.234$$

$$\text{extent} - De = (0 + 0 + 0.51 \times 7.5 + 0.56 \times 5)/30$$
$$= 0.221$$

GENERAL FORM OF DISTRESS PREDICTION MODEL

It can be hypothesized that a pavement treatment with a major maintenance action (such as an overlay) will develop distresses at an increasing rate with increasing time or traffic, as illustrated in Figure 1. However, the field observations indicate that the distresses develop in steps (Figure 1). This is because measurements of component distresses are discrete (zero, low, medium, etc.). Therefore, it will be more realistic to use a step function form for a distress prediction model. But because of the complex nature of step functions, a continuous function form was considered suitable for this study, with a modification to the last part of the curve so that maximum value of distress will not exceed unity (see Equations 3 and 4). This defined the general shape of the distress function as a curve with a slow rate of growth in the initial part, an increasing rate in the middle, and a flat part (or almost constant value) at the end. This may be called a S-shaped curve also. An equation of the following form was assumed to represent this shape:

$$D = e^{-(A/T)} \qquad (5)$$

where

D = distress in the pavement (severity or extent),
T = time or traffic at which distress D is observed, and
A = parameter that represents the pavement characteristics.

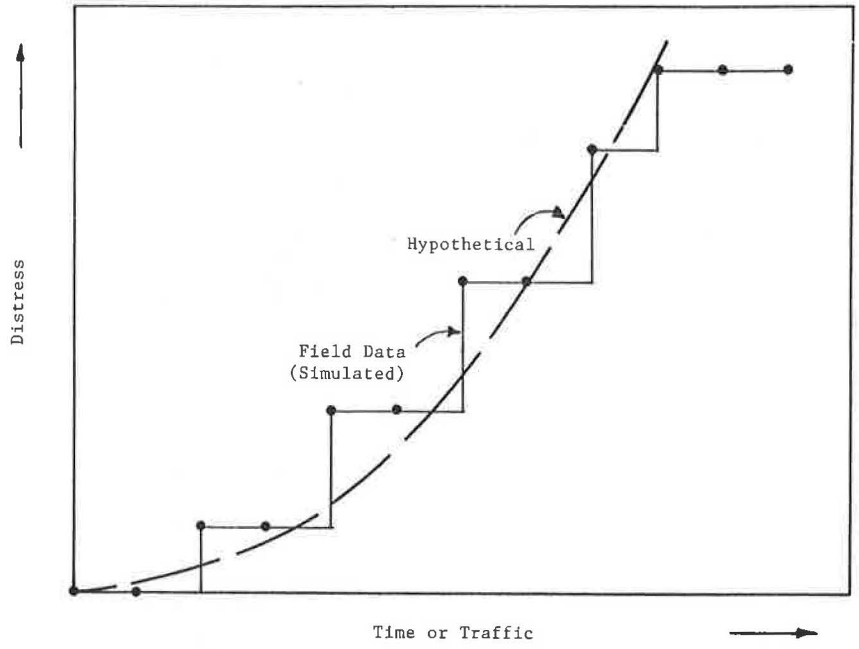

FIGURE 1 Plots of hypothetical distress and simulated field distress data.

Typical curves for three different values of Parameter A are shown in Figure 2. These curves indicate that the general form of the model represented by Equation 5 will satisfy the expected requirements of the distress function described.

Several parameters affect the development of distresses in pavements. Pavement type, M&R action, layer thicknesses and their strengths, properties of subgrade soil, and environment are some of the parameters considered important in this case, and their inclusion in the model was investigated. The results of these investigations indicated that the effects of layer thicknesses and their moduli of elasticity can be combined into a single parameter, H, defined as follows:

$$H = 0.1\left[\sum_{m=1}^{n} Hm\ (Em/20{,}000)^{1/3}\right] \quad (6)$$

where

H = pavement layer parameter,
Em = modulus of elasticity of mth layer of pavement,
Hm = thickness of mth layer of pavement, and
n = number of layers in the pavement above subgrade.

Similarly, the subgrade characteristics were related to pavement performance via a parameter, I, defined as follows:

$$I = Es/1{,}000 \quad (7)$$

where I is the subgrade parameter and Es is the modulus of elasticity of subgrade material.

Because it was decided to develop separate distress prediction models for each M&R action, and each distress group severity and extent, it was not necessary to include these variables in the equation. The effect of environmental factors was not included at this time either because AASHTO Regional Factor does not vary significantly over Ohio. Therefore, Constant A was related to other parameters as follows:

$$A = a1\ H^{a2}\ I^{a3} \quad (8)$$

where $a1$, $a2$, and $a3$ were assumed to be constants to be determined from the regression analysis of appropriate data, as will be explained later. Thus, Equation 5 was rewritten as follows:

$$D = \exp[-(a1\ H^{a2}\ I^{a3})/(T + 1)^{a4}] \quad (9)$$

where T was assumed to represent cumulative traffic since the time of last major maintenance. The value of T was estimated in terms of millions of 18-kip equivalent single axle loads (or E-18). A constant of 1 was added to avoid any numerical inconsistency when T is equal to zero.

DATA COLLECTION

The following data were collected for this study:

- Thickness and modulus of elasticity of pavement layers,
- Subgrade strength,
- Traffic data, and
- PCR data.

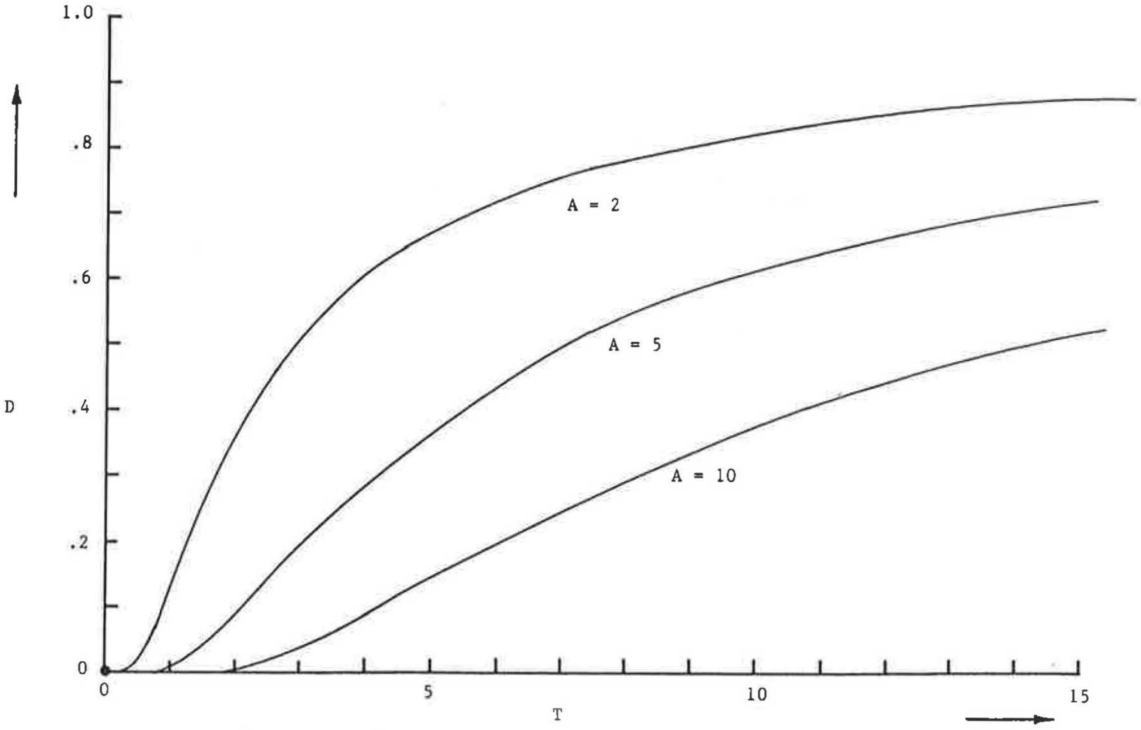

FIGURE 2 Typical plots of assumed distress function (Equation 5).

Most of the data were available in the computer files maintained by ODOT. Thickness of original pavement and the types of layer materials were recorded in the design file. This file also contained the subgrade soil classification (according to ODOT classification system). Maintenance data files contained information related to past maintenance performed on the pavement including the thickness and material types of overlays. Therefore, the data required to calculate *H*- and *I*-parameters of Equations 6 and 7 respectively were obtained from these two files. Because only material codes were recorded in these data files, a representative modulus of elasticity was used for each material code. The information for this purpose was obtained from ODOT engineers. Table 3 summarizes the strength properties of ODOT's pavement materials. The soil classification obtained from design files was converted into modulus of elasticity as indicated in Table 4.

Traffic data used in this study were obtained from the road inventory files maintained by ODOT. These files contained traffic count summaries and the years of these surveys. Total traffic and percentage of B- and C-trucks (ODOT's designation of heavy vehicles) are recorded in these files for all roads in Ohio. These data were used to estimate the parameter *T* used in Equation 9.

Pavement condition rating (or PCR) surveys conducted annually on all Interstate and other divided highways of Ohio are recorded in the PCR files of ODOT. PCR records for 1985–1989 were available when analysis was performed for this study. Therefore, the same were used to calculate the distresses in the pavements.

TABLE 4 STRENGTH PROPERTIES OF ODOT'S SOILS

Soil Type	CBR[1]	Es psi[2]
A-1, A-2, A-3	11.0	13,200
A-4	8.4	10,100
A-5	7.3	8,800
A-6	6.5	7,800
A-7	5.2	6,200

[1] CBR values were obtained from the ODOT Design Manual Chart.

[2] Modulus of elasticity, Es was determined by the following relationship:

$$Es = 1,200 * CBR$$

The values of Es were rounded to the nearest 100 psi.

DATA ANALYSIS

Computer programs were developed to extract and process the data contained in each file mentioned. The data from

TABLE 3 STRENGTH PROPERTIES OF ODOT'S PAVEMENT MATERIALS

Code	Description	E, psi
	Up through 1965	
T-71	Reinforced Portland Cement Conc. Pavement	4,500,000
I-22	Subbase	20,000
T-33	Bitum. Macadam Surface Course, Type A or B	100,000
I-18	Stabilized Crushed Aggregate Shoulder	50,000
B-21	Waterproofed Aggregate Base Course	100,000
T-31	Asph. Conc. Surface Treatment Using No. 6 Aggregate and Bituminous Materials	450,000
B-35	Asph. Conc. Leveling Course or Base Course	300,000
I-19	Insulation Course, Water, Gran., Blast	100,000
T-35	Asphaltic Concrete Surface Course	450,000
B-70	Portland Cement Concrete Base Course	4,500,000
B-33	Bituminous Aggregate Base	200,000
B-219	Waterproofed Aggregate Base Course	100,000
I-7	Reinforced Concrete Pavement for Ramps	4,500,000
B-20	Waterbound Macadam Base Course Using No. 2 Stone	100,000
	1966 - present	
451	Reinforced Portland Cement Conc. Pav.	4,500,000
404	Asphalt Concrete	450,000
402	Asphalt Concrete Leveling Course	450,000
403	Asphalt Concrete Preleveling Course	450,000
301	Bituminous Aggregate Base	450,000
310	Subbase	20,000
409	Seal Coat Cover Aggregate Using No. 8 Aggregate and Bituminous Material	200,000
305	Portland Cement Concrete Base	4,500,000
304	Aggregate Base	50,000
453	Continuously Reinforced Portland Cement Concrete Pavement	4,500,000
848	Asphalt Concrete	450,000
804	Cement Stabilized Base or Subbase	3,500,000
302	Asphalt Concrete	450,000
453	Continuously Reinforced Portland Cement Concrete Pavement	4,500,000
801	Portland Cement Concrete Base	4,500,000

TABLE 5 PARTIAL LISTING OF DATA PREPROCESSED FOR DEVELOPING DISTRESS PREDICTION MODELS

District	County	Route	Beginning Mile Post	Ending Mile Post	Pvt. Type	M&R Action	Traffic Param. T in E-18 (millions)	H	I	Distress Group #1		Distress Group #2		Distress Group #3		Distress Group #4	
										Ds	De	Ds	De	Ds	De	Ds	De
01	ALL	030	0	2.7	4	050	0.442552	7.909	6.200	0.20	0.43	0.00	0.00	0.00	0.00	0.15	0.25
01	ALL	030	0	2.7	4	050	0.907228	7.909	6.200	0.22	0.43	0.00	0.00	0.00	0.00	0.25	0.29
01	ALL	030	2.7	12.7	4	070	0.351849	7.062	6.200	0.10	0.33	0.00	0.00	0.00	0.00	0.12	0.60
01	ALL	030	2.7	12.7	4	070	0.721287	7.062	6.200	0.27	0.50	0.00	0.00	0.00	0.00	0.36	0.60
01	ALL	030	2.7	12.7	4	070	1.109193	7.062	6.200	0.33	0.57	0.08	0.08	0.00	0.00	0.36	0.60
01	ALL	030	2.7	12.7	4	070	1.516500	7.062	6.200	0.39	0.57	0.06	0.10	0.00	0.00	0.60	0.68
01	ALL	075	0	9.2	4	050	2.235982	7.627	6.200	0.25	0.49	0.08	0.14	0.00	0.00	0.36	0.40
01	ALL	075	0	9.2	4	050	3.438490	7.627	6.200	0.48	0.59	0.13	0.24	0.00	0.00	0.28	0.46
01	ALL	075	0	9.2	4	050	4.701141	7.627	6.200	0.32	0.51	0.16	0.34	0.00	0.00	0.49	0.55
01	ALL	075	0	9.2	4	050	6.026914	7.627	6.200	0.37	0.60	0.27	0.40	0.00	0.00	0.36	0.60
01	ALL	075	0	9.2	4	050	7.418986	7.627	6.200	0.52	0.61	0.27	0.40	0.04	0.02	0.89	0.74

these files were combined into the file that contained the required information for each pavement of the highways. This file was further processed by another computer program to estimate the parameters *H* and *I* of each pavement as well as *T* and distress group severity (*Ds*) and extent (*De*) for each pavement selected for this study. The most recent maintenance action and the year it was performed were also recorded in this file. The output of this program (see Table 5 for a typical output) was used to develop distress prediction models for various M&R actions. A simplified flowchart of major steps in this process is presented in Figure 3. Following examples briefly illustrate the method of calculating various parameters of distress prediction model represented by Equation 9.

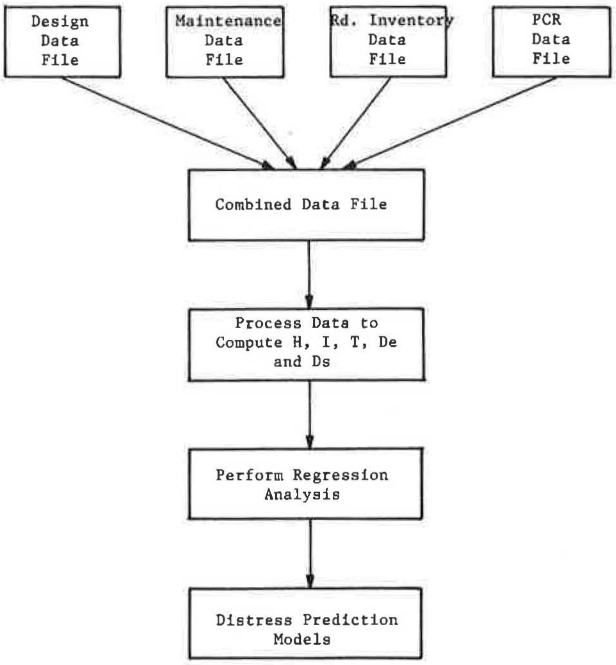

FIGURE 3 Simplified flowchart of process to develop distress prediction models.

Calculation of Parameters *H* and *I*

An asphalt overlay 6 in. thick was placed on an existing rigid pavement (JRCP) in 1985. The rigid pavement is 9 in. thick and is supported on a 6 in. granular subbase. The subgrade soil at the site was A-6 (ODOT classification).

Using these data, material properties of pavement layers were determined from Tables 3 and 4 as follows (see Equation 6):

	Hm (*in.*)	Em (*psi*)
Overlay (AC)	6	450,000
Base (PCC)	9	4,500,000
Subbase (granular)	6	20,000
Subgrade (A-6)	–	7,800

The value of the parameter *H* is calculated as follows:

$$H = 0.1[6.0\ (450{,}000/20{,}000)^{1/3} + 9(4{,}500{,}000/20{,}000)^{1/3} + 6(20{,}000/20{,}000)] = 7.768$$

Similarly, the parameter *I* is calculated as follows (see Equation 7):

$$I = 7{,}800/1{,}000 = 7.8$$

Calculations of Parameter *T*

The following traffic data were obtained for the pavement of the preceding example:

Traffic Data	*Value*
Total B- and C-trucks per day	3,000
Functional class of highway	01 (see Table 6)
Number of lanes in both directions	6
Year of traffic surveys	1987

These data were used to calculate the parameter T as illustrated:

1. Total number of B- and C-trucks in one direction = 3,000 × 0.5 = 1,500/day (assuming a 50-50 distribution in each direction.
2. Total number of trucks in the design lane = 1,500 × 0.8 = 1,200/day (0.8 is the design lane factor for a six-lane highway as recommended in ODOT design manual).
3. Number of B- and C-trucks in the design lane: the proportions of B- and C-trucks in the truck traffic were obtained from the ODOT data. A complete list of B- and C-truck distributions for various functional classes of roads in Ohio is presented in Table 6. Using values from this table for Functional Class 01, the numbers of B- and C-trucks in the design lane are as follows:

number of B-trucks/day = 0.875 × 1,200 = 1,050

number of C-trucks/day = 0.125 × 1,200 = 150

4. Calculations of E-18: these calculations require equivalency factors developed by the ODOT engineers for rigid and flexible pavements listed in Table 6. Because the original pavement was a rigid pavement, the E-18 factors related to rigid pavements were used for these calculations.

E-18 equivalent of B-trucks/day = 1,050 × 2.0591
= 2,162.06

E-18 equivalent of C-trucks/day = 150 × 0.2883 = 43.25

total E-18/day = 2,162.06 + 43.25 = 2,205.31

therefore,

total E-18/year = 2,205.31 × 365 = 804,938

5. Calculations of T for PCR years: the estimates of E-18 relate to the traffic survey of 1987. This pavement was overlaid in 1985, so traffic estimates for 1985 and 1986 were also required to calculate the appropriate value of T. This was done by assuming a constant traffic growth rate of 2 percent, which is an average value recommended by ODOT for highly populated areas represented by Functional Classes 01, 02, and 03 in Table 6. Traffic on all other classes of roads is assumed to grow at 1.5 percent per year. The calculations of the T-parameter for this example are summarized in Table 7.

Calculations of Distress Values *Ds* and *De*

The relationships represented by Equations 3 and 4 were used to estimate the distress group severity (*Ds*) and extent (*De*) of all observed distresses in each pavement. An example following these equations illustrates the method of calculating *Ds* and *De* of each distress group.

All available data for composite pavements of divided highways were analyzed using the method outlined in this section. As mentioned, a computer program was developed to perform these calculations. A partial listing of relevant data obtained from the output of this program is given in Table 5.

REGRESSION ANALYSIS OF DATA TO DEVELOP DISTRESS PREDICTION MODELS

Distress prediction models were developed by performing a regression analysis of data obtained from various ODOT files and processed as outlined. The general form of distress prediction models was represented by Equation 9. It is a nonlinear equation, therefore nonlinear regression analysis of the data was performed using the statistical package SAS. How-

TABLE 6 DISTRIBUTION OF B- AND C-TRUCKS AND 18-kip ESALs FOR VARIOUS FUNCTIONAL CLASSES OF ROADS (BASED ON ODOT'S 1986–1987 DATA)

Funct. Class	Descript.	Distribut.		18K-ESAL (Rigid)		18K-ESAL (Flexible)	
		B	C	B	C	B	C
01	Rural Interstate	0.875	0.125	2.0591	0.2883	1.5085	0.2744
02	Rural Principal Arterial	0.800	0.200	2.0836	1.8464	1.4969	1.1817
03	Rural Minor Arterial	0.333	0.667	2.0836	1.8464	1.4969	1.1817
41	Urban Interstate	0.667	0.333	2.1503	0.8846	1.5888	0.6698
42	Urban Fwy & Exwy	0.250	0.750	2.1503	0.8846	1.5888	0.6698
43	Urban Principal Arterial	0.500	0.500	1.5690	0.7034	1.1803	0.6092
44	Urban Minor Arterial	0.143	0.857	1.5690	0.7034	1.1803	0.6092

TABLE 7 SUMMARY OF CALCULATIONS TO ESTIMATE TRAFFIC PARAMETER *T*

Year	Annual Truck Traffic Millions of E-18	Cumulative Traffic, T Millions of E-18	Remarks
1985	0.7737	0	Major Maintenance Performed
1986	0.7892	0.7737	
1987	**0.8049**	1.529	Traffic Survey Year
1988	0.8210	2.3678	
1989	0.8375	3.1888	

ever, initial trials indicated that this method was not suitable for the type of data available for the study. The scatter as well as limited number of data (data were available for 5 years only) did not make it possible to obtain reasonable models from engineering considerations. Therefore, the equation was converted into a linear model and linear regression analysis was performed to obtain coefficients $a1-a4$ (see Equation 9). This transformation was performed by taking the natural log of both sides of the equation twice, as indicated later. The first transformation resulted in the following form of equation:

$$\ln D = -(a1\ H^{a2}\ H^{a2}\ I^{a3})/(T + 1)^{a4}$$

Because the estimated value of D varies from 0 to 1, the left side ($\ln D$) is zero or negative (≤ 0). Therefore, multiplying both sides of the equation and taking the natural log of both sides, the equation will transform to the form:

$$\ln(-\ln D) = \ln a1 + a2 \ln H + a3 \ln I - a4 \ln(T + 1) \qquad (10)$$

Equation 10 is a linear combination of transformed variables, so a linear regression analysis was performed after transforming the original variables into $[\ln(-\ln D)]$, $\ln H$, $\ln I$, and $\ln(T + 1)$. The regression coefficients $a1-a4$ were thus obtained for each prediction model.

As indicated earlier, distress prediction models were obtained for each distress group severity and extent. This resulted in eight models for each M&R action. There were five different M&R actions considered as flexible overlays on the existing rigid pavements by the ODOT engineers. A brief description of these overlays, along with their code numbers as assigned by ODOT engineers, is as follows:

Description of M&R Action	*Action Code*
Nonstructural AC overlay with minimum repair (thickness ≤ 3 in.)	050
Nonstructural AC overlay with repairs	060
Structural AC overlay with minimum repair (thickness > 3 in.)	070
Structural AC overlay with repairs	080
Crack and seat with AC overlay	090

Forty equations were thus developed for the composite pavements. A partial listing of regression coefficients $a1-a4$ obtained from this analysis is shown in Table 8.

The results of this analysis indicated that because of considerable scatter in the data, the coefficients $a2$ and $a3$ were

TABLE 8 REGRESSION COEFFICIENTS OF DISTRESS PREDICTION MODEL FOR M&R ACTION 050

Distress Group	Distress	a1	a2	a3	a4
#1	Ds	1.00	0.1197	0	-0.1718
	De	.52	0.1318	0	-0.1926
#2	Ds	2.78	0.0693	0	-0.3718
	De	2.04	0.0814	0	-04.188
#3	Ds	3.63	0.1830	0	-0.2308
	De	3.89	0.1607	0	-0.2037
#4	Ds	1.58	-0.1417	0	-0.8623
	De	0.69	0.1210	0	-0.4476

sometimes negative. A negative sign for either of these coefficients means that increasing values of H or I will result in more damage, so this was considered impractical. Therefore, the analysis was performed without the parameter that was associated with the negative coefficient. The listing of coefficients $a1-a4$ shown in Table 8 shows that the coefficient $a3$ is zero for the M&R Action 050. This decision was made after the statistical significance of coefficient $a3$ in the equation was checked with the help of an F-test. In this case it was found that its presence in the equation was not statistically significant at a 95 percent confidence level.

COMPARISON OF RESULTS WITH FIELD OBSERVATIONS

The distress prediction models developed for various M&R actions were used to compare the estimated distress values with those directly calculated from field data. A typical example of this comparison is shown in Table 9. Distress data for a segment of Interstate Route 75 in Allen County, Ohio, is listed in this table. The estimated values of *Ds* and *De* for each distress group are also listed in the last column of this table. These comparisons show that the estimates obtained from prediction models are comparable with those obtained from field data within the practical limits.

Several pavements were selected to compare the estimated PCR values with the observed PCR values. These comparisons also indicated that the estimated PCR values were comparable with the PCR calculations from field data. An example of these comparisons is shown in Table 10, which includes two pavements.

Highway segments along Interstate and state routes were also analyzed to compare the estimated PCR for the entire route (simulated network) with the PCR calculated from field data. Space limitations will not allow these comparisons to be documented. However, Table 11 shows a typical comparison of PCR estimates with those obtained from field data of State Route 033 during 1987–1990. These segments were treated with M&R Action 060 in 1986.

The comparisons indicated that the models were capable of producing comparable estimates of *Ds*, *De*, and PCR in most of the cases. Therefore, these models were considered suitable for use with the PMS-III program as originally intended.

TABLE 9 COMPARISON OF OBSERVED AND ESTIMATED DISTRESSES (ALLEN COUNTY, ROUTE I-75, MILEPOST 0.00, M&R ACTION 050)

ITEM		T mil E-18	Observed Distress	Estimated Distress
Distr. grp. # 1	Sev	2.236	0.25	0.353
		3.438	0.48	0.373
		4.701	0.32	0.388
		6.027	0.37	0.402
		7.419	0.52	0.413
	Ext	2.236	0.49	0.582
		3.438	0.59	0.600
		4.701	0.51	0.615
		6.027	0.60	0.627
		7.419	0.61	0.637
Distr. grp. # 2	Sev	2.236	0.08	0.126
		3.438	0.13	0.159
		4.701	0.16	0.187
		6.027	0.27	0.212
		7.419	0.27	0.235
	Ext	2.236	0.14	0.230
		3.438	0.24	0.275
		4.701	0.34	0.313
		6.027	0.40	0.345
		7.419	0.40	0.373
Distr. grp. # 3	Sev	2.236	0.00	0.018
		3.438	0.00	0.024
		4.701	0.00	0.030
		6.027	0.00	0.035
		7.419	0.04	0.040
	Ext	2.236	0.00	0.014
		3.438	0.00	0.019
		4.701	0.00	0.023
		6.027	0.00	0.027
		7.419	0.02	0.030
Distr. grp. # 4	Sev	2.236	0.36	0.650
		3.438	0.28	0.721
		4.701	0.49	0.768
		6.027	0.36	0.802
		7.419	0.89	0.828
	Ext	2.236	0.40	0.594
		3.438	0.46	0.636
		4.701	0.55	0.667
		6.027	0.60	0.692
		7.419	0.74	0.712

TABLE 10 COMPARISON OF OBSERVED AND ESTIMATED PCR FOR INDIVIDUAL HIGHWAY SEGMENTS (M&R ACTIONS 050 AND 060)

Seg. #	M&R ACTION	H	I	T Mil E-18	OBSERVED PCR	ESTIMATED PCR
1	050	7.9	6.2	8.118	71	72
				9.840	74	70
				11.596	74	69
				13.388	74	68
				15.216	69	67
				17.080	69	66
2	060	7.4	6.2	5.008	78	80
				6.323	74	78
				7.665	74	77
				9.003	70	76
				10.429	69	74
				11.853	69	73

TABLE 11 COMPARISON OF OBSERVED AND ESTIMATED PCR FOR ROUTE 033 IN OHIO (ALL SEGMENTS TREATED WITH M&R ACTION 060 IN 1986)

Beg. M.P.	Length Miles	Estimated PCR 1987	1988	1989	1990
21.70	0.04	92	91	90	89
22.50	3.80	92	91	90	89
26.30	3.50	91	90	89	87
29.80	0.30	90	89	87	85
30.10	0.70	90	89	87	85
Weighted Avg. of Estimates		91	90	89	88
Weighted Avg. of Field Data		98	90	90	87

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This paper describes an analytical procedure to develop distress prediction models suitable for a network-level PMS. The data were obtained from the ODOT files for road design, road maintenance, road inventory, and PCR.

Computer programs were developed to process the data in these files and combine all required data into one file (see Figure 3). These data were further processed to obtain suitable input for regression analysis so that the regression coefficients $a1-a4$ of Equation 9 could be determined. The linear transformation of Equation 9 as represented by Equation 10 was used to analyze the available data.

Distress prediction models were obtained for rigid pavements overlaid with AC of various thicknesses as well as cracked and seated rigid pavements overlaid with AC. Five different M&R actions were identified by the ODOT staff, which related to flexible overlays on existing rigid pavements. Eight equations were developed for each M&R action: one equation for each distress group severity and one equation for its extent. Thus, 40 equations were developed to satisfy the needs of prediction models for asphalt overlays on rigid pavements.

The predictive capabilities of the models were assessed as the estimated distress values were compared with the field-observed distress values of several pavements. The PCR estimates of selected pavements were compared with the PCR values calculated from field data (PCR surveys), and the PCR estimates of several pavements located on selected routes were compared with the calculated PCR obtained from field data. All these comparisons indicated that the predictive capabilities of the models obtained from the procedure described were reasonable. Some typical comparisons are shown in Tables 9, 10, and 11. The results of these comparisons indicated that distresses in rigid pavements overlaid with AC layer(s) can be predicted by the component layer thickness and its modulus of elasticity, subgrade soil strength, and traffic. A general form of relationship represented by Equation 9 was found to be suitable for this purpose.

Pavement condition records were available for 5 years. Therefore, these limits should be kept in mind if the models are to be extrapolated beyond the range of data used to develop them. A periodic updating of these models with future data will expand the range of their applicability.

ACKNOWLEDGMENTS

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Use of Expert Systems in Managing Pavement Maintenance in Egypt

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An attempt to apply expert systems to management of pavement maintenance in developing countries is presented. The development of this system is based, however, on data from Egypt. A twofold system has been developed to assist highway agencies that lack in-house experts in evaluation of asphalt pavements and assessment of maintenance and rehabilitation needs. The evaluation subsystem is an interactive algorithmic computer program, the output of which is an index-type rating of pavement condition—namely, the pavement condition index—calculated from distress data obtained from visual condition surveys. The maintenance and rehabilitation subsystem is an expert system that simulates a consultation between the engineer and an expert in the field of pavement maintenance. This expert system can be run as a stand-alone program with input data supplied by the user engineer, or it can be called from inside the environment of the algorithmic program to analyze its data base. The system has been developed and verified using data from portions of the Egyptian road network where comprehensive visual inspection data are available.

Most in-service pavements were built years ago; few new pavements are being constructed now. A high percentage of the total network mileage has deteriorated to conditions considered a functional failure, not performing the intended function of serving users safely and comfortably and instead damaging vehicles, slowing travel, increasing fuel consumption, and sometimes causing a hazardous ride.

A deteriorated pavement network needs localized repairs (e.g., crack sealing, pothole filling) and extended rehabilitation of entire pavement sections. Unfortunately, maintenance and rehabilitation (M&R) funds can not keep pace with M&R requirements; thus, there is a need for standard, practical decision-making procedures that can be applied to define what, where, and when M&R work should be done (*1*).

In developing countries—Egypt, for instance—highway agencies suffer (a) low M&R budget, and (b) the absence of an efficient system for managing the investments in pavements. This normally yields a random application of the limited funds to fill the most extreme needs for repair. The remaining budget proves inadequate to serve the total area involved, and the assumed recurrent maintenance suffers or, in most cases, is omitted altogether. The subsequent budget period usually shows that the pavement has deteriorated more rapidly than expected because of lack of maintenance, so more of the small budget is required for heavy remedial work, and the downward cycle of deterioration continues.

Many components of pavement maintenance management are complex and poorly structured, making algorithmic computations difficult (*2*). Pavement maintenance management requires the knowledge and expertise of experienced pavement engineers. Artificial intelligence (AI), a relatively new computer application and programming technology, and expert systems, a subset of AI (*3,4*), provide efficient and effective tools for handling expertise and decision logics. Thus, expert systems have great potential for addressing pavement maintenance needs (*5–13*). An expert system can systematically formalize and use the thought process and experience of experts as well as incorporate algorithmic computations when appropriate.

The selection and scheduling of M&R activities to a diversity of roadway section types, conditions, traffic characteristics, and such are repeated tasks in any highway agency that can benefit from the rule-based logic of an expert system, because these assignments are not made on the basis of exact engineering criteria. This is particularly true in developing countries, where such systems can play an important role in offsetting the lack of experience.

SYSTEM DESCRIPTION

This work effort aims at developing a simplified pavement condition evaluation system that uses microcomputer technology and that allows the user to select the most appropriate maintenance or rehabilitation action needed for upgrading pavement condition via an expert system consultation.

The system consists of two programs. The first is an algorithmic program that allows for the recording of pavement surface distress information, handles pavement condition index (PCI) calculations, and acquires data on the applicability of a variety of maintenance and rehabilitation activities and their unit costs and projected service lives. This program manages data input, storage, and retrieval. It also generates condition reports that can be used by the second program. The collection of distress data and the calculations required to convert them to a condition index are based on the PCI procedure, in which the network under consideration is divided into a set of branches (e.g., major streets), each of which is further divided into homogeneous sections (e.g., street blocks). Finally, each section is divided into several sample units. A random sample is selected from these sample units, and a detailed visual inspection is performed. The details of the procedure are available elsewhere (*14*).

The second program is an expert system that determines and ranks the maintenance and rehabilitation actions to be

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taken, on the basis of the data passed to it either manually by the user or automatically by the first algorithmic program. This expert system program is easy to use and is readily adaptable to allow for the incorporation of new required rules or changes in the existing ones. EXSYS Shell (*15*) was used to develop this program.

Developing the system to take the form of two programs captures the advantages and power of the two programming techniques. The algorithmic program carries the burden of the immense amount of computations; the expert system program suits the symbolic and heuristic nature of human expertise. The system has been developed and verified using data from portions of the Egyptian road network where comprehensive visual inspection data is available (*16*).

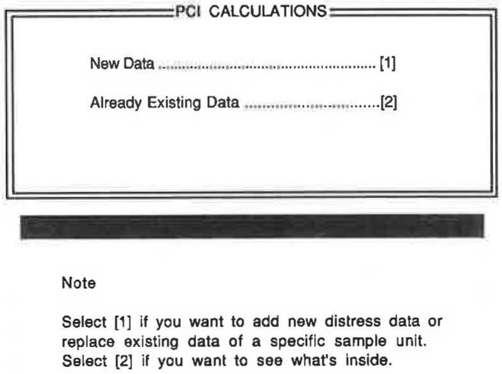
Algorithmic Program

Input

As a simple data manager, the program accepts new data or shows previously stored data upon a user's request. Figure 1 shows the first screen, in which the computer asks for the user's selection.

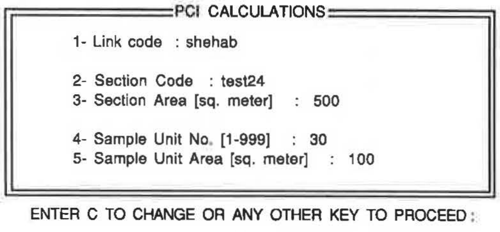
The program then responds (Figure 2), inquiring about branch or link code, which is a set of alphanumeric characters that facilitates reference to the link. It also asks for section code and its area and for sample unit number and its area. Typographical errors can be corrected upon the user's request.

The next phase is the input of distress data. The user supplies the data from condition surveys at random, and the program accumulates and arranges these data. Figure 3 shows distress types considered, the units in which they are measured, and the input process of existing distress types.



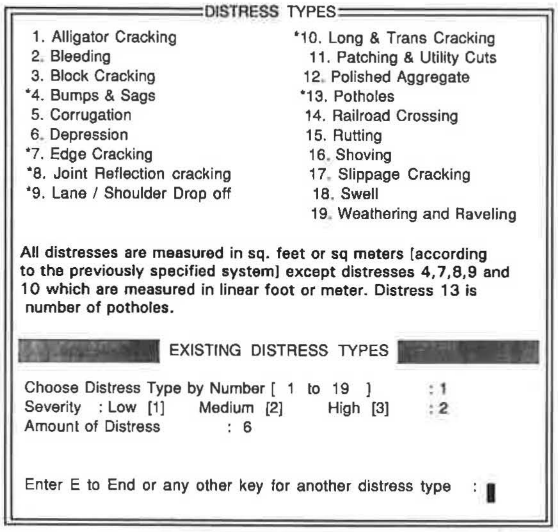
```
=PCI CALCULATIONS=

    New Data ............................................... [1]

    Already Existing Data ...........................[2]


Note

Select [1] if you want to add new distress data or
replace existing data of a specific sample unit.
Select [2] if you want to see what's inside.
```

FIGURE 1 New input or move to data base.



```
=PCI CALCULATIONS=
  1- Link code  : shehab

  2- Section Code  : test24
  3- Section Area [sq. meter]   :  500

  4- Sample Unit No. [1-999]   :  30
  5- Sample Unit Area [sq. meter]   :  100

ENTER C TO CHANGE OR ANY OTHER KEY TO PROCEED :
```

FIGURE 2 Identification data.



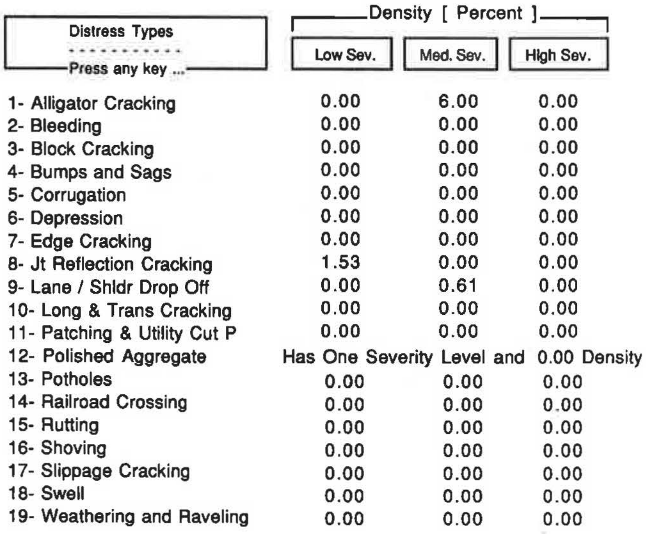
```
=DISTRESS TYPES=
 1. Alligator Cracking               *10. Long & Trans Cracking
 2. Bleeding                          11. Patching & Utility Cuts
 3. Block Cracking                    12. Polished Aggregate
*4. Bumps & Sags                     *13. Potholes
 5. Corrugation                       14. Railroad Crossing
 6. Depression                        15. Rutting
*7. Edge Cracking                     16. Shoving
*8. Joint Reflection cracking         17. Slippage Cracking
*9. Lane / Shoulder Drop off          18. Swell
                                      19. Weathering and Raveling

All distresses are measured in sq. feet or sq meters [according
to the previously specified system] except distresses 4,7,8,9 and
10 which are measured in linear foot or meter. Distress 13 is
number of potholes.

               EXISTING DISTRESS TYPES

Choose Distress Type by Number [  1  to  19  ]          : 1
Severity  : Low  [1]     Medium  [2]     High  [3]      : 2
Amount of Distress            :  6


Enter E to End or any other key for another distress type   :
```

FIGURE 3 Input of existing distress types.

Output

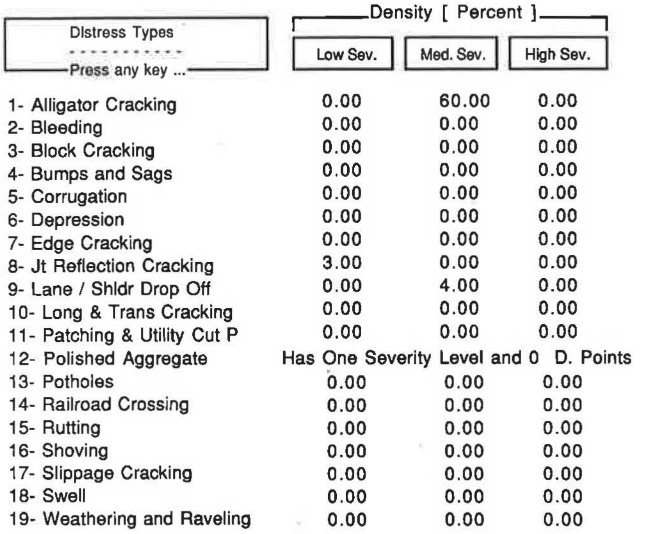
Data summaries appear as follows: (a) the density matrix, which includes the density of each distress type–severity combination, that is, the amount of each in percentage of the total area of the sample unit (Figure 4); (b) the deduct values associated with each distress type–severity combination (Figure 5); and (c) the deduct points of each in percentage of the total deduct points (Figure 6). This helps the maintenance decision maker to know the relative effect of each distress type–severity combination on the condition of the pavement as reflected by the number of deduct points.

Finally, the sample unit PCI, computed as described by Shahin and Kohn (*14*), is displayed in Figure 7.



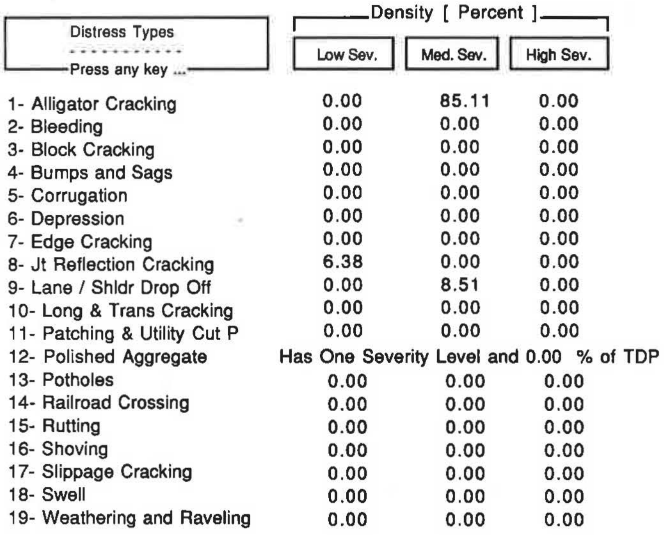
```
 Distress Types             ______Density [ Percent ]______
 -----------
 Press any key ...         Low Sev.    Med. Sev.    High Sev.

1- Alligator Cracking        0.00        6.00        0.00
2- Bleeding                  0.00        0.00        0.00
3- Block Cracking            0.00        0.00        0.00
4- Bumps and Sags            0.00        0.00        0.00
5- Corrugation               0.00        0.00        0.00
6- Depression                0.00        0.00        0.00
7- Edge Cracking             0.00        0.00        0.00
8- Jt Reflection Cracking    1.53        0.00        0.00
9- Lane / Shldr Drop Off     0.00        0.61        0.00
10- Long & Trans Cracking    0.00        0.00        0.00
11- Patching & Utility Cut P 0.00        0.00        0.00
12- Polished Aggregate    Has One Severity Level and 0.00 Density
13- Potholes                 0.00        0.00        0.00
14- Railroad Crossing        0.00        0.00        0.00
15- Rutting                  0.00        0.00        0.00
16- Shoving                  0.00        0.00        0.00
17- Slippage Cracking        0.00        0.00        0.00
18- Swell                    0.00        0.00        0.00
19- Weathering and Raveling  0.00        0.00        0.00
```

FIGURE 4 Density of existing distresses.



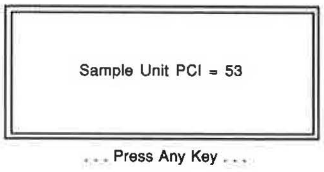
```
 Distress Types          ____Density [ Percent ]____
 - - - - - - - - - -     Low Sev.   Med. Sev.   High Sev.
 ---Press any key ...---

1- Alligator Cracking          0.00      60.00     0.00
2- Bleeding                    0.00      0.00      0.00
3- Block Cracking              0.00      0.00      0.00
4- Bumps and Sags              0.00      0.00      0.00
5- Corrugation                 0.00      0.00      0.00
6- Depression                  0.00      0.00      0.00
7- Edge Cracking               0.00      0.00      0.00
8- Jt Reflection Cracking      3.00      0.00      0.00
9- Lane / Shldr Drop Off       0.00      4.00      0.00
10- Long & Trans Cracking      0.00      0.00      0.00
11- Patching & Utility Cut P   0.00      0.00      0.00
12- Polished Aggregate      Has One Severity Level and 0  D. Points
13- Potholes                    0.00      0.00      0.00
14- Railroad Crossing           0.00      0.00      0.00
15- Rutting                     0.00      0.00      0.00
16- Shoving                     0.00      0.00      0.00
17- Slippage Cracking           0.00      0.00      0.00
18- Swell                       0.00      0.00      0.00
19- Weathering and Raveling     0.00      0.00      0.00
```

FIGURE 5 Deduct points due to existing distresses.



```
 Distress Types          ____Density [ Percent ]____
 - - - - - - - - - -     Low Sev.   Med. Sev.   High Sev.
 ---Press any key ...---

1- Alligator Cracking          0.00      85.11     0.00
2- Bleeding                    0.00      0.00      0.00
3- Block Cracking              0.00      0.00      0.00
4- Bumps and Sags              0.00      0.00      0.00
5- Corrugation                 0.00      0.00      0.00
6- Depression                  0.00      0.00      0.00
7- Edge Cracking               0.00      0.00      0.00
8- Jt Reflection Cracking      6.38      0.00      0.00
9- Lane / Shldr Drop Off       0.00      8.51      0.00
10- Long & Trans Cracking      0.00      0.00      0.00
11- Patching & Utility Cut P   0.00      0.00      0.00
12- Polished Aggregate      Has One Severity Level and 0.00  % of TDP
13- Potholes                   0.00      0.00      0.00
14- Railroad Crossing          0.00      0.00      0.00
15- Rutting                    0.00      0.00      0.00
16- Shoving                    0.00      0.00      0.00
17- Slippage Cracking          0.00      0.00      0.00
18- Swell                      0.00      0.00      0.00
19- Weathering and Raveling    0.00      0.00      0.00
```

FIGURE 6 Deduct points (DP) as percentage of total DP.



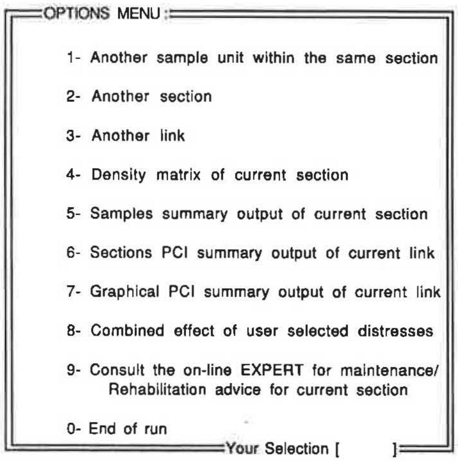
```
Sample Unit PCI = 53

. . . Press Any Key . . .
```

FIGURE 7 Sample unit PCI.

Options

The first three options shown in Figure 8 are provided to facilitate proceeding in the process of data input or help the user move around in the environment of stored data.

Option 4 shows the distress type–severity combinations of the current section extrapolated from distress data of the sample units in that section.



```
==OPTIONS MENU :=========================================
  1- Another sample unit within the same section
  2- Another section
  3- Another link
  4- Density matrix of current section
  5- Samples summary output of current section
  6- Sections PCI summary output of current link
  7- Graphical PCI summary output of current link
  8- Combined effect of user selected distresses
  9- Consult the on-line EXPERT for maintenance/
       Rehabilitation advice for current section
  0- End of run
=========================Your Selection [       ]=======
```

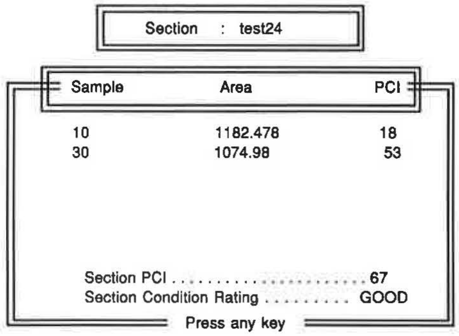
FIGURE 8 Options menu.

Option 5 (Figure 9) shows a summary output of the current section displaying its sample units, their areas, and their PCI values, and finally the PCI of the entire section and its condition rating.

Option 6 (Figure 10) summarizes the output of the current link, displaying its sections, their areas, their PCI values, and the pavement condition rating.

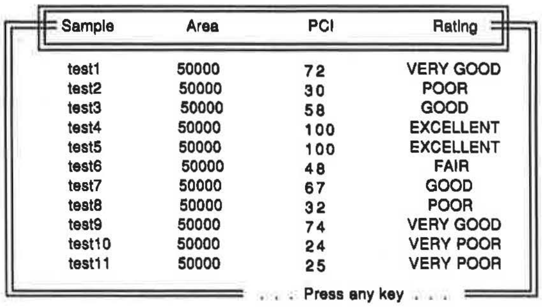
Option 7 (Figure 11) displays a graphical summary of the branch's condition. This is of particular importance to the decision maker in adjusting the decision of the expert system. For example, suppose a link consists of 25 sections, 24 of which are badly deteriorated, and the expert system's advice is to overlay them all. Only one of the sections is in good condition and requires only some recurrent maintenance such as crack sealing and small patches. In this case, it is more appropriate to take an overlay decision for the entire link, including the one in good condition.

Option 8 enables the user to determine the combined effect of some selected distress types on the condition of the pavement (Figures 12 and 13). To explain, consider the following case: low-severity block cracking extended over a wide portion of the pavement can result in, say, 20 deduct points, yielding a PCI of 80. A localized high-severity alligator cracking can cause the same 20 deduct points, reflecting (but deceptively) the same pavement condition. In fact, alligator



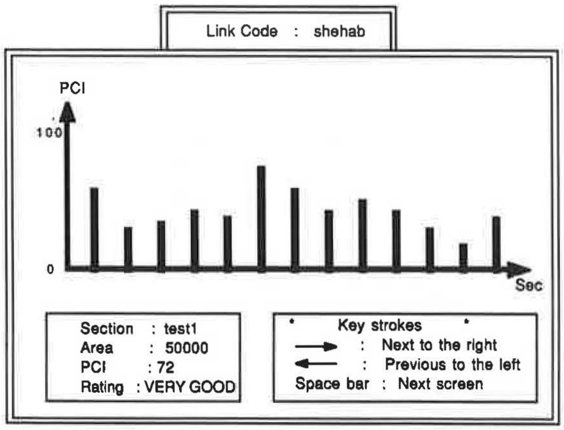
```
      Section  :  test24

  Sample          Area            PCI
  10              1182.478        18
  30              1074.98         53

  Section PCI . . . . . . . . . . . . . . . . . . . 67
  Section Condition Rating . . . . . . . . .  GOOD
  ============== Press any key ==============
```

FIGURE 9 Summary of sample units in current section.



```
Sample      Area      PCI       Rating

test1       50000     72        VERY GOOD
test2       50000     30        POOR
test3       50000     58        GOOD
test4       50000     100       EXCELLENT
test5       50000     100       EXCELLENT
test6       50000     48        FAIR
test7       50000     67        GOOD
test8       50000     32        POOR
test9       50000     74        VERY GOOD
test10      50000     24        VERY POOR
test11      50000     25        VERY POOR
              . . . Press any key . . .
```

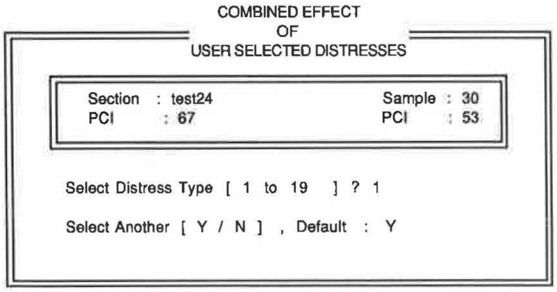
FIGURE 10 Summary of sections in current link.



```
Section  : test1
Area     : 50000
PCI      : 72
Rating   : VERY GOOD
```

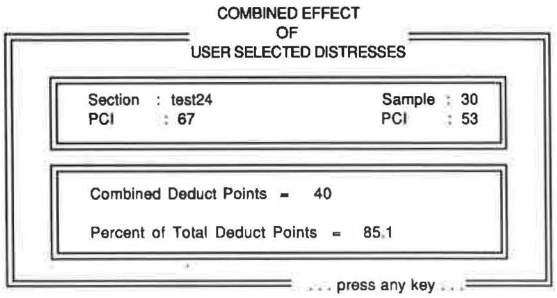
```
•   Key strokes   •
——►  :  Next to the right
◄——  :  Previous to the left
Space bar  :  Next screen
```

FIGURE 11 Graphical summary of current link.



```
COMBINED EFFECT
OF
USER SELECTED DISTRESSES

Section  : test24                Sample : 30
PCI      : 67                    PCI    : 53

Select Distress Type  [ 1 to 19 ] ? 1

Select Another [ Y / N ] , Default : Y
```

FIGURE 12 Inclusion of distress for combined effect.



```
COMBINED EFFECT
OF
USER SELECTED DISTRESSES

Section  : test24                Sample : 30
PCI      : 67                    PCI    : 53

Combined Deduct Points  =  40

Percent of Total Deduct Points  =  85.1

                  . . . press any key . . .
```

FIGURE 13 Combined effect of chosen distresses.

cracking is one of what are called load-related distresses, the existence of which indicates that the pavement is structurally weak and incapable of carrying the traffic loads. This structural inadequacy might require rebuilding the defected area or at least a full-depth patch. On the other hand, the remedy for the block cracking, which probably developed as a result of pavement aging or shrinkage of the asphalt surface, can be simple crack sealing. This means that although the deduct value is the same in both cases, the causes and remedial actions might differ drastically. So, through this option, the decision maker can choose, for instance, the load-related distresses and examine their combined effect on pavement condition.

Finally, Option 9 prepares for calling the expert system. This option is described in detail in the following paragraph.

First, the program displays all the maintenance activities for the user to determine whether any is not applicable. A maintenance activity may be not applicable for many reasons, such as the lack of materials, the absence of skilled labor or equipment or prohibitive cost. The user can set one activity or more as not applicable or, if difficulties exist but they are not prohibitive, the user can set the activity as applicable but not desirable. This type of data is entered by filling the activity applicability matrix as shown in Figure 14.

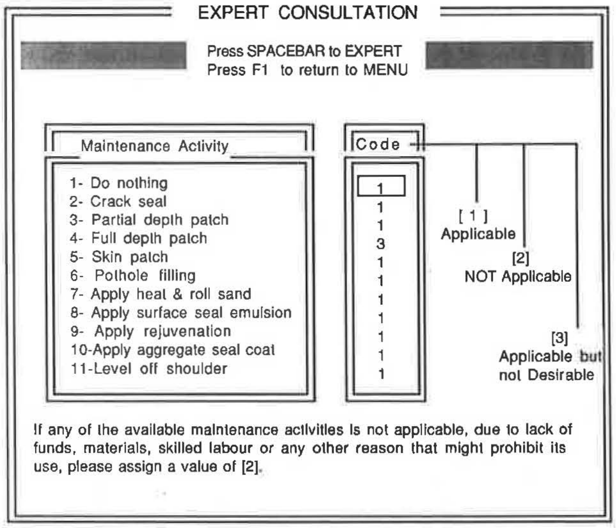
Next, the program displays the cost/life matrix (Figure 15), which contains all maintenance activities, each with its unit cost and associated service life. The unit cost can be in the form of the present worth.

At this point, all data are ready for calling the expert system, the output of which will be the M&R activities needed to upgrade the pavement condition.

Expert Consultation

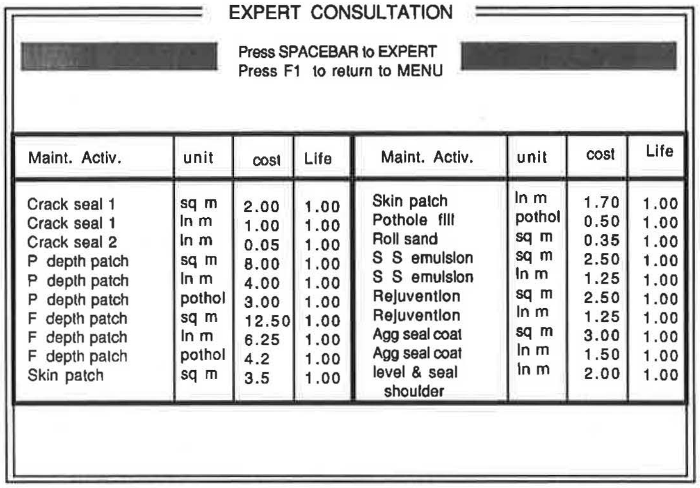
Main Output

The expert system works on the data passed to it by the algorithmic program, using the rules in the knowledge base.



```
EXPERT CONSULTATION

Press SPACEBAR to EXPERT
Press F1  to return to MENU

Maintenance Activity                     Code
1- Do nothing                            1
2- Crack seal                            1
3- Partial depth patch                   1
4- Full depth patch                      3
5- Skin patch                            1
6- Pothole filling                       1
7- Apply heat & roll sand                1
8- Apply surface seal emulsion           1
9- Apply rejuvenation                    1
10-Apply aggregate seal coat             1
11-Level off shoulder                    1

[ 1 ] Applicable
[2] NOT Applicable
[3] Applicable but not Desirable

If any of the available maintenance activities is not applicable, due to lack of
funds, materials, skilled labour or any other reason that might prohibit its
use, please assign a value of [2].
```

FIGURE 14 Activity applicability matrix.



EXPERT CONSULTATION

Press SPACEBAR to EXPERT
Press F1 to return to MENU

Maint. Activ.	unit	cost	Life	Maint. Activ.	unit	cost	Life
Crack seal 1	sq m	2.00	1.00	Skin patch	ln m	1.70	1.00
Crack seal 1	ln m	1.00	1.00	Pothole fill	pothol	0.50	1.00
Crack seal 2	ln m	0.05	1.00	Roll sand	sq m	0.35	1.00
P depth patch	sq m	8.00	1.00	S S emulsion	sq m	2.50	1.00
P depth patch	ln m	4.00	1.00	S S emulsion	ln m	1.25	1.00
P depth patch	pothol	3.00	1.00	Rejuvention	sq m	2.50	1.00
F depth patch	sq m	12.50	1.00	Rejuvention	ln m	1.25	1.00
F depth patch	ln m	6.25	1.00	Agg seal coat	sq m	3.00	1.00
F depth patch	pothol	4.2	1.00	Agg seal coat	ln m	1.50	1.00
Skin patch	sq m	3.5	1.00	level & seal shoulder	ln m	2.00	1.00

FIGURE 15 Cost/life matrix.

Using the PCI value of the current section and PCI limits that correspond to the highway class, the expert system determines whether to rehabilitate the entire pavement section or to make localized remedial actions for each existing distress type.

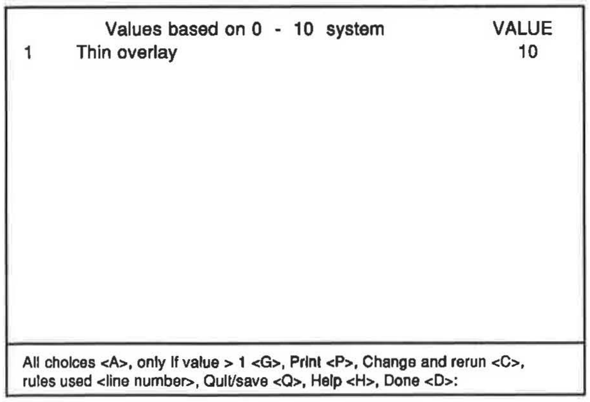
Examples of the rehabilitation decision are (a) apply a thin (functional) overlay, (b) apply a thick (structural) overlay, or (c) strengthen the pavement and then apply an overlay. The type of rehabilitation depends on the PCI value and the class of the highway. An example of the output screen is shown in Figure 16: the pavement section needs a thin overlay, and because this solution is unique (i.e., there are no possible alternatives), it takes a probability value of 10 (the maximum on a 0–10 scale).

If the pavement condition does not dictate rehabilitation, the expert system determines the suitable maintenance actions, gives them equal probability of 5, then checks whether any of the candidate actions is not applicable. If so, it is assigned a lower probability value of 1. The rest of the applicable activities are mutually compared for cost effectiveness, and the one that has lower cost/life value is assigned a higher probability value of 9.

Finally, a list of all candidate activities is displayed, the activities arranged according to final averaged probability value—the most likely first, the next likely second, and so on. For example, for a medium-severity depression, and according to activities applicability and cost/life data shown in Figure 17, the output takes the form shown in Figure 18, which indicates that a partial-depth patch is the most probable maintenance action to be taken. Besides, a comparison of the probability values indicates the relative likelihood of maintenance actions. If more than one maintenance action receive equal final probability values, they are displayed in alphabetical order, which means no real difference in rank. The process repeats until all existing distress types are considered.

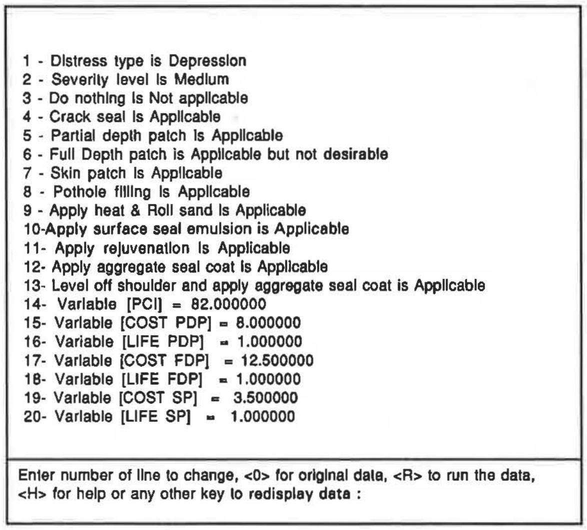
Supporting Outputs

The normal options available in most expert system shells were used in this program to provide the user with several supporting outputs as described in the following.



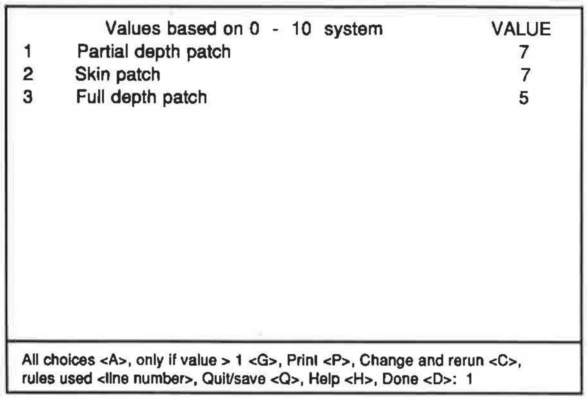
```
        Values based on 0 - 10 system              VALUE
1    Thin overlay                                     10

All choices <A>, only if value > 1 <G>, Print <P>, Change and rerun <C>,
rules used <line number>, Quit/save <Q>, Help <H>, Done <D>:
```

FIGURE 16 Example of rehabilitation output.



```
1 - Distress type is Depression
2 - Severity level is Medium
3 - Do nothing is Not applicable
4 - Crack seal is Applicable
5 - Partial depth patch is Applicable
6 - Full Depth patch is Applicable but not desirable
7 - Skin patch is Applicable
8 - Pothole filling is Applicable
9 - Apply heat & Roll sand is Applicable
10-Apply surface seal emulsion is Applicable
11- Apply rejuvenation is Applicable
12- Apply aggregate seal coat is Applicable
13- Level off shoulder and apply aggregate seal coat is Applicable
14- Variable [PCI] = 82.000000
15- Variable [COST PDP] = 8.000000
16- Variable [LIFE PDP] = 1.000000
17- Variable [COST FDP] = 12.500000
18- Variable [LIFE FDP] = 1.000000
19- Variable [COST SP] = 3.500000
20- Variable [LIFE SP] = 1.000000

Enter number of line to change, <0> for original data, <R> to run the data,
<H> for help or any other key to redisplay data :
```

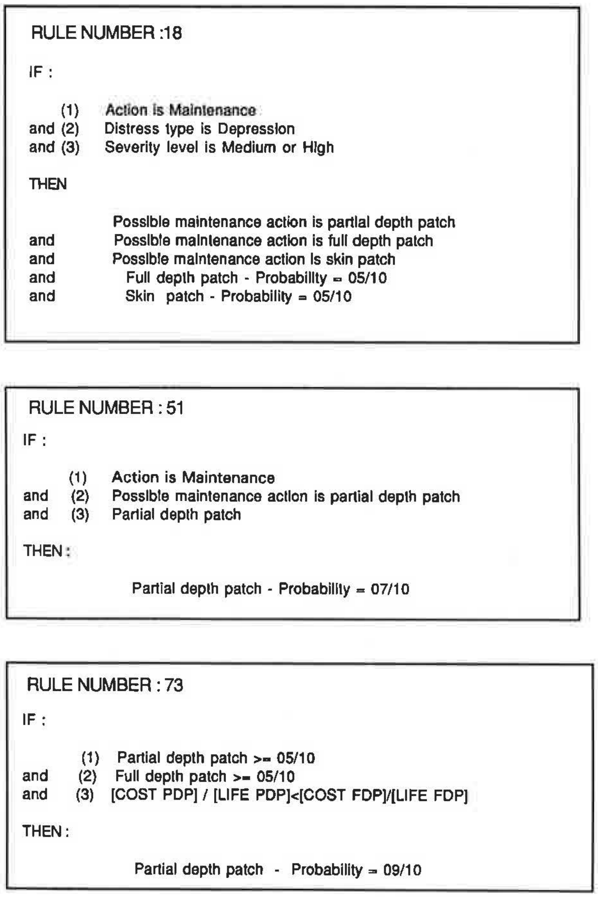
FIGURE 17 Data summary.



```
          Values based on 0  -  10  system                VALUE
1     Partial depth patch                                    7
2     Skin patch                                             7
3     Full depth patch                                       5

All choices <A>, only if value > 1 <G>, Print <P>, Change and rerun <C>,
rules used <line number>, Quit/save <Q>, Help <H>, Done <D>: 1
```

FIGURE 18 Example of maintenance output.

Tracing the Decision The system allows the user to trace back how the program arrived at its final value for a specific choice by entering the line number of any choice; the program will respond by displaying all rules used to determine the value of that choice. For instance, in Figure 18, a user who wants to know how the final value for the partial-depth patch was reached would enter "1," which is the line number for this choice, and the computer displays the rules as shown in Figure 19.



```
RULE NUMBER :18
IF :

     (1)   Action is Maintenance
and (2)   Distress type is Depression
and (3)   Severity level is Medium or High

THEN

            Possible maintenance action is partial depth patch
and        Possible maintenance action is full depth patch
and        Possible maintenance action is skin patch
and          Full depth patch - Probability = 05/10
and          Skin  patch - Probability = 05/10
```

```
RULE NUMBER : 51
IF :

       (1)   Action is Maintenance
and   (2)   Possible maintenance action is partial depth patch
and   (3)   Partial depth patch

THEN :

                 Partial depth patch - Probability = 07/10
```

```
RULE NUMBER : 73
IF :

        (1)   Partial depth patch >= 05/10
and    (2)   Full depth patch >= 05/10
and    (3)   [COST PDP] / [LIFE PDP]<[COST FDP]/[LIFE FDP]

THEN :

                 Partial depth patch  -  Probability = 09/10
```

FIGURE 19 Asking how conclusions were drawn.

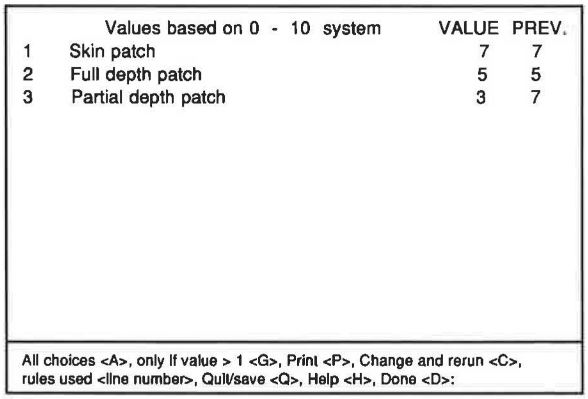
Checking Triggered Rules When a rule is displayed, the user has the option of asking how the computer knows a condition in the IF part is true. To do this, the user enters the line number of the IF condition. The computer will answer with one of several responses:

1. It may display the rule or rules that led it to derive the information. A rule used for derivation will have information about the condition the user is asking about in its THEN part. The user can then continue asking how the computer knew that the rule's IF conditions were true and so on until the end of the chain of rules.
2. The computer may respond that the user provided the information to the program.
3. If the information was provided by an external program call, the computer gives the user the name of that program.
4. The computer may respond that it does not yet know if the condition is true or not. This can occur when the user asks the computer WHY in response to its question. The rule displayed may not have been fully tested yet.

Changing and Rerunning Of the very powerful facilities given to the user is the change and rerun option. It is an easy way to test how changes in input affect conclusions. The user can change one or more items of the input data while holding the others constant, rerun the program using the adjusted data, and see the effect of the changes on the outcome. The original values of the choices can be saved for comparison with the new values.

This option gives the maintenance decision maker the ability to have a dialogue with the expert system. Considering the previous example of medium-severity depression, suppose that the decision maker wants to see what happens if partial-depth patches are not applicable. Changing Item 5 in Figure 17 to "partial-depth patch is not applicable" will yield the new output shown in Figure 20, along with the previous output for comparison.

To change the data, the user is asked if he or she wishes to save the current values for comparison with the new ones that will be calculated. The program will then display a list of all information that the user provided earlier. The user enters the number of the statement to be changed and the



```
          Values based on 0  -  10  system          VALUE  PREV.
1     Skin patch                                         7      7
2     Full depth patch                                   5      5
3     Partial depth patch                                3      7

All choices <A>, only if value > 1 <G>, Print <P>, Change and rerun <C>,
rules used <line number>, Quit/save <Q>, Help <H>, Done <D>:
```

FIGURE 20 Changing and rerunning data.

program will ask for changes. The user answers the questions with the new data to be tried and continues changing statements. If, because of the changes the user made, the program needs more information, it will ask for it. The program finally displays the new list of choices. If the user opted to have the previous values saved for comparison, they too will be displayed.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

In this research effort, a microcomputer-based pavement maintenance decision support system was developed using artificial intelligence and expert systems technology. This system is intended to be a stand-alone or independent maintenance decisions support system and to be part of a continuing effort to produce an integrated pavement maintenance management system. The developed system is twofold so that it allows for pavement condition monitoring and evaluation via an algorithmic program using the PCI procedure. It also allows the user to select the most appropriate maintenance or rehabilitation actions needed to upgrade the condition of the pavement via an expert system program.

The developed system is easy to use, and, more important, the knowledge base of the expert system is adaptable to incorporate new maintenance and rehabilitation strategies and to expand the user-expert consultation details when required.

Conclusions

The outcome of this work is not a new theory, a new understanding of an existing one, or any other theoretical output. Instead, it is a practical output in the form of a simple working system built to fulfill the need of the Egyptian highway agencies and engineers involved in pavement evaluation and maintenance. The developed system represents an attempt to apply the technology of artificial intelligence and expert systems to the domain of pavement maintenance management in developing countries. However, added to the value of the developed pavement maintenance management expert system, a number of conclusions can be drawn:

1. A pavement maintenance management expert system is possible, justified, and appropriate, and pavement management is an ideal application area for expert systems technology.
2. The large number of mathematical computations involved in a pavement maintenance management system makes the development of an expert system difficult. To overcome this problem, an algorithmic program using one of the conventional programming languages can be built to relieve the burden of these computations and free the expert system program to handle the heuristic rules supporting the maintenance decisions.
3. This expert system will be valuable to Egyptian highway agencies, especially the local limited ones, that lack in-house expertise. It is also a useful tool for novices to enhance their M&R skills.

Recommendations

To enhance the system capabilities, the following recommendations are suggested:

1. Pavement evaluation not only should be based on visual inspection data but also must incorporate roughness, a measure of structural capability, and a safety measure. This will enhance the exactness and effectiveness of M&R decisions.
2. A life-cycle cost-analysis procedure, including modules to calculate the service lives of M&R alternatives, can replace the user in providing the necessary data for the cost/life matrix.
3. An external data base including information on material, labor, and equipment requirements for different M&R alternatives can ease (or replace) the task of filling the activity applicability matrix.
4. Other rehabilitation techniques not now included can be added if applicable.
5. The system assumes that the M&R activities will be performed at the same year of evaluation, which is almost never the case. The system can be enhanced to allow the user to specify the year of implementation and have the system give M&R advice appropriate for the pavement's projected condition in that year.

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Network-Level Pavement Management in New York State: A Goal-Oriented Approach

DONALD N. GEOFFROY AND JOHN J. SHUFON

The New York State Department of Transportation practices pavement management at two distinct levels: network, which deals with program development, and project, which addresses treatment selection. This two-tiered approach works well in New York, where decisions on project and treatment selection are made in 11 regional offices given policy guidance from the main office in Albany. The department's network-level pavement management system is described. The system is goal-driven and is designed to operate in a decentralized decision-making environment. Each step in the network level process is discussed—from needs estimating through goal setting and performance monitoring. In addition, an improved pavement condition survey methodology is introduced. The survey involves the use of photographic scales of pavement condition and the collection of specific distress symptoms called dominant distresses. Pavement management systems must be tailored to the organizational structure of the implementing agency. Although the decision-making processes of pavement management systems are generally not transportable, the principles of a goal-oriented approach to managing pavements are, and they should be considered by highway agencies that are developing a network-level pavement management system.

The highway network in New York State is aging. More than a third of the 15,000–centerline-mi state highway system was constructed during the Interstate "big build" era, and many facilities are simultaneously reaching the end of their service lives. This problem has been exacerbated throughout the years by budget cutbacks for labor-intensive, low-profile preventive maintenance activities such as crack sealing and drainage-ditch cleaning. The challenge facing the New York State Department of Transportation (NYSDOT) is to repair the thousands of miles of highways concurrently falling into poor condition while properly maintaining the rest of the network to avert a future infrastructure crisis. All this must be accomplished in an environment of fiscal austerity.

Recognizing the need for better information and a systematic process to help department management make judicious decisions on funding levels, project priorities, and pavement repair strategies and timing, Commissioner Franklin E. White in July 1987 appointed a pavement management steering committee. The committee was charged with recommending the appropriate direction for the department to follow in approaching the long-term goal of a comprehensive, department-wide system for managing the condition and usefulness of, and expenditures for, the pavement structures of the state highway system.

In January 1989 the steering committee released a comprehensive plan that provided the course of action necessary to achieve the commissioner's strategic goal. The plan, which consists of 23 recommendations, addresses all aspects of managing pavements from planning through design and construction. Central to its theme is the clear distinction between network- and project-level pavement management and the data requirements associated with each. The purpose of this paper is to review the progress made in implementing a network-level pavement management system (PMS) in New York State. The paper will introduce a newly developed network pavement condition survey and demonstrate how the condition data feed the department's goal-oriented capital programming. For the sake of completeness, the department's project-level PMS will also be briefly outlined.

ORGANIZATIONAL STRUCTURE OF NYSDOT

It is well documented that to be successful, a PMS must be tailored to the organizational structure, culture, and decision-making process of the implementing agency (*1–5*). Pavement management systems are generally not transportable: what works for one agency may not work for another. To understand network-level pavement management in New York State, one must understand how the department is organized and how decisions are made in developing a program of pavement projects.

NYSDOT is a large organization responsible for managing a complex highway system that accommodates more than 50 billion vehicle-mi of travel a year. The department consists of a central (or main) office and 11 regional offices dispersed throughout the state. The main office is divided into functional divisions (e.g., Planning, Design, Technical Services, Highway Maintenance, etc.), each headed by a division director who reports directly to executive management. The main office is responsible for developing policy, establishing goals, allocating funds to the regions, and monitoring regional accomplishments. In addition, the main office is responsible for preparing the department's annual budget and selling the budget to oversight organizations such as the governor's budget division and the legislature. The budgeting process in New York State is complex. Funding is provided to the department in two separate allocations: (a) operating funds, which finance salaries, equipment, and some materials for pavement repairs

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to be done by agency forces, and (b) a capital allocation, which funds work to be done through competitively bid contracts. Resources are allocated to each region in the same manner.

The organizational structure of each regional office parallels that of the main office. Regional group directors report to the regional director (RD) and receive program guidance from both the RD and the main office division director of the functional area. The regions are responsible for developing a 5-year program of capital and maintenance projects for pavements and bridges; preparing plans, specifications, and estimates; and supervising the construction and maintenance of their highway systems.

Everyday pavement maintenance activities are performed by highway maintenance personnel in 65 field offices called residencies, each headed by a licensed professional engineer. Residency boundaries are generally coterminous with county lines. The resident engineers (REs), who report to the regional group director for highway maintenance, serve as the department's first-line pavement managers. Their intimate knowledge of the highway systems under their auspices is invaluable to developing a program of pavement projects. The underlying precept of the department's network-level PMS is that massive amounts of data would need to be collected in order to replicate the firsthand knowledge of the RE. This is particularly true in New York State, where pavement age, traffic loadings, climate, soils, and terrain vary considerably among regions and sometimes even between adjacent residencies. Coupling the RE's highway system experience with technical tools at the network level is a major strength of the department's PMS. An overview of the system is presented.

OVERVIEW OF NETWORK-LEVEL SYSTEM

Pavement management at the network level deals with summary information about the entire highway network. As such, it involves policy and programming decisions frequently made by upper management (*6*). Figure 1 presents a flowchart of the NYSDOT network-level PMS. The system is goal-driven and is designed to operate in a decentralized decision-making environment where choices on which pavement sections to treat and when and how to treat them are made in the regions given policy and technical guidance from the main office. Data from the annual network pavement condition survey are used to monitor the general health of the highway network, to estimate regional needs, to set goals, and to be input to the fund allocation process. Working within its allocation and given its pavement goal, each regional office develops a comprehensive 5-year program of pavement projects that integrates the spectrum of treatments from preventive maintenance through major rehabilitation.

Project lists and summary information are submitted to the main office, where the program is reviewed to ensure compliance with the pavement goals. Once approved, implementation begins for the first-year element of the program. The network survey data are used to measure the impact of program implementation on condition and to provide feedback for setting the next year's goals. It is important to understand that the output of the NYSDOT network-level system is a program of pavement projects with an estimate of project scope and cost. Details about designing the specific engineering treatment based on least life-cycle costs are addressed as part of the project-level PMS. The rest of this paper describes each of the process boxes shown in Figure 1.

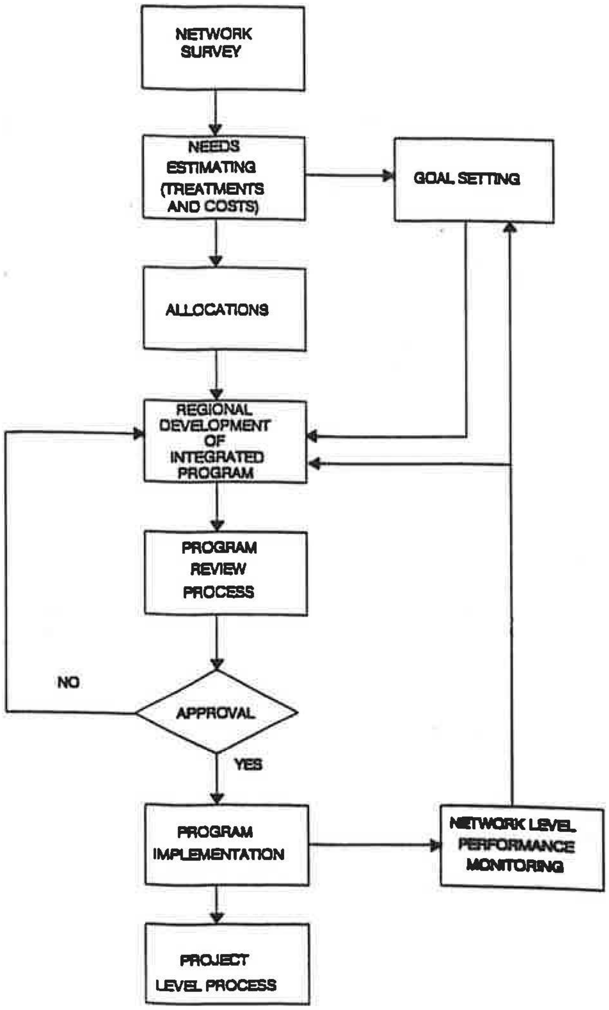

FIGURE 1 NYSDOT network-level pavement management process.

Network Pavement Condition Survey

Accurate and current pavement condition data are vital to sound pavement management. The amount and type of data collected depend primarily on the intended uses of the data in the management process. Pavement condition is often assessed by analyzing data on pavement distresses, roughness, structural adequacy, and friction. Clearly, collecting and processing these data for each highway link every year would be ideal. This is not possible, however, on large highway systems such as New York's without a very large expenditure of funds. Given the decentralized approach to managing pavements in New York, which builds on the firsthand knowledge and experience of the REs, the data-collection requirements for network-level activities are significantly less than many of the more traditional pavement management systems. Very detailed data are collected only after a pavement section has been programmed in order to select the appropriate treatment. This two-tiered approach leads to efficient data collecting, processing, and reporting.

Since 1981, NYSDOT has used a moving-vehicle windshield survey to assess the pavement condition of the network. Data are collected by crews from the 11 regional offices trained to use NYSDOT's pavement condition survey methodology. The rating procedure involves carefully developed photographic scales in which photographs show the condition of pavements at various stages of deterioration rather than specific distresses. This procedure was designed to permit rapid and repeatable estimates of overall condition at a low cost. The development and use of photographic scales by NYSDOT has been well documented over the years (*7–9*).

In 1990, the network survey method was enhanced and implemented. The survey was modified to enable each highway section to be classified into one of five general treatment categories:

- Do nothing,
- Preventive maintenance,
- Corrective maintenance,
- Rehabilitation, and
- Major rehabilitation.

Table 1 lists typical repair actions for each of the general treatment categories.

The new surface rating measure was developed by a panel of pavement experts assembled from the main office and the Albany regional office. The photographic scales, one for each pavement type, were constructed so that each of the scale points represents a pavement requiring one of the general treatments. Table 2 shows this relationship. In addition, the panel decided that a dominant distress feature would be added to the survey method. A dominant distress is defined as a specific distress symptom, observable at survey speeds, that requires a treatment more extensive than the treatment category triggered solely by the surface rating (*10*). For example, if a full-depth asphalt concrete pavement were to receive a surface rating of 6, the treatment category assigned would be corrective maintenance. However, if a structural problem such as alligator cracking were present, a treatment more extensive than corrective maintenance would be required for proper repair. Thus, alligator cracking is a dominant distress for a flexible pavement. Table 3 lists the distress symptoms that the expert panel determined to be dominant.

In May 1990, crews (a driver and a rater) from the 11 regional offices were trained to use the improved survey procedure. Data were collected during the late spring and early summer, batch processed and summarized in the main office, and supplied back to the regions in the early fall, in time to be used for the development of each region's annual Goal-Oriented Capital Program (GOCP). As in past years, data integrity was assured through main office audits of the re-

TABLE 1 NETWORK-LEVEL DISTRESS TREATMENT CATEGORIES AND TYPICAL REPAIR ACTIONS

TREATMENT CATEGORIES	PAVEMENT TYPE		
	RIGID	FLEXIBLE	OVERLAY
Do Nothing	-----	-----	-----
Preventive Maintenance	Reseal Joints	Fill Cracks	Fill Cracks
Preventive Maintenance (High Cost)	Reseal Joints & Patch	Fill Cracks & Patch	Fill Cracks & Patch
Corrective Maintenance	Reseal Joints Patch, Grind	1 1/2" AC Overlay	1 1/2" AC Overlay
Corrective Maintenance (High Cost)	-----	Milling, Patching 1 1/2" AC Overlay	Milling, Patching 1 1/2" AC Overlay
Rehabilitation	4" AC Overlay	2 1/2"AC Overlay	2 1/2" AC Overlay
Rehabilitation (High Cost)	5" AC Overlay	4" AC Overlay	4" AC Overlay
Major Rehabilitation	Rubblize/6" AC or Reconstruct	Reconstruct	Mill, Rubblize, 6" AC or Reconstruct

TABLE 2 SURFACE RATING SCALE

Scale Point	Surface Distress Frequency	Surface Distress Severity	Treatment Category
10	None (Recently Constructed or Rehabilitated)	None	Do Nothing
9	None	None	Do Nothing
8	Infrequent	Very Slight	Preventive Maint.
7	Infrequent to Occasional	Slight	Preventive Maint. (high cost)
6	Occasional to Frequent	Moderate	Corrective Maint.
5	Frequent	Moderate to Severe	Rehabilitation
4	Frequent	Severe	Rehabilitation (high cost)
3	Very Frequent	Very Severe	Major Rehabilitation
2	Very Frequent (Travel Difficult)	Very Severe	Major Rehabilitation
1	Very Frequent (Facility Impassable)	Very Severe	Major Rehabilitation

gional ratings. This process—called shadow scoring—showed the regional teams rated the roads consistent with the way they were trained and, most important, consistent among themselves (*11*). Total cost for the field element of the survey was approximately $100,000, which included fringe benefits and travel.

Needs Estimating

Needs estimates are an important product of a PMS. They are used in reporting to the legislature and to help the regions shape their programs of pavement projects. To achieve better estimates of pavement needs, the following tasks have been accomplished:

- Treatment matrices that link the condition information to the treatment categories were developed and computerized. Table 4 shows a matrix for overlaid pavement structures; the codes for the table follow.

Dominant Distresses	*Code*
Alligator cracking, isolated	A_i
Alligator cracking, general	A_g
Widening dropoff	W
No dominant distress	N
Not applicable	NA

TABLE 3 DOMINANT DISTRESSES FOR NEW YORK STATE PAVEMENTS

Distress	Pavement Type	Frequency Measure [1]
Faulting	Rigid	Present or Absent
Spalling (joint or mid-slab)	Rigid	Isolated or General
Alligator Cracking	Flexible or Overlaid	Isolated or General
Widening Dropoff	Overlaid	Present or Absent

[1] "Isolated" is defined as the distress symptom exists on less than 20% of the pavement section.

TABLE 4 NETWORK TREATMENT MATRIX FOR OVERLAID PAVEMENTS

Surface Rating	Dominant Distress/s	Treatment Strategy
10	NA	Do Nothing
9	NA	Do Nothing
8	NA	PM
7	N or W	PM
	A_i	PM (High Cost)
	A_g	Corrective Maintenance
6	N	Corrective Maintenance
	A_i	Corrective Maintenance (High Cost)
	W or A_g	Rehabilitation
	W and (A_i or A_g)	Rehabilitation
5	N or A_i	Rehabilitation
	W or A_g	Rehabilitation (High Cost)
	W and (A_i or A_g)	Rehabilitation (High Cost)
4	N or A_i	Rehabilitation (High Cost)
	W or A_g	Major Rehabilitation
	W and (A_i or A_g)	Major Rehabilitation
1-3	N to All	Major Rehabilitation

- The average contract costs for each of the treatments given in Table 1 were obtained from the department's Bid Analysis Management System and determined for each region and throughout the state. Costs were further stratified by lane configuration for three scenarios: pavement repair only (which is just the cost to restore the pavement), pavement plus roadside appurtenances (which includes pavement, shoulder, and guide rail repair), and the repair of all deficiencies at the candidate project site (which is the total contract cost). Table 5 shows a matrix for the total contract cost scenario.
- Computer software was developed that links the cost data to the treatment matrices. Needs estimates are now available on a statewide basis and by region, county, route, residency, or any other variable in the Sufficiency System, which is the mainframe data base that stores the department's inventory and pavement condition information.

Table 6 summarizes the results of the needs estimating process for the entire New York State highway system.

Goal Setting

Goal setting is at the heart of the department's capital program development process. Goals are used to underscore priorities, guiding the regions into developing pavement programs consistent with policies established by executive management. The goal-setting process at the statewide level starts with consideration of

- Department mission,
- State transportation requirements,
- Anticipated resource levels, and
- Existing and historical condition of the transportation system and past funding levels in support of each element of the system.

Staff from the department's Office of Planning and Program Management annually evaluate these considerations and develop tentative statewide goals for pavement and bridge condition, safety, and capacity for review by executive management. On the basis of this review, executive management establishes the statewide program emphasis, sets statewide goals, and provides each region with their tentative requirements (*12*). Because a goal must be realistic and achievable, each region has the opportunity to negotiate with executive management before the final goal statement is adopted.

Pavement goals are supplied to the regions during the early fall for use in updating the 5-year GOCP. The goal instruction package consists of a goal statement, measures of performance, and project selection criteria. Typically, the pavement goal focuses on reducing the lane miles of pavement rated poor (surface rating of 5 or less) and fair (surface rating of 6) during the annual pavement condition survey. In 1991 the regional goal statements were expanded to include a measure to ensure that priority be given to high-volume facilities.

Allocations

For goal-driven systems to be successful, resource allocations must be linked to program objectives. Historically in New York State, the allocation of pavement moneys to each region has been based on many factors, including demographics, mileage, system usage, and, to a lesser extent, pavement condition. Over the past few years, however, the allocation formulas have been revised to include an expanded pavement

TABLE 5 STATEWIDE COST ESTIMATES FOR TOTAL CONTRACT COST

Treatment Strategy	---------Lane Configuration[1]---------				
	4D	6D	2U	4U	6U
RESEAL JOINTS	$ 15	$ 18	$ 15	$ 20	$ 21
RES JNTS, PATCH SPALLS	16	20	16	21	23
RES JNTS, PAT SPALLS, GRIND	38	42	38	43	45
4" ACC OVRLY, 3" SHLDRS	268	249	300	256	241
5" ACC OVRLY, 3" SHLDRS	304	285	335	291	277
9" PCCP RECONSTRUCTION	1456	1424	1496	1429	1407
FILL CRACKS	7	9	7	10	11
FILL CRKS, PATCH PVMT	17	19	17	20	21
1.5" ACC ARMOR COAT & SHLDRS	85	75	99	77	69
1.5" OVRLY, SHLDRS, MILLING	110	99	123	101	93
2.5" ACC OVRLY & SHLDRS	128	112	148	115	103
4" ACC OVRLY & 3" SHLDRS	234	210	266	213	196
RUBBLIZE, 6" OVRLY, 3" SHLDRS	532	508	563	511	494
10.5" ACCP RECONSTRUCTION	$693	$669	$725	$673	$655

NOTE: Costs are given in thousands of dollars per lane mile.

[1] A "D" denotes a divided highway, a "U" denotes an undivided highway.

condition element that specifically addresses lane mileage of pavements rated poor and fair. Furthermore, the staff responsible for setting goals is assigned to the same functional section as the staff responsible for overseeing the allocation activity. This organization fosters the linkage between goals and the resources required by the regions to meet these goals.

Regional Development of Integrated Program

Developing a balanced program of capital and maintenance projects is the responsibility of the regional offices. Each region has established a regional program committee, chaired by the regional director, consisting of senior managers from each of the functional groups including Planning and Program Management, Design, Maintenance, and Traffic Engineering and Safety. The committee receives input from many sources beginning with the resident engineers. The REs submit lists of highway sections that are candidates for repair by either agency or contract forces. The criteria used to select candidate project sites include pavement condition ratings, technical guidelines on project selection, and the intimate knowledge and experience of the REs with their highway systems. Cri-

TABLE 6 SUMMARY OF 1990 PAVEMENT CONDITION SURVEY STATEWIDE NEEDS

TREATMENT CATEGORY	TOTAL LANE MILES	% OF TOTAL LANE MILES	ESTIMATED TOTAL REPAIR COSTS ($000's)		TOTAL CONTRACT COST
			PVMT ONLY	PVMT & FURN.	
DO NOTHING	2,769	7.5%	0	0	0
PREVENTIVE MAINTENANCE	15,201	41.3%	$ 100,073	$ 100,073	$ 169,543
CORRECTIVE MAINTENANCE	7,839	21.3%	369,923	470,840	718,369
REHABILITATION	10,664	29.0%	1,034,417	1,487,114	2,242,533
MAJOR REHABILITATION	332	0.9%	125,140	193,238	285,736
TOTAL	36,805	100.0%	$1,629,553	$2,251,265	$3,416,181

teria used by the REs to recommend who will do the work (i.e., agency or contract forces) include the scope of the repair action required, resources available for repair by agency forces, and logistics such as distance of the candidate project sites from the residency offices.

Safety considerations also play an important role in project site selection and priority action. For example, the department has recently implemented a program called SAFE-PAVE, which requires the identification and evaluation of pavement sections that are candidates for a single-course (1½-in.) overlay with higher-than-average wet-weather accident rates. If the analysis shows that an overlay will reduce accident rates, these sections receive priority for treatment.

Other sources for candidate projects include considerations particular to each region, such as improvements to corridors of statewide significance, economic development, and citizen complaints.

The number of candidate projects resulting from this process always exceeds the resources available. The department's Infrastructure Needs Assessment Model (INAM) is used to assist the regional program committee in determining the program of projects that best achieves the pavement goal given the resources available. This model calculates the cost of a user-specified pavement program and predicts the impact of the proposed program on network condition (*13*). Use of the model is an iterative process. Alternative treatment strategies are tested and the results presented to the committee. The program of projects that meets the pavement goals and best satisfies all other considerations is selected and submitted to the main office for approval.

The NYSDOT approach to program development allows the regions considerable flexibility in determining which pavement sections will be treated—as long as the pavement goals are achieved. Technical tools are used throughout the process to assist in developing a program but do not replace the collective expertise and experience of the department's regional engineering managers.

Program Review Process

This activitiy involves the main office review of the regional update of the 5-year program. All programs (pavements, bridges, safety, and capacity) are reviewed each year to ensure their compliance with the program emphasis and goals established by executive management. The program descriptive materials prepared by the regions consist of project lists, the rationale used and any trade-offs made in arriving at program choices, and summary statistics showing planned accomplishments along with forecasts of condition at the end of the program life.

Each program is compared to preestablished program evaluation criteria. The evaluation is performed by staff-level representatives from several functional groups within the main office. During this phase, the regions are kept informed of any concerns—in particular, shortcomings in goal attainment. On the basis of these concerns, the programs may be revised by the regions and resubmitted or the regions may choose not to revise the programs to reflect the concerns raised during the staff review. Each regional director is then invited to make a formal presentation (and defense) of the proposed program before a special committee consisting of executive management and chaired by the commissioner of transportation. Any unresolved concerns raised during the staff review must be addressed by the RD at this time. The presentation results in program approval or in conditional approval, which means that while the committee is in substantial agreement with the proposal, some minor issues still require discussion and resolution. After negotiations, final approval is obtained and the first-year element of the 5-year program can begin.

The pavement management function in the main office plays an important role in the review of the regional pavement programs. An analysis is conducted to determine if the programs submitted by the regions reflect a proper integration of maintenance and capital actions. In addition, INAM runs are reviewed and other analyses conducted to determine if program implementation would result in goal attainment. Figure 2 shows an example chart submitted by each region. Data from this chart are used to compute paving and treatment cycles and to assess the balance of capital and maintenance improvement. It should be noted that the categories of work listed on this chart are directly related to the general treatment categories output from the network pavement condition survey. Figure 3, prepared by the main office review team, illustrates how the proposed programs are evaluated against the network survey results. The left portion of the table summarizes the network survey results for each region; the right portion shows the project mix and costs of the proposed program. The purpose of the table is to determine whether the proposed project mix is consistent with the survey results. For example, a program would not be approved if only 10 percent of the total lane miles programmed involved preventive maintenance treatments while the network survey indicated 40 percent of a region's lane miles were candidates for preventive maintenance.

It should be recognized that, though the review process is comprehensive, the merits of individual pavement projects are not assessed. The NYSDOT network-level PMS gives the regions flexibility in project selection but demands accountability in meeting program objectives.

Program Implementation

Program implementation involves all the steps leading to the rehabilitation of a pavement section programmed for work. These activities—in particular, the selection of alternative treatments and maintenance strategies, project design, and construction—are addressed as part of the department's project-level pavement management process.

Network-Level Performance Monitoring

Performance monitoring is an essential element of any goal-oriented management system. The NYSDOT network-level PMS relies on the results of the annual pavement condition survey to evaluate the effectiveness of program implementation and to provide feedback for setting the next year's goals. From a pavement management perspective, performance monitoring also involves the evaluation of the effectiveness of pavement repair strategies and the generation of performance curves. The department is developing performance curves for each of the network treatment categories

	SFY 92/93		SFY 93/94		SFY 94/95		SFY 95/96		SFY 96/97		5-Year Total	
Treatment Category	Lane-Miles	$ M	Lane-Miles	$ M	Lane-Miles	$ M	Lane-Miles	$ M	Lane-Miles	$ M	Lane-Miles	$ M
A. PREVENTIVE MAINTENANCE (PAVING)												
Single-Course Overlay by State Forces												
From Operating Allocation												
From Capital Allocation												
Single-Course Overlay by Contract												
TOTAL PM (PAVING) ACTIONS												
B. CAPITAL PAVEMENT PROGRAM												
R&P (includes all multi-course overlays)												
Other Rehab Strategies (includes recon, full-depth recycling, CPR, etc.)												
TOTAL REHABILITATION ACTIONS												
C. OTHER PM (NON-PAVING)												
Crack/Joint Sealing/Filling by State Forces												
Crack/Joint Sealing/Filling by Contract												
Chip Seal and Slurry Seal												
TOTAL PM (NON-PAVING) ACTIONS												
TOTAL PAVING (PM & CAPITAL): A + B												

FIGURE 2 Regional reporting of 5-year integrated pavement program.

by region, pavement type, and condition rating before treatment. Figure 4 shows example performance curves for 1½-, 2-, and 2½-in. resurfacing of a full-depth asphalt pavement in the department's Hornell region. Equations for each of these curves was developed and the area under the curves determined to identify the treatment that provides the most condition years of service at the least cost. The results of these activities will furnish valuable input to regional program development—to the forecasting capability of the Infrastructure Needs Assessment Model, in particular.

PROJECT-LEVEL PAVEMENT MANAGEMENT

A project-level PMS addresses the technical aspects of selecting the engineering treatment or series of treatments to be applied to a pavement section programmed for repair. Over the past year, the NYSDOT has made great progress in implementing a project-level PMS. New technical tools have been developed and existing methodologies integrated into a systematic process for treatment selection. Specific accomplishments include the development of a detailed pavement evaluation methodology (*14*), the development of treatment selection guidelines based on life-cycle cost considerations (*15*), and the preparation of engineering instructions to guide designers in selecting appropriate treatments. Full-scale implementation of the project-level system is scheduled for early 1992.

CONCLUSIONS

A pavement management system must be tailored to the organizational structure and decision-making processes of the implementing agency. The New York State network-level PMS was designed to operate in a decentralized decision-making environment where choices on project selection are made by experienced engineers in regional offices. The system is goal-driven, and although it allows flexibility in individual project selection, it requires accountability in overall program development. The authors acknowledge that the system described in this paper may not be appropriate for all highway

	1991 Sufficiency Survey		Proposed 5-Year Program			
Treatment Category	Lane-Miles	% of System	Lane-Miles	% of Program	% LM Treated To LM Needs	Programmed ($ millions)
Do Nothing						
Preventive Maintenance						
Preventive Maintenance (Paving)						
Rehabilitation						
Major Rehabilitation						
Totals						

FIGURE 3 GOCP review comparison table.

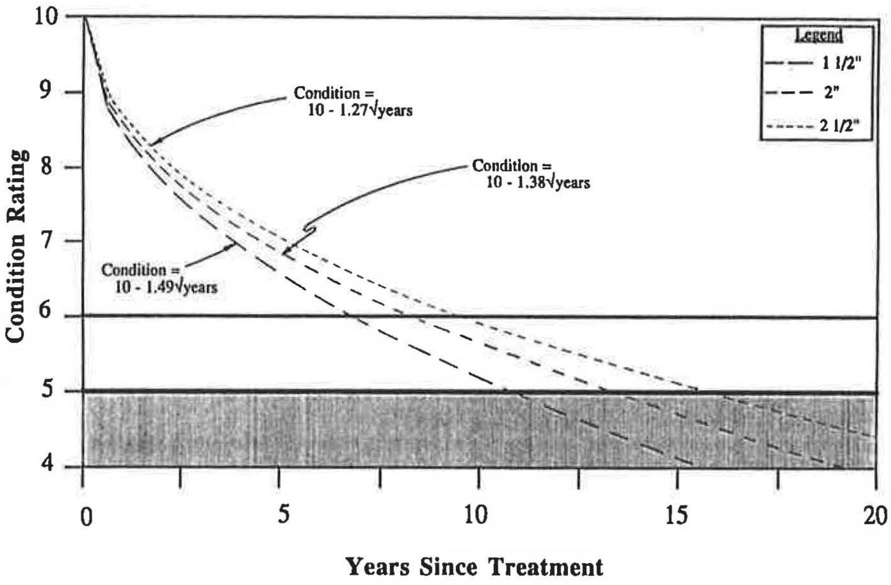

FIGURE 4 Performance curves for alternative treatments of flexible pavement.

agencies, but the principles of a goal-oriented approach to managing pavements are transportable and should be considered by organizations currently in the process of developing a network-level system.

ACKNOWLEDGMENTS

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Life-Cycle Cost Versus Network Analysis

EDWIN C. NOVAK, JR., AND WEN-HOU KUO

Pavement management systems (PMSs) are typically designed to select projects and treatments on the basis of which alternatives have the lowest project life-cycle cost. Ways to use project life-cycle cost analysis to increase the total cost of network preservation are illustrated. In addition, it is not a handy PMS tool for policy makers to use to spend available funds more efficiently. It is proposed that the policy level use network life-cycle cost analysis to minimize the total cost of network preservation. Economic analysis would then be a three-step process: network life-cycle cost analysis, to establish program development constraints that minimize the total cost of preservation; program analysis, to select the combination of projects and treatments that meet policy constraints and maximize program benefits; and engineering analysis, to minimize project cost. Network life-cycle cost analysis is based on the remaining service life and strategy analysis concepts, which are not in wide use. Therefore, these methods are explained briefly. Conceptually, network and project life-cycle cost analysis are similar in that for network analysis, the lane-mile length of each alternative program is used in place of each alternative project, and each alternative program's average design service life is substituted for alternative project treatments.

FHWA's latest pavement policy (*1*) requires economic analysis (life-cycle cost) to be taken into account when maintenance, rehabilitation, and reconstruction (MR&R) alternatives are selected. To comply with this policy statement, agencies typically use project life-cycle cost (LCC) to select MR&R treatments for proposed MR&R projects. The general concept is that by selecting the lowest LCC treatment for each proposed project and then by selecting the optimal combination of proposed projects, the agency and FHWA are ensured that the total long-term cost of preservation is minimized.

The FHWA policy on economic analysis goes on by asking agencies to weigh LCC results against the needs of the entire system. It explains that available funds may not permit selection of the lowest LCC treatment and that investments in projects must be timely to avoid more costly repairs in the future. These factors should be taken into account when developing MR&R programs, but there is little guidance about how they can be objectively accomplished. Nevertheless, the FHWA shows concern for network-level considerations when selecting treatments and recognizes that what is best for the project may not be best for the network.

This paper proposes that the relationship between total long-term cost of network preservation and performance has the highest priority in the process of economic analysis of alternatives. That is, the LCC of preserving networks is of first-order importance, and the LCC of preserving projects is of third-order importance. Maximizing program benefits is considered to be of second-order importance.

When LCC analysis is applied to networks instead of projects, the following two conceptual changes are necessary:

1. Lane-mile length of alternative MR&R programs is substituted for projects.
2. The average design service life (ADSL) of alternative MR&R programs is substituted for MR&R treatments.

The purpose of network LCC analysis is to establish the MR&R program development constraints needed to guide program development so that it will achieve long-term network condition and funding goals at minimum total cost. Only the simplest form of network LCC analysis is presented to illustrate methodology. Network LCC analysis should be an attractive form of economic analysis because it is a policy-level tool that provides for top-down decision making yet it is easy to understand and to display results for the the consideration of many alternative funding and network condition schemes. And using network LCC analysis can substantially reduce the total cost of network preservation compared to that possible with project LCC analysis. Network LCC analysis is based on remaining service life (RSL) presented by Baladi et al. (*2*) and illustrated in Figure 1. Definitions of the terms used in this paper are defined in another paper by Novak and Kuo in this Record.

BASIC NETWORK LCC ANALYSIS CONCEPTS

The performance of projects, networks, and MR&R programs or strategies are all characterized by their lane-mile length and RSL or design service life (DSL). RSL and DSL are the same at the time of construction. With time, condition

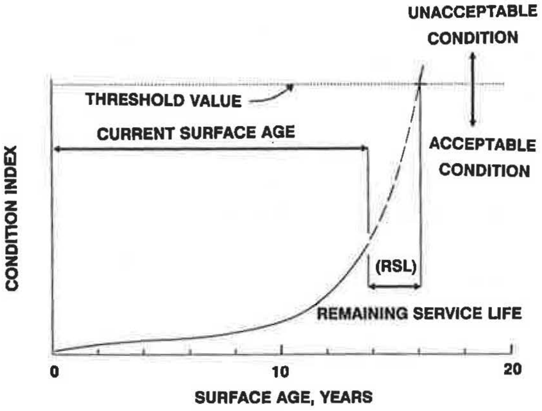

FIGURE 1 Conceptual diagram of remaining service life.

Michigan Department of Transportation, Materials and Technology Division, Lansing, Mich. 48909.

deteriorates and RSL decreases. When condition reaches the threshold value defining acceptable condition, its RSL is zero. The performance of networks and MR&R programs is based on the average RSL (ARSL) or ADSL of the projects they are made up of. For networks, the sections of pavement of most concern are those in unacceptable condition. They make up the majority of projects considered for annual MR&R programs. Figure 2 illustrates the network rehabilitation process (based on RSL). Network performance expressed in terms of RSL enables an accounting process to be used to keep track of the rate at which projects or uniform sections deteriorate from each higher to each lower RSL category and the rate they are rehabilitated out of lower RSL categories. It also keeps track of which higher RSL category the designer's estimate of DSL would place the projects or uniform sections.

Relationship Between Network Performance and MR&R Strategy

The condition of a network is simply the percentage of network having an RSL of zero, which is the same as the percentage of network in unacceptable condition. Network condition is a function of the network's rate of deterioration and the annual MR&R program. For network analysis, it is beneficial to deal with MR&R strategies instead of MR&R programs. MR&R strategy is defined as the percentage of network to be annually rehabilitated from each lower to each higher RSL category. For convenience, MR&R strategies can be generalized to percentage of network preserved annually and its average DSL. The relationship between network condition (at equilibrium) and the generalized form of MR&R strategy is as follows:

$$P_0 = 100 \text{ percent} - (P \times \text{ADSL}) \qquad (1)$$

where P_0 is the network condition and P is the percentage of network annually preserved.

If the MR&R strategy is to preserve 4 percent of the network's length and its ADSL is 20 years, 80 percent of network would be in acceptable condition. Assumptions are that ADSL estimates are accurate, the MR&R strategy of 4 percent of the network and ADSL of 20 years is used annually, and enough time has elapsed for the network to reach equilibrium.

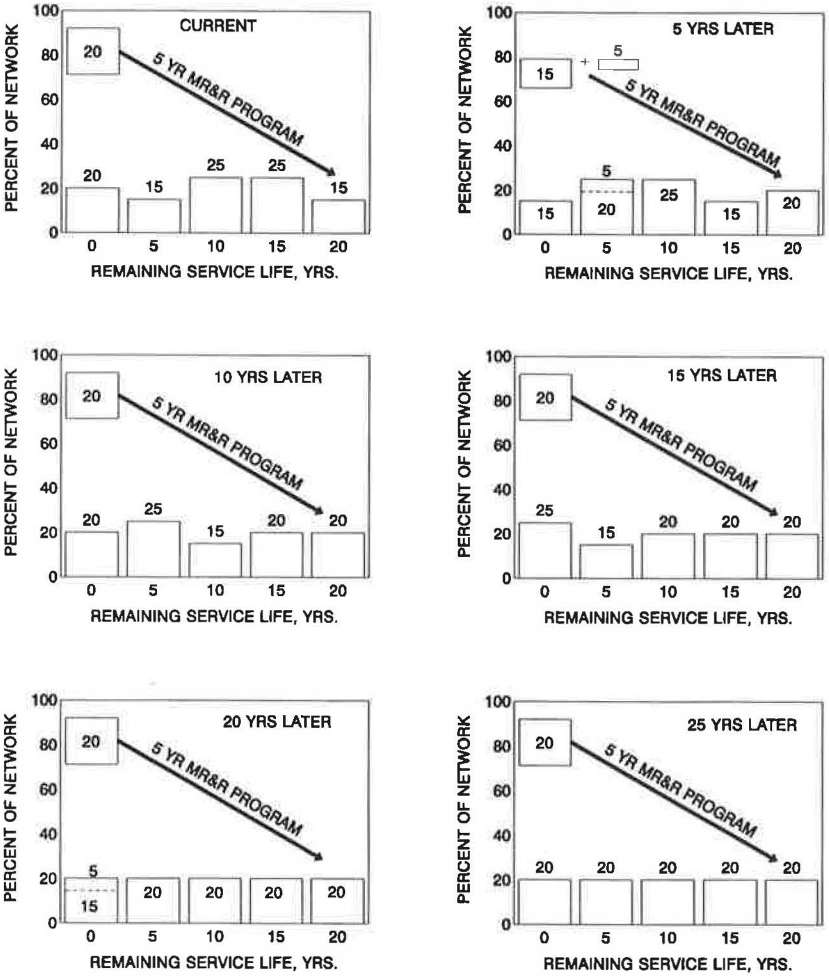

FIGURE 2 Illustration of network performance and rehabilitation processes (5-year program: ADSL = 20 years, % network = 20).

The network's ARSL is calculated as follows:

$$\text{Network ARSL} = \sum X_i Y_i / 100 \qquad (2)$$

where X_i is the RSL of the ith uniform section and Y_i is the percentage of network in the ith uniform section.

This calculation is the same as taking moments about the zero RSL category. On the basis of Equation 2, it can be seen that the ADSL of the MR&R strategy is directly related to the network's ARSL.

Cost of Alternative MR&R Programs

The MR&R strategy provides the lane-mile length of projects to be designed into each RSL category. A simple cost matrix based on historical MR&R program cost data provides the average lane-mile cost that corresponds to the DSL of the designated networks previously constructed projects. The cost of alternative programs is simply the project of its lane-mile length and the appropriate cost per lane mile. Figure 3 shows a simple cost matrix based on historical project cost data.

Annual or 5-year MR&R program cost estimates are based on the MR&R strategy that would be used as a constraint for MR&R program development and the lane-mile cost data shown in Figure 3. Annual MR&R program cost estimates are based on the following equation:

$$\text{MR\&R program cost} = P/100 \times L \times C_x \qquad (3)$$

where L is the lane-mile length of the network and C_x is the lane-mile cost of the designated DSL category.

Reactive Maintenance Cost

The cost of reactive maintenance is based on procedures reported by Richardson (*3*). Simply, it is the product of the lane miles of pavement in unacceptable condition and the historical cost of reactive maintenance per lane mile of pavement in unacceptable condition.

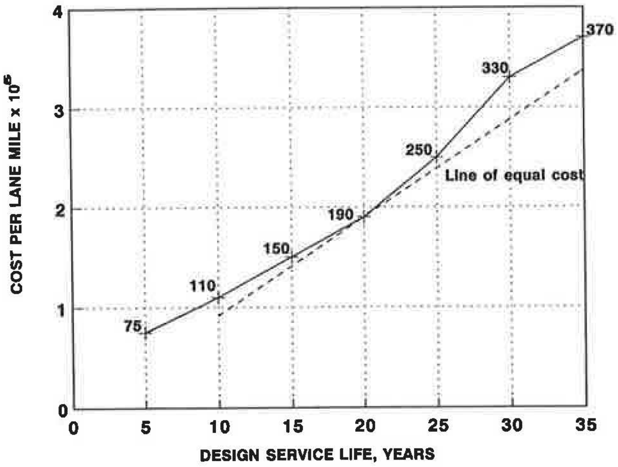

FIGURE 3 **Simple cost matrix based on historical as-built MR&R project cost data.**

Annual reactive maintenance cost ($RMC) is computed on the basis of the following equation:

$$\$RMC = [P_0 + (P_5 - P)/2]/100 \times L \times C_x \qquad (4)$$

where P_5 is the percentage of network that annually deteriorates into the zero RSL category and C_x is the network's reactive maintenance cost per lane mile.

Effects of Inflation

An objective of network LCC analysis is to provide administrators with the actual long-term cost of annual MR&R programs and reactive maintenance. These costs, when compared with anticipated revenues, should include the effects of the cost of inflation. When the costs of MR&R treatments are expected to increase with each year of delayed action, treatments with high initial cost and long life tend to provide lowest network LCC. For project LCC analysis that discounts money, investment in short-life treatments of low initial cost tend to have lower project LCC

NETWORK LIFE-CYCLE COST

Network LCC is the sum of the annual preservation cost that is accumulated over the LCC analysis period. This annual cost is computed on the basis of the annual cost of reactive maintenance plus the annual cost of MR&R programs. Network LCC is the same as the total cost of network preservation over the analysis period. To illustrate network LCC analysis, the following information is assumed:

1. The RSL of each uniform section that makes up the network is available and summarized as shown in Figure 4.
2. The assumed cost per lane mile to move (by MR&R treatment) pavements from any lower RSL category to any higher RSL category is shown in Figure 3.
3. The length of the network is assumed to be 1,000 lane-mi.
4. The annual cost per lane mile for reactive maintenance of pavements in unacceptable condition is assumed to be $2,500/lane-mi.

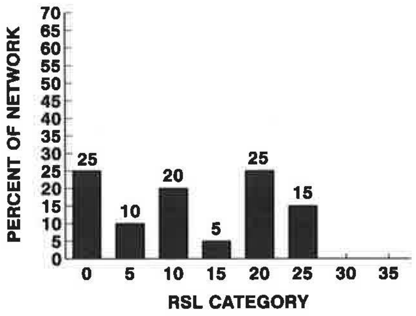

FIGURE 4 **Initial network performance used as example of network life-cycle cost analysis.**

Network LCC is based on the need of an agency to minimize the total cost of pavement preservation and the need to control the relationship between cost of preservation and the network's condition over long periods of time. An analysis period of 40 years is used to insure that the network's condition and annual preservation cost have stabilized. It is assumed that funding level and size of the annual MR&R program are to be as consistent from year to year as possible.

Network Life-Cycle Cost Analysis

It is assumed that the network whose performance is shown in Figure 4 is to be improved so that zero percent of it will be in unacceptable condition at the end of 5 years, and this condition is to be maintained for the 40-year analysis period. The objective is to minimize annual MR&R program cost as well as network LCC. To do this, the lowest-cost MR&R strategies are to be used. Figure 3 indicates that the lowest cost per lane mile DSL is 20 years and that the 10-, 15-, and 25-year categories have only slightly higher cost.

Figure 4 illustrates that 25 percent of the network is currently in unacceptable condition, and 10 percent will become unacceptable within 5 years. Therefore, 35 percent of the network must be moved out of the zero RSL category in the first 5 years, as Figure 5 illustrates. The percentage of network in each RSL category at the end of each 5-year analysis period is computed as the sum of the percentage of network rehabilitated into each category plus the percentage of network that deteriorates into it from the next higher RSL category. On the basis of Equation 3 and the lane-mile cost data shown in Figure 3, the cost of the MR&R program for the first 5 years is given in Table 1.

The cost of reactive maintenance must be added to the cost of the MR&R program to determine the total cost of preservation. Based on Equation 2 and a cost of $2,500/lane-mi, the cost of reactive maintenance during each year of the first 5-year periods is given in Table 2.

Figure 5 indicates the performance of the network at the end of 5 years. Because 20 percent of the network will deteriorate into the zero RSL category in the 5- to 10-year analysis period, it is necessary to rehabilitate 20 percent of the network out of the same category to meet the network condition objective of zero percent of the network in unacceptable condition. Figure 6 illustrates the MR&R strategy selected. The estimated cost of reactive maintenance is zero; the MR&R program cost (when calculated as for the first 5 years) is $48,000,000.

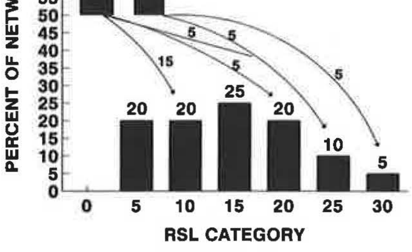

FIGURE 5 Performance of network after 5 years and MR&R strategy used to preserve it.

Figure 6 shows the performance of the network at the end of 10 years. Because 20 percent of the network will deteriorate into the zero RSL category in the next 10- to 15-year period, it is necessary to rehabilitate 20 percent of the network out of this category to meet the network condition objective. Figure 7 shows that the MR&R strategy selected is the same as for a 5- to 10-year period, so the total estimated cost of preservation for a 10- to 15-year period is $48,000,000.

Figure 7 illustrates the performance of the network at the end of 15 years. Because 25 percent of the network will deteriorate into the zero RSL category in the 15- to 20-year analysis period, 25 percent of the network must be rehabilitated. Figure 8 illustrates the MR&R strategy selected. The estimated cost of reactive maintenance is zero, and the MR&R program cost is $60,500,000.

Figure 8 presents the performance at the end of 20 years. Because 20 percent of the network will deteriorate into the zero RSL category in the 20- to 25-year analysis period, 20 percent of the network must be rehabilitated. Figure 9 illustrates the MR&R strategy selected. The estimated cost of reactive maintenance is zero, and the MR&R program cost is $47,000,000.

Figure 9 presents the performance of the network at the end of 25 years. The network's performance is now stable if the same MR&R strategy is used from this point on. Hence, the MR&R program cost of all future 5-year MR&R programs should be the same as the cost for the 20- to 25-year period. The estimated cost of reactive maintenance is zero, and the total cost of MR&R programs for the three 5-year periods between Years 25 and 40 is $141,000,000.

The total 40-year netowrk LCC is given in Table 3.

MR&R Program Development Constraints

If this network LCC analysis were to be accepted by policy makers, the MR&R strategy and estimated cost would become funding and MR&R program development constraints. That is, those responsible for program development would be required to select projects and treatments whose lane-mile length and ADSL meet or exceed MR&R strategy constraints and whose cost is equal to or less than the funding constraint. Policy makers are responsible for the first level of economic analysis (minimize network preservation cost). The program development process is then responsible for the second (maximize program benefits) and third levels (minimize project cost).

PROJECT LIFE-CYCLE COST

The Michigan Department of Transportation (DOT) uses a simplified project LCC analysis procedure based on the Minnesota DOT's method of pavement selection. For project LCC estimates, Michigan considers five alternative rehabilitation and two alternative reconstruction treatments. The major maintenance schedule and descriptions of each rehabilitation and reconstruction alternative are shown in Figures 10

TABLE 1 COST OF MR&R PROGRAM FOR FIRST 5 YEARS

DSL Cat.	No. Per.		$P/100$		L		C_x	=	$MR&R Program
10	1	×	.15	×	1,000	×	$110,000	=	$16,500,000
20	1	×	.05	×	1,000	×	$190,000	=	9,500,000
25	1	×	.10	×	1,000	×	$250,000	=	25,000,000
30	1	×	.05	×	1,000	×	$330,000	=	16,500,000
									$67,500,000

TABLE 2 COST OF REACTIVE MAINTENANCE FOR FIRST 5 YEARS

Year	$[P_0 + (P_5 - .2P)/2]/100$		L		C_x		$RMC
1	[.25 + (.02 - .07)/2]	×	1,000	×	2,500	=	$ 562,000
2	[.20 + (.02 - .07)/2]	×	1,000	×	2,500	=	438,000
3	[.15 + (.02 - .07)/2]	×	1,000	×	2,500	=	312,000
4	[.10 + (.02 - .07)/2]	×	1,000	×	2,500	=	188,000
5	[.05 + (.02 - .07)/2]	×	1,000	×	2,500	=	62,000
							$1,562,000

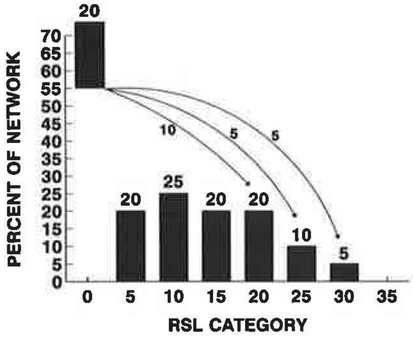

FIGURE 6 Performance of network at end of 10 years and 5-year MR&R strategy used to preserve it.

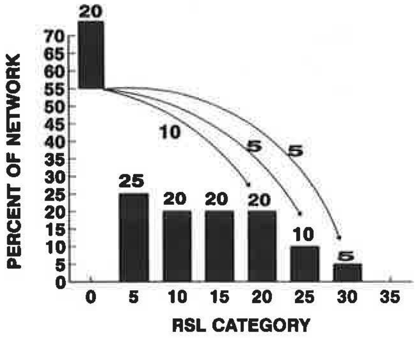

FIGURE 7 Performance of network at end of 15 years and 5-year MR&R strategy used to preserve it.

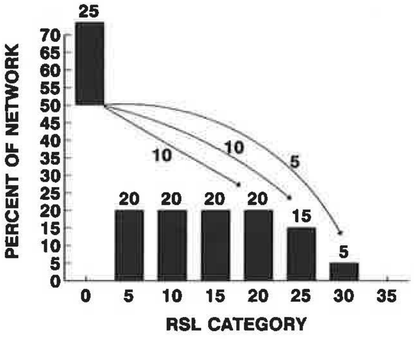

FIGURE 8 Performance of network at end of 20 years and 5-year MR&R strategy used to preserve it.

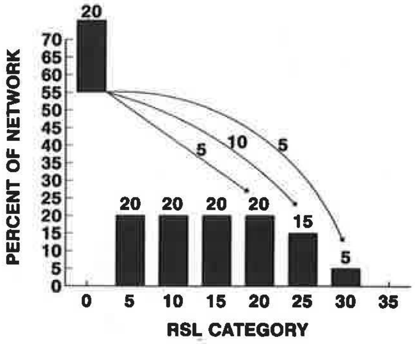

FIGURE 9 Network performance at end of 25 years and 5-year MR&R strategy used to preserve it.

TABLE 3 40-YEAR NETWORK LCC

Time Period (yr)	$RMC		$MR&R Program Cost		$Total Pres. Cost
0 to 5	$1,562,000	+	$ 67,500,000	=	$ 69,062,000
5 to 10	-0-	+	48,000,000	=	48,000,000
10 to 15	-0-	+	48,000,000	=	48,000,000
15 to 20	-0-	+	60,500,000	=	60,500,000
20 to 25	-0-	+	47,000,000	=	47,000,000
25 to 40	-0-	+	141,000,000	=	141,000,000
					$413,250,000

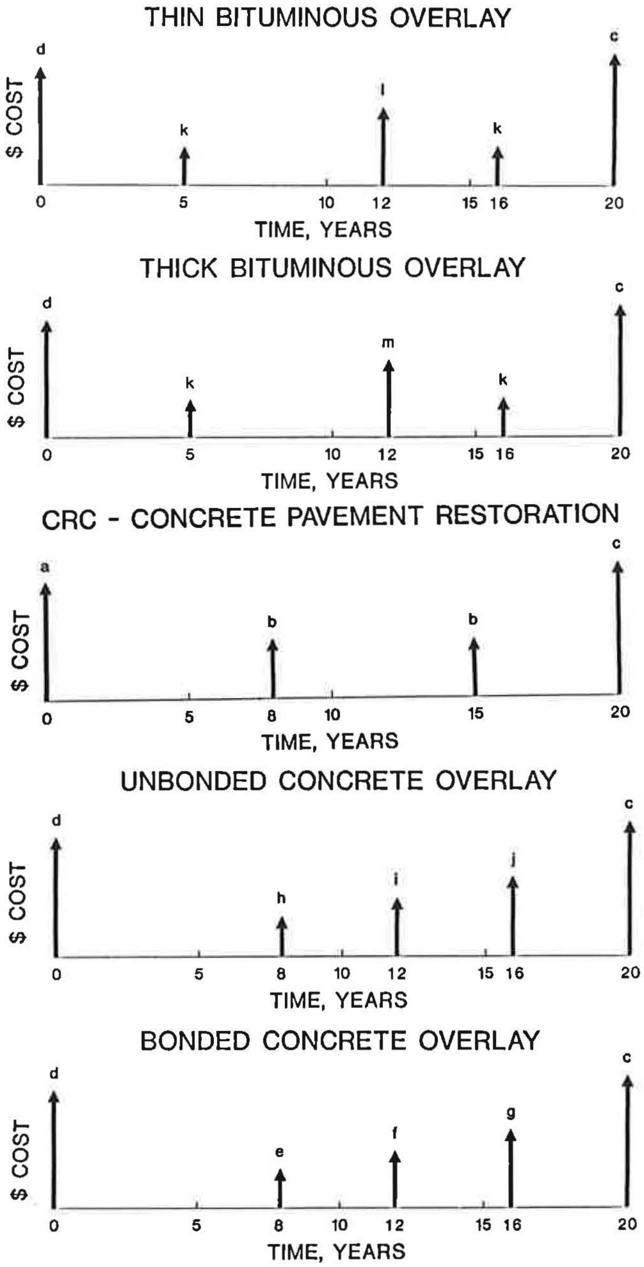

FIGURE 10 Major maintenance schedule for rehabilitation treatments used for project life-cycle cost analysis.

and 11, respectively. The treatments are described in Tables 4 and 5. Selection of the best MR&R treatment is based on the total discounted cost per mile. This cost estimate assumes that all five rehabilitation treatments provide 20 years of extended life and that both reconstruction treatments provide 35 years of service life. Project LCC analysis is based on the cost of the scheduled maintenance, and it is discounted at the annual rate of 4.5 percent. To simplify analysis, factors such as current and future network condition, cause and rate of deterioration, traffic load, salvage value, agency and user savings, user cost, administrative cost, and reactive maintenance cost are not considered.

The basis for using project LCC analysis is that alternative pavement types require different interim improvement expenditure at different points along the project lifetime scale to keep them serviceable; the incremental costs for pavement type must be accumulated in a way that keeps cost truly comparable. The purpose of project LCC is to combine the cost of the initial investment with the present value of future contract maintenance expenditures. Because the department has no data to indicate how maintenance expenditures are allo-

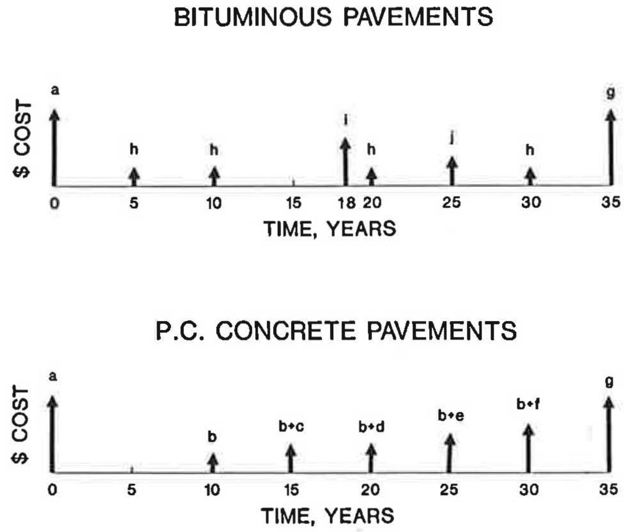

FIGURE 11 Major maintenance schedule for reconstruction treatments used for project life-cycle cost analysis.

TABLE 4 DESCRIPTION OF TREATMENTS, FIGURE 10

Letter	Description of Treatments
a	Repair cracks, replace joints, replace old patches, grinding, replace seals, subbase underdrains, and undersealing
b	Replace joints, grinding, replace seals, repair cracks
c	Reconstruct/rehabilitate
d	Initial rehabilitation
e	Replace 10% of seals and 15% of joints
f	Replace 10% of seals, 20% of joints; repair 100% of cracks
g	Replace 10% of seals and 10% of joints
h	Replace 10% of seals
i	Replace 10% of seals, 5% of joints; repair cracks
j	Replace 10% of seals, 5% of joints
k	Crack fill 3,000 ft
l	Remove 440 psy, replace 440 psy, replace 20% of joints
m	Remove 330 psy, replace 330 psy, replace 10% of joints

NOTE: All treatments include cost to maintain traffic.

cated to projects over their normal life cycle, Figures 10 and 11 are the hypothetical timing and extent of maintenance treatments that the Michigan DOT uses.

Project Life-Cycle Cost Analysis

A frequent concern is whether to rehabilitate or reconstruct. Assuming the proposed project is a rigid pavement, the thick overlay option from Figure 10 is compared with the rigid reconstruction option from Figure 11.

TABLE 5 DESCRIPTION OF TREATMENTS, FIGURE 11

Letter	Description of Treatments
a	Initial construction
b	Replace seals
c	Replace 5% of joints
d	Replace 10% of joints
e	Replace 15% of joints
f	Replace 20% of joints
g	Rehabilitate/reconstruct
h	Crack fill 3,000 ft
i	Mill 4.0 in.; recycle 130T, 140L, 170B
j	170 psy overlay

NOTE: All treatments include cost to maintain traffic.

Figures 12 and 13 show estimated initial costs and major maintenance costs used for the rehabilitation and reconstruction options. Assuming a discount rate of 4.5 percent and an analysis period of 40 years, the rehabilitation project LCC is $281,600 and the reconstruction project LCC is $370,700.

On the basis of these results, the rehabilitation alternative would be selected because it is has the lowest project LCC. Economic analysis based on project LCC would have been completed at this point. Any further consideration of this project will include only the rehabilitation treatment selected by the project LCC method.

Impact of Project LCC on Total Cost of Network Preservation

The simplest way to look at the impact of alternative treatments on network LCC is to assume all preservation projects are rehabilitation projects that have the expense stream shown in Figure 12. It is assumed that each rehabilitation project will reach unacceptable condition at the end of its 20-year extended life and that network condition objectives are to eliminate all pavement in unacceptable condition. To meet this network condition objective, 100 percent of the network must be rehabilitated every 20 years, or 5 percent yearly. Assuming a network length of 1,000 lane-mi, 50 lane-mi must be annually rehabilitated at a cost of $150,000/lane-mi for an annual program cost of $7,500,000. After 5 years, the program cost would increase by $5,000/lane-mi for 50 lane-mi so the 5-year-old projects could receive their scheduled major maintenance. This would increase annual program cost by $250,000, so total annual preservation cost would be $7,750,000. After 12 years, the program cost would increase by $70,000/lane-

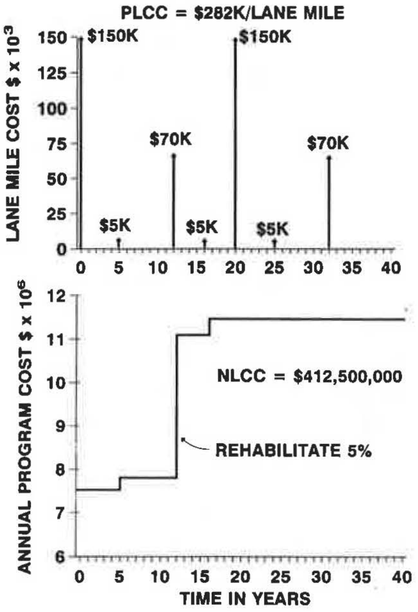

FIGURE 12 **Cost of major maintenance scheduled for project's rehabilitation and its life-cycle cost (*top*); and annual MR&R program cost and network life-cycle cost if all projects programmed use same rehabilitation treatment (*bottom*).**

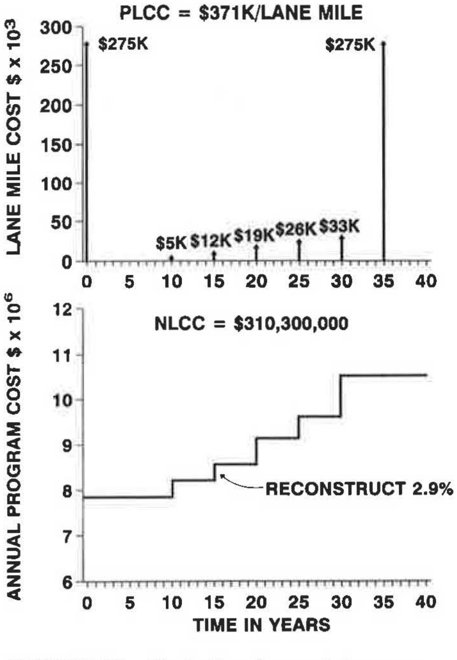

FIGURE 13 **Cost of major maintenance scheduled for project's reconstruction and its life-cycle cost (*top*); and annual MR&R program cost and network life-cycle cost if all projects programmed use same reconstruction treatment (*bottom*).**

mi for 50 lane-mi so the 12-year-old projects could receive their scheduled major maintenance. This would increase annual program cost by $3,500,000; total annual preservation cost would be $11,250,000. After 16 years, the program cost would increase again by $5,000/lane-mi for 50 lane-mi so that the 16-year-old projects could receive their scheduled major maintenance. This would increase annual program cost by $250,000, and total annual program cost would be $11,500,000. After 16 years, the program cost would be stable. These results are shown in the lower half of Figure 13. The network LCC is the sum of the total annual preservation programs during the 40-year analysis period. The network LCC of the rehabilitation alternative is $412,500,000.

This same procedure was used for the reconstruction option, which rehabilitates 100 percent of the network every 35 years, or 2.9 percent of the network annually (Figure 13). Again assuming a network 1,000 lane-mi long, 29 lane-mi must be reconstructed annually to meet the network condition objective. The initial program cost would be $7,975,000, and it would increase after 10, 15, 20, 25, and 30 years as scheduled major maintenance is conducted. Total annual program cost would stabilize at $10,673,000, as shown in the lower half of Figure 13. The network LCC for the reconstruction option is $310,000,000.

It is interesting to see that the results of project LCC and network LCC analysis are opposite. The rehabilitation treatment's LCC (based on Michigan's method) is 32 percent less than that of the reconstruction treatment. However, the cost to preserve the network over 40 years is 33 percent less if all projects are reconstructed rather than rehabilitated. This difference would have been even greater if the effect of inflation on cost of MR&R treatments were included in the analysis. Another variable is network condition over the 40-year analysis period. Annual programs consisting of all rehabilitation projects will reach the target network condition in 20 years; programs consisting of all reconstruction projects will require 35 years to reach the same condition objective.

SUMMARY

This paper illustrates how assessing network performance in terms of RSL enables life-cycle costing to be applied to alternative MR&R programs (MR&R strategies) in place of alternative MR&R treatments. This in turn enables analysis of the LCC of preserving networks, not projects. The advantages of network LCC analysis are that it enables policy makers to

1. Minimize total cost of network preservation.
2. Control future cost and condition of networks.
3. Control MR&R program development by specifying the following program development constraints: MR&R strategy, funding level, and rank order of MR&R program benefits.
4. Estimate the stream of annual preservation program expenses over 40 or more years.
5. Monitor the effectiveness of MR&R program development staffs.

Network LCC analysis requires complete, high-quality pavement condition data to estimate cost of preventive maintenance and repair alternatives, to determine cause and rate of deterioration, to determine current condition, to estimate the DSL of alternative MR&R treatments, and to estimate project benefits such as ride-quality improvement. These data are determined by processing pavement condition data through PMS application software that performs project-level analysis on all uniform sections within the network (*4*).

Project LCC analysis eliminates all but one alternative MR&R treatment for network-level analysis. And an example is given to show that the lowest project LCC alternative can have a much higher total cost of network preservation than an alternative that has a higher project LCC. The PMS developed for the Michigan DOT calculates the cost, DSL, and benefits for each of 30 to 40 alternative MR&R treatments (depending on pavement type) for each of the networks' uniform sections. This pool of information provides a better chance to optimize than does the use of one alternative treatment per proposed project. Network LCC based on the most cost-effective MR&R opportunities in the entire network can greatly improve funding efficiency.

CONCLUSIONS

1. Project LCC analysis does not account for the impact of alternative MR&R treatments on network performance. Therefore, it is a valid method of economic analysis only when all treatments have the same DSL.
2. Treatments selected on the basis of lowest project LCC analysis will not necessarily preserve networks at lowest total cost.
3. It should be difficult for agencies to minimize the total long-term cost of preservation and control long-term condition and funding requirements if MR&R projects and treatments are selected on the basis of project LCC
4. Network LCC analysis gives policy makers an analysis tool needed to control future network condition and funding requirements at minimum total cost of preservation.
5. Network LCC analysis requires its users to establish threshold values for acceptable condition and collect complete and high-quality pavement condition data.
6. Project life-cycle costing has the advantage that it is considered an acceptable method for selecting alternative MR&R treatments regardless of completeness of analysis, subjectivity of definitions, consideration of total cost of network preservation, and consideration of network performance.
7. Network LCC provides for three levels of economic analysis. The first level is network life-cycle cost analysis, whose purpose is to minimize (for a given level of annual funding or network condition) the total cost of pavement preservation. The second level is MR&R program analysis, whose objective is to maximize program benefits. The third level is engineering analysis of projects, whose objective is to minimize project cost.
8. The use of project LCC analysis favors investment in MR&R treatments having lower initial cost and short life; use of network LCC analysis favors investment in treatments having higher initial cost and long life.

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Determination of Pavement Distress Index for Pavement Management

D. A. GRIVAS, B. C. SCHULTZ, AND C. A. WAITE

A methodology is presented for determining a pavement distress index (PDI) needed for pavement management purposes. It involves a Delphi-like process for the acquisition of expert opinion through a series of questionnaires and the derivation of weighted average condition measures. Emphasis is placed on making the methodology useful for a wide range of pavement preservation decision-making purposes. The index formulation is based on two types of information, namely, (a) individual distress ratings along nominal lengths of pavement, and (b) a set of weighting values associated with the various distress types and severity-extent combinations. The PDI is used as a condition measure in various other analytical methodologies within the pavement management system of the New York State Thruway Authority. Important aspects of the methodology are discussed and the index calculation technique is demonstrated in an illustrative example. It is concluded that the developed index is a viable single measure of pavement surface condition useful for pavement management purposes.

TABLE 1 POSSIBLE DISTRESS STATES (1990)

Distress Type	Valid Ratings
Asphalt Pavement (Overlaid)	
Centerline cracking	N, SL, SG, ML, MG, LL, LG, TL, TG
Other types of lane cracking	N, SL, SG, ML, MG, LL, LG, TL, TG
Surface defects	N, SL, SG, ML, MG, LL, LG
Rutting	N, LL, LG, TL, TG
Transverse cracking	N, SL, SG, ML, MG, LL, LG, TL, TG
Edge cracking	N, SL, SG, ML, MG, LL, LG, TL, TG
Shoulder	
Shoulder defects	N, SL, SG, ML, MG, LL, LG
Lane/shoulder displacement	N, SL, SG, ML, MG, LL, LG
Concrete Pavement (Original)	
Loss of transverse joint seal	N, LL, LG
Transverse joint spalling	N, SL, SG, ML, MG, LL, LG, TL, TG
Transeverse joint faulting	N, LL, LG
Longitudinal joint spalling	N, SL, SG, ML, MG, LL, LG, TL, TG
Slab surface defects	N, SL, SG, ML, MG, LL, LG, TL, TG
Slab cracking	N, SL, SG, ML, MG, LL, LG, TL, TG

The New York State Thruway Authority (NYSTA) and Rensselaer Polytechnic Institute (RPI) are cooperating to develop a pavement management system (PMS) for the authority's network, which consists of 641 centerline-mi (2,763 lane-mi) of Interstate-type highway. More than 90 percent of the network is composed of asphalt overlay pavement (AC). The rest is primarily the original jointed mesh–reinforced portland cement concrete (PCC). Throughout the system, shoulder surfaces are built of asphalt cement concrete.

Since 1989, NYSTA's PMS distress survey has been applied to the system annually. The survey technique (*1*) involves three personnel making visual distress estimates of the driving lane and shoulder from a vehicle driven on the shoulder at slow speeds. The distress types measured vary with pavement type. This intensive data collection activity results in eight distress ratings for each ⅒ mi of road surveyed. Table 1 summarizes the distress types and their possible ratings. The ratings are coded to represent linguistic assessments of distress severity and extent. The first letter S,M,L, or T denotes small, medium, large, or total level of severity, respectively; the second letter G or L denotes general or local extent. Thus, rating ML indicates medium severity and local extent, and rating TG indicates total severity and general extent. The code N stands for no distress.

Data collected by the distress survey represent measures of specific pavement surface features taken at regular intervals. The availability of such detailed data is essential for many pavement management tasks, but there is also the need to characterize pavement distress condition in a more aggregate manner. This is accomplished by combining individual distress data into indexes that summarize the condition of each pavement segment or project. Thus, distresses reflecting the condition of a specific pavement component such as slab, joint, shoulder, or an entire lane are combined into indexes descriptive of the condition of the specific component. The resulting indexes are referred to as slab distress index, joint distress index, shoulder distress index, and lane distress index. Consideration of all distresses on a given pavement segment produces a single index called the pavement distress index (PDI).

OBJECTIVES

The availability of an appropriate PDI is considered an important requirement of NYSTA's PMS. Specific objectives in developing the PDI are to (a) combine distress data in a manner that reflects NYSTA maintenance practices and that is meaningful to field personnel and middle and upper management, and (b) create a sufficiently responsive condition measure that can be used for network-level analysis.

Like the PMS itself, the PDI is developed through a staged process. Desirable early products included tabular and graph-

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ical summaries of the surface condition of defined pavement segments. Other uses of PDI include the following:

- Monitor pavement surface condition over time.
- Define uniform condition sections for project-level analysis.
- Compare condition of candidate projects.
- Assist in project priority ranking for budgeting purposes.
- Conduct correlation analysis with other engineering parameters.

METHODOLOGY

It is well recognized that each individual distress type contributes in a distinct manner toward the aggregate pavement condition. For each distress type, relative severities are not equivalent (e.g., the difference between small and medium transverse crack may not be the same as the difference between small and medium surface defect). Thus, determination of an overall distress index must accommodate the relative significance of each distress type and magnitude (severity and extent).

The approach followed in this study to calculate PDI values uses weights determined on the basis of expert opinions. In general, each weight value represents the importance that maintenance personnel give to the task of correcting a specific pavement deficiency identified through surface distresses. This approach also enabled the capture of existing maintenance practices and the use of generated information to improve consistency of judgments throughout the network.

Use of Expert Opinion

Opinions were solicited from experienced maintenance personnel using a technique derived from the well-known Delphi method (*2*). The applied technique involved mainly a series of questionnaires, to which responses were solicited anonymously so that conformity pressure and individual domination would be minimized.

The task of soliciting expert opinion was accomplished through a series of three questionnaires. These involved multiple-choice questions, a modification of the "traditional" open-ended Delphi format. This modification helped expedite the completion of the questionnaires and facilitate quantification of responses.

The first questionnaire was distributed to 25 agency-designated pavement maintenance experts. Questions focused on repair priorities for isolated distress states; an example of the questions might be this: "When deciding on the need for maintenance work, how much importance do you assign to alligatored edge cracks?" Participants were asked to choose one of the following responses:

- Condition does not warrant repair;
- Very low priority repair;
- Low-priority repair;
- Medium-priority repair;
- High-priority repair; and
- Condition is critical; repair is of highest priority.

Questions about the relative significance of roadway components (e.g., concrete slab versus joint) and of the various distress types were phrased in a similar manner. Information on current maintenance practices was also collected. Generally, opinions expressed by maintenance personnel tended to confirm the severity progression of the distress scales.

After completion of the initial study, 9 personnel were selected from the original pool of 25 to participate in further refinement of the responses, as well as in other knowledge-acquisition activities. Logistics necessitated the reduction in the original number of participants.

In the second questionnaire, the nine participants' original responses to each question were summarized graphically. Participants were asked to review the group response and indicate whether they agreed with the majority opinion. Those who disagreed were asked for a brief written explanation. The results of the second questionnaire indicated that consensus was improved in almost every question. Furthermore, consistency was verified in the use of the distress scales; for example, participants assigned higher repair priorities to increasing severities of a given distress.

Finally, the third questionnaire aimed to achieve two objectives, namely, (a) review and confirm responses to questions that were asked for the first time in the second questionnaire and (b) refine the relative significance of each distress type.

Data Reduction

The information generated through the series of questionnaires was used to establish (a) the relative significance (for maintenance decision making) of each distress type and (b) the repair priority of each distress type–severity combination. Consensus on distress type–severity priorities was easily established, but the obtained responses were not consistent about the relative significance of each distress type. Thus, the effort of deriving a composite index for pavement surface condition had to address two major issues, namely, (a) accounting for repair priority of distress extents and (b) resolving inconsistency about the relative significance of each distress type.

The opinions on repair priorities were quantified by mapping the responses into integers (Table 2), the mean values of which were taken as the needed priority values. For example, for a particular distress state, if seven respondents

TABLE 2 INTEGER MAPPING FOR QUANTIFYING REPAIR PRIORITY

Repair Priority	Value
Condition does not warrant repair	0
Very low priority repair	1
Low priority repair	2
Medium priority repair	3
High priority repair	4
Condition is critical; repair is of highest priority	5

indicated a high priority and two indicated medium priority, then the resulting priority value was 3.78. Priority values generated in this manner are summarized in Tables 3 and 4 for overlaid and concrete pavements, respectively.

Values of the relative significance (significance score) of each distress type were proportionally scaled so that those corresponding to major significance, minor significance, and insignificant would be assigned scores of 100, 50, and 0, respectively. The significance score for each distress type was determined as the mean value of the significance scores provided by the nine participants. The resulting values for each distress type are given in Table 5.

Significance scores were interpreted as representing the relative importance of each distress type and priority values as representative of the cells associated with the general extent of various distress type–severity combinations.

Weight Determination

The values of the weights associated with each distress state are determined using the priority values and significance scores. The applied procedure involves the following steps.

1. Record the significance scores and priority values in a blank weight table. If two or more surface conditions are combined in one level of the scale, the priority values associated with each of the conditions are averaged and the average value is recorded.

TABLE 5 SIGNIFICANCE SCORES

Distress Type	Significance Score
OVERLAID PAVEMENT	
Centerline cracking	70.5
Other types of lane cracking	75.1
Surface defects	62.5
Rutting	88.3
Transverse cracking	75.7
Edge cracking	40.2
Shoulder defects	39.7
Lane/shoulder displacement	51.8
CONCRETE PAVEMENT	
Loss of transverse joint seal	71.6
Transverse joint spalling	79.3
Transverse joint faulting	73.2
Longitudinal joint spalling	78.6
Slab surface defects	76.6
Slab cracking	68.2
Shoulder defects	38.1
Lane/shoulder displacement	58.4

TABLE 3 PRIORITY VALUES FOR OVERLAID PAVEMENT (1990)

Distress Type	Severity				
	None	Small	Medium	Large	Total
Centerline cracking	0.00	1.05	2.89	3.22	4.11
Other types of lane cracking	0.00	1.00	2.89	3.11	4.11
Surface defects	0.00	1.57	3.27	4.11	–
Rutting	0.11	–	–	3.22	4.00
Transverse cracking	0.00	1.11	2.99	3.01	4.00
Edge cracking	0.00	1.11	3.00	3.11	4.00
Shoulder defects	0.00	1.00	2.08	2.95	–
Lane/shoulder displacement	0.00	2.00	3.11	4.11	–

TABLE 4 PRIORITY VALUES FOR CONCRETE PAVEMENT (1990)

Distress Type	Severity				
	None	Small	Medium	Large	Total
Loss of transverse joint seal	0.00	–	–	3.29	–
Transverse joint spalling	0.00	1.33	3.00	3.38	4.00
Transverse joint faulting	0.00	–	–	3.00	–
Longitudinal joint spalling	0.00	1.33	2.38	3.67	3.89
Slab surface defects	0.00	0.88	1.22	2.88	3.50
Slab cracking	0.00	1.00	1.14	2.11	3.88
Shoulder defects	0.00	1.00	2.08	2.95	–
Lane/shoulder displacement	0.00	2.00	3.11	4.11	–

2. Identify the "anchor cell" in the table. To do this, consider (only) those distresses that have a "total" scale. The TG cell of the distress that has the highest significance score will be the anchor cell. Assign this cell the arbitrary weight of 10.

3. Calculate weights for the other TG cells in the table on the basis of the ratio of the significance scores as follows:

$$(W_{\mathrm{TG}}) = \frac{(SS_{\mathrm{TG}})}{(SS_a)} \times (10) \qquad (1)$$

where

W_{TG} = weight for a given TG cell,
SS_{TG} = significance score associated with the column that holds the given TG cell, and
SS_a = significance score associated with the column that holds the anchor cell.

4. Calculate weights for all the remaining "general" cells in the columns that contain a TG cell. For each column, use the ratio of priority values as follows:

$$W_{\mathrm{LG}} = \left(\frac{P_{\mathrm{LG}}}{P_{\mathrm{TG}}}\right) \times \mathrm{W}_{\mathrm{TG}} \qquad (2)$$

where

W_{LG} = weight for an LG cell (in the same column as the TG cell),
P_{LG} = priority value associated with the LG cell (in the same column as the TG cell),

P_{TG} = priority value associated with the TG cell, and
W_{TG} = weight for a TG cell.

5. Use the LG weight from the column that holds the anchor cell to calculate the weight for the remaining LG cells (i.e., those scales that have a maximum rating of LG). Use the ratio of significance scores, as in Step 3.
6. Calculate weights for all the remaining "general" cells in the columns that have LG cell as the maximum rating. Use the ratio of priority values as in Step 4.
7. Calculate weights for all the "local" cells in the table as a weighted average of the "general" cells immediately above and below the "local" cell. For example, in a given column the weighted average is expresses as follows:

$$W_{ML} = 0.75\ (W_{MG}) + 0.25\ (W_{SG}) \qquad (3)$$

where

W_{ML} = weight associated with the ML cell of a given column,
W_{MG} = weight associated with the MG cell of a given column, and
W_{SG} = weight associated with the SG cell of a given column.

In summary, weights are generated by assigning the value of 10 to the highest severity-extent combination of the distress that had the highest significance score and then using the priority values and significance scores to proportionately scale weights for the remainder of the cells in the table that correspond to a general extent. Weights for the local cells were derived by taking a weighted average (3:1) of the weights representing the next worst state and the next best state, respectively. The procedure was applied separately for overlaid and concrete pavements and the resulting values for the weights are presented in Tables 6 and 7, respectively.

It should be noted that the weighting factor for Level N of the distress Rutting in Table 6 is greater than zero. This is the (deliberate) case for rutting only and reflects the possibility that maintenance action may still be required even when Rutting is rated N. Rating of N is assigned to sections that exhibit up to 0.5 in. of rutting due to limitations of the current visual distress survey procedure (*1*).

TABLE 6 PDI WEIGHTS FOR OVERLAID PAVEMENT (1990)

	Distress Type							
Rating	A	B	C	D	E	F	G	H
N	0.00	0.00	0.00	0.28	0.00	0.00	0.00	0.00
SL	1.53	1.55	1.63	–	1.78	0.95	0.92	1.72
SG	2.04	2.07	2.18	–	2.38	1.26	1.23	2.30
ML	4.72	5.00	3.95	–	5.40	2.87	2.22	3.25
MG	5.61	5.98	4.54	–	6.41	3.41	2.55	3.57
LL	6.09	6.33	5.41	6.11	6.44	3.51	3.35	4.43
LG	6.25	6.44	5.70	8.05	6.45	3.54	3.62	4.72
TL	7.55	7.99	–	9.51	8.04	4.30	–	–
TG	7.98	8.51	–	10.0	8.57	4.55	–	–

A = Centerline cracking
B = Longitudinal cracking
C = Surface defects
D = Rutting
E = Transverse cracking
F = Edge cracking
G = Shoulder cracking
H = Lane/shoulder displacement

TABLE 7 PDI WEIGHTS FOR CONCRETE PAVEMENT (1990)

	Distress Type							
Rating	G	H	I	J	K	L	M	N
N	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SL	1.04	2.27	–	2.50	–	2.54	1.82	1.66
SG	1.38	3.03	–	3.33	–	3.39	2.43	2.22
ML	2.49	4.29	–	6.46	–	5.39	3.14	2.45
MG	2.86	4.71	–	7.50	–	6.06	3.37	2.53
LL	3.76	5.84	5.80	8.21	5.85	8.53	6.81	4.14
LG	4.06	6.22	7.63	8.45	7.80	9.35	7.95	4.68
TL	–	–	–	9.61	–	9.77	9.23	7.62
TG	–	–	–	10.0	–	9.91	9.66	8.60

G = Shoulder defects
H = Lane/shoulder displacement
I = Loss of transverse joint seal
J = Transverse joint spalling
K = Transverse joint faulting
L = Longitudinal joint spalling
M = Slab surface defects
N = Slab cracking

INDEX CALCULATION

Distress indexes are determined by developing a repair priority score for a given segment and converting the result to a value between 0 and 100. Index values are reported on a 100-point scale, with 100 being the maximum possible score for a given index. Consequently, high index values represent pavement surfaces that exhibit relatively minor distress, and, inversely, low index values correspond to pavement surfaces that are highly distressed.

Related distresses are similarly combined to produce subindexes that are representative of the surface condition of various roadway components. For overlaid and concrete pavements, the two main roadway components considered are the (driving) lane and the shoulder. For concrete pavements, the lane is further divided into slabs and joints. Indexes determined include the index (PDI), lane distress index (LDI), shoulder distress index (SDI), and (for concrete pavement only) joint distress subindex (JDS) and slab distress subindex (SDS). All indexes are calculated similarly and are reported on a 100-point scale. Thus, for example, the expression for determining the pavement distress index for overlaid pavement is as follows:

$$\mathrm{PDI} = \frac{100\left(\sum_{d=A}^{H} W_{\bar{r}d} - \sum_{d=A}^{H} W_{\bar{r}d}\right)}{\sum_{d=A}^{H} W_{\bar{r}d}} \qquad (4)$$

where

$W_{\bar{r}d}$ = weight for the distress state specified by the highest possible rating $\bar{r}$, for Distress Type d;

W_{rd} = weight for the distress state specified by Rating r for Distress Type d for overlaid pavement;

r = linguistic distress rating, with $r \in$ {N, SL, SG, ML, MG, LL, LG, TL, TG}; and

d = distress type for overlaid pavement, with $d \in$ {A, B, C, D, E, F, G, H} (as identified in Table 6).

ILLUSTRATIVE EXAMPLE

Table 8 lists an example of distress ratings for an overlaid pavement segment and the values of the weights and maximum weights that correspond to each distress rating. The calculations for determining each index of interest are as follows:

Overlaid pavement distress index:

$$\sum_{d=A}^{H} W_{\bar{r}d} = (7.98 + 8.51 + 5.70 + 10.00 + 8.57 + 4.55 + 3.62 + 4.72) = 53.65$$

$$\sum_{d=A}^{H} \mathrm{W}_{rd} = (5.61 + 0.00 + 2.18 + 0.28 + 5.40 + 3.41 + 1.23 + 2.30) = 20.41$$

$$\mathrm{PDI} = \frac{100}{53.65} (53.65 - 20.41) = 61.96$$

Overlaid lane distress index:

$$\sum_{d=A}^{F} W_{\bar{r}d} = (7.98 + 8.51 + 5.70 + 10.00 + 8.57 + 4.55) = 45.31$$

$$\sum_{d=A}^{F} \mathrm{W}_{rd} = (5.61 + 0.00 + 2.18 + 0.28 + 5.40 + 3.41) = 16.88$$

$$\mathrm{LDI} = \frac{100}{45.31} (45.31 - 16.88) = 62.76$$

TABLE 8 DISTRESS CONDITION FOR ILLUSTRATIVE EXAMPLE

Distress	Rating	Weight	Maximum Weight
A. Centerline cracking	MG	5.61	7.98
B. Other types of lane cracking	N	0.00	8.51
C. Surface defects	SG	2.18	5.70
D. Rutting	N	0.28	10.00
E. Transverse cracking	ML	5.40	8.57
F. Edge cracking	MG	3.41	4.55
G. Shoulder defects	SG	1.23	3.62
H. Lane/shoulder displacement	SG	2.30	4.72

Overlaid shoulder distress index:

$$\sum_{d=G}^{H} W_{\bar{r}d} = (3.62 + 4.72) = 8.34$$

$$\sum_{d=G}^{H} \mathrm{W}_{rd} = (1.23 + 2.30) = 3.53$$

$$\mathrm{SDI} = \frac{100}{8.34} (8.34 - 3.53) = 57.67$$

The results of the index calculations for the illustrative example are given in Table 9.

DISCUSSION OF RESULTS

Use of Expert Opinion

The Delphi technique used to acquire expert opinion enabled the development of a consensus on repair priorities. Using questionnaires proved to be a convenient device for interacting with a large number of participants at different locations, and anonymity prevented undesirable individual domination. As expected, agreement between experts increased with increasing iterations.

Shortcomings of the applied method were due to the excessive demands placed on the participants' time and to the relatively large number of distress states involved. In an effort to offset these shortcomings, questions were presented in multiple-choice format rather than in the unstructured form associated with the Delphi technique.

Although participants were encouraged to write explanatory notes when necessary, doing so was rare. The multiple-choice format may have suppressed some valuable information that might have otherwise surfaced in a less-directed (e.g., verbal) exchange. Nevertheless, the used questionnaires did serve the goal of keeping the activity focused on repair priority.

TABLE 9 SUMMARY OF RESULTS FOR ILLUSTRATIVE EXAMPLE

Index	Unscaled Values $\Sigma \mathrm{W}_{rd}/\Sigma \mathrm{W}_{\bar{r}d}$	Percentage $100(\Sigma \mathrm{W}_{rd}/\Sigma \mathrm{W}_{\bar{r}d})$	Reported Value (100-percentage)
LDI	16.88/45.31	37.25%	62.75
SDI	3.53/8.34	42.33%	57.67
PDI	20.41/53.65	38.04%	61.96

Data Reduction

Responses identified some distress types (particularly those relating to shoulders) to be much less significant than others. This was determined to be consistent with the use of distresses for pavement maintenance purposes.

Significance scores were introduced as a means to adjust the weights so that they better reflect the relative importance of each distress type. Priority values were used to determine weights associated with the remaining ratings for each distress type in proportion to the maximum value. The fact that priority values generally increased monotonically with the severity of a distress was considered supportive evidence that the derived relationships were appropriate.

The process for deriving weights for the "local" cells involved a simple weighted average (75/25) of the "general" weights immediately above and below the "local" cell, respectively. This weighting was considered by the experts to be reasonable and consistent with the use of the distress scales (i.e., the requirement that the contribution of a "local" distress state be less than the "general" state of the same severity level and greater than the "general" state that is one severity level lower).

Index Calculation

The calculation method produces the total significance of the distress present in a pavement section by using weighting factors as deduct points. For every distress state that exists, the corresponding weighting factor is deducted from the maximum (worst) possible combination of weights for the roadway component of interest. After all deductions have been considered, the remaining value is scaled proportionately to the maximum and reported on 100-point scale.

The scaled value denotes the calculated cumulative distress condition relative to the maximum value a given distress index may receive.

The PDI is useful for comparing projects on the basis of exhibited surface distresses. The established indexes permit comparison from the perspective of (a) lane condition, (b) shoulder condition, and (c) overall pavement condition. For concrete pavements, comparisons of joint and slab condition can also be made.

Comparisons between projects of different pavement types is often necessary. As PDI may be interpreted to represent the percentage of maximum repair priority, it is reasonable to directly compare the PDI values of projects of different pavement type. Projects with similar PDI have similar repair priorities for their respective combinations of surface distresses, regardless of pavement type.

Finally, it should be noted that nondistress aspects of pavement condition (e.g., ride quality and drainage) and performance (e.g., deterioration rate, remaining life) may be quite different for projects with similar PDI values, even if such projects belong to the same pavement type. Based solely on distresses, PDI is a useful tool for pavement surface condition analyses and project comparisons; but it must be supplemented with other engineering parameters for comprehensive project evaluations.

SUMMARY AND CONCLUSIONS

This study presented the methodology followed in determining PDI needed for pavement management purposes. The methodology involved a Delphi-like process for the acquisition of expert opinion and the derivation of weighted average condition measures for various pavement components. Emphasis was placed on making the methodology useful for making a wide range of maintenance decisions.

On the basis of the findings of the study, the following conclusions may be drawn:

- The developed pavement distress index is a viable single measure of pavement surface condition. It is used mainly for network-level analysis.
- The applied Delphi technique enabled the establishment of the relative importance of the various distresses and helped achieve adequate consensus among experts.

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Guidelines for Management of Chip and Sand Seal Coating Activities in Indiana

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Highways are important to the nation's infrastructure: both industry and the public depend on them. Unfortunately, highways are deteriorating at an alarming rate. At the same time, funding for maintenance and repairs is diminishing. Making a case for spending more on maintenance is difficult for several reasons. Logically, performance effectiveness and life-cycle costs should control the decisions about pavement maintenance. Chip and sand seal coating are increasingly used to deal with pavement surface deterioration and to defer capital spending. But the implementation of seal coating is usually left to field managers. Generally, only broad statements guiding such activities are provided. This lack of specific guidance creates problems. The Indiana Department of Transportation (INDOT) wanted to develop, using only currently available data, a guide to help its staff make decisions and to create a consistent practice of seal coating across the state. A life-cycle costing analysis of seal coating is presented. The economic analysis was used to better understand the optimal timing for seal coats. National practice review and an expert opinion survey within INDOT were used to consolidate the state of practice. These sources of information are reviewed, and ways the information was used for developing decision criteria are demonstrated. A decision tree was developed for types of pavement surface distress using data gathered in Indiana. The decision tree suggests a preferred solution and, if funding is a problem, offers a priority ranking for the projects. Recommendations about when to use chip seals and sand seals, and where a choice exists are summarized. The guidelines are designed to meet the needs and constraints of INDOT, but with adjustments they can be used in other jurisdictions.

In the last decade, highway departments have experienced a new working environment characterized by four features:

1. A sharp rise in roadway repair needs due to the aging of the infrastructure (mainly built in the 1960s and 1970s).
2. Declining resources due to the increased competition of public services for the tax dollar and the erosion of the value of money by inflation.
3. The announcement of a new federal policy that stated four expectations or requirements (*1*, p. 1358): (a) the states were to establish systematic procedures for analyzing roadway repair needs; (b) minimum life expectancy of 8 years (5 for special cases approved by FHWA) from newly built, rehabilitated, or reconstructed projects; (c) minimum skid resistance requirements from newly rehabilitated or resurfaced roadways; and (d) the provision of economic analysis in support of requests for funds FHWA. The policy also stated that it would be inappropriate for the states to forfeit maintenance in order to obtain federal funds.
4. The massive retirement of well-trained field staff and their replacement by less-experienced personnel.

This new working environment created pressures to rationalize practices and procedures, establish rules and criteria to guide staff and operations in their decisions, and evaluate the economics of various key maintenance operations within their jurisdiction to create changes that maximize the returns on the dollars spent.

The Indiana Department of Transportation (INDOT) has expressed the desire to investigate, among other things, the practice of chip and sand seal coating, the economics of this practice not only from the agency viewpoint but from that of users as well, and the development of management criteria for unified practice across the state. This paper summarizes the findings and recommended criteria on the chip and sand sealing component of the study.

STUDY APPROACH

Guidance to any maintenance activity such as chip and sand sealing can be stratified into three levels as shown in Figure 1. The highest level of guidance defines whether seal coating is an option for a particular agency and the general circumstances and purposes of its use. The second level refers to the presence of a given set of criteria regarding the attributes of roadway sections that should receive seal coats. The third level refers to the standards to be observed in relation to what materials to use and the procedures to follow, given that seal coating is justified on a certain road section. INDOT has a general statement at the policy level and specifications that cover the areas of materials and procedures; it did not, however, have approved criteria for guiding decisions at the project-management level, and such decisions rested fully with field staff. The department requested that such criteria be developed, taking into consideration the general economics of seal coating to agency and users. Furthermore, two major constraints were imposed:

- The study was to use currently available data (no new data would be collected); and
- The recommended guidelines were to be simple for field staff to use and easy to integrate with INDOT's Roadway Management Project, under development at the time.

The study objective was thus limited to developing management criteria and evaluating the general adequacy of the

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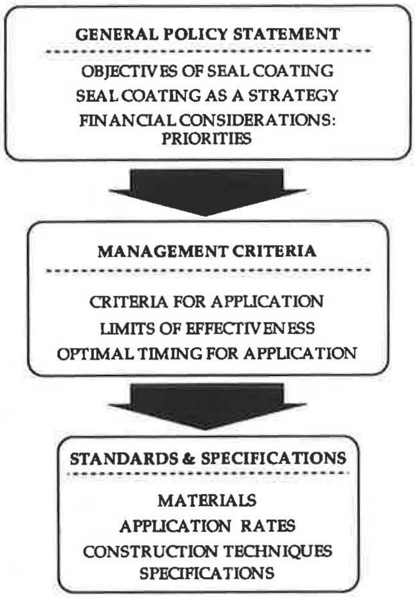


FIGURE 1 Three levels of guidance to seal coating.

policy statement and suggesting modifications as necessary. The study was not for evaluating, revising, or establishing new specifications for materials and construction procedures. To achieve the stated objective, three major areas were identified:

1. A literature review and phone interviews with other state DOTs and research centers;
2. Reviews and evaluations of available data and documents within INDOT, conferences with INDOT staff as required, and an expert opinion survey; and
3. Economic evaluation of seal coating, considering agency and user costs for understanding the general economics of the practice and the optimal timing for seal coating.

The results of these components were grouped, analyzed, evaluated, and synthesized to produce the required guidelines. The results of each component are documented separately in the following.

CHIP AND SAND SEAL COATING AS A PRACTICE

Seal coating is a wide topic. Some seals are applied without aggregates such as fog and construction seals; others are applied with aggregates. When aggregates are used, they can be either mixed in with the emulsions (such as slurry and cape seals) or applied separately (such as chip and sand seals). This study focused on chip and sand seals.

Chip and Sand Seals Defined

Chip and sand sealing involves the application of one or more layers of asphalt-based bituminous material; each layer is immediately followed by the application of a cover of aggregates (in varying thicknesses). These applications are applied to pavements with asphaltic surfaces (flexible and composite—asphalt overlay on rigid), but not on rigid. For chip seals, the cover aggregate is composed of crushed stone, gravel, or slag; for sand seals, the aggregates are either rock screenings or natural sand.

Usage

Chip seals are applied as a blanket cover over oxidized, raveled, and spalled (as in overlaid) pavements; eroded wheelpaths; permeable surfaces; and aging and cracked pavements. They are also often used to restore skid resistance and, more recently, as a strategy to defer capital spending.

Sand seals are generally used to restore a dry, weathered, or oxidized surface; to benefit pavements that have lost some of their matrices; to improve skid resistance; and to reduce raveling.

It is important to point out that both chip and sand seals are surface dressings that affect surface qualities but that have no structural strength to them. Consequently, they cannot, and should not be expected to, treat structural deficiencies or problems.

Factors Affecting Seal Coating Quality

Six major factors affect the ultimate quality of chip seals; they include

- Ambient conditions during and after construction: air and pavement temperature, moisture, and wind;
- Surface preparation before seal coating: whether the pavement is clean and dry or whether it is open, flushing, patched, or shaded;
- Materials: type and grade of asphalt; method of storing and handling of asphalt; type, size, and condition of cover aggregates; and application rates;
- Equipment: distributor spray bar height, nozzle orientation with respect to the bar, spray tip size and cleanliness, and pump condition; spreader gates and auger roller condition.
- Operation coordination: preapplication preparation; control of material application and rolling during the operation; traffic control and brooming of excess aggregates after the rolling; and
- Postsealing inspections: checking of aggregate embedment into asphalt; application of fog seal to compensate for low asphalt application rate or correction of situations in which there is too much asphalt; and reinforcement of weight restrictions.

The cost-effectiveness of any activity depends highly on the quality of materials and workmanship. In the economic evaluation of seal coating, it was assumed that appropriate quality controls were applied and the seals were properly done.

Major Issues

There are many issues related to seal coats; these issues can be grouped under three categories: policy, project management, and materials and workmanship. Each will be discussed separately.

Policy

Policy-related issues concern the principles that should guide the use of seal coating; the superiority of chip or sand seals; situations in which seal coating is cost-effective; and the use of seal coating on high-volume roads and high-truck-volume roads. A summary of the findings follows.

Seal coating is invariably used on roads with asphaltic surfaces but not on concrete. Because of the relatively thicker seal layer, some consider chip seals to be superior to sand seals. Others argue that both seals are useful and have their own applications; hence, the question of which seal is superior is irrelevant. Instead, the question should address when to use which. Seal coating is normally applied on low-volume roads and on roads with a low percentage of trucks. However, some agencies have used it on high-volume roads with very strict traffic controls attached, both during and after construction. Objections cited against the use of seal coats on high-volume roads include the problem of flying stones (and the damage they do to other vehicles) and public objections to closing the road for days to allow for proper curing. Two innovations were reported to have a potentially significant impact on resolving these issues: the sandwich seal (French Dressing) and the application of emulsion breaking agents (*2*). The sandwich seal calls for first applying a dry layer of aggregates on the old surface and rolling it with light steel rollers, next spraying the emulsion, and finally applying a layer of dry aggregates and rolling it with pneumatic rollers. When the emulsion-breaking agents are sprayed on the dry aggregate, the breaking agent reacts with the emulsion and causes it to break much faster than it would without the agent. With regard to truck traffic, conflicting messages are found: some claim the heavy weight of trucks can create severe damage in the thin seal coat layer; others claim that, once the seal coat cures, the weight does not matter—instead, the volume of traffic does.

Project Management

Issues related to project management include items such as what performance indicators to use for seal coating purposes, conditions or criteria to use for identifying candidate projects for seal coating purposes, which type of projects should receive priority when funds are short, and how to measure the cost-effectiveness of seal coating for a given situation.

Little information was found in the literature on this area. Telephone inquiries with a number of states were essential. Most states contacted do not currently have any approved and documented guidelines in this area. However, several agencies indicated that their organizations were working to identify such criteria. Two studies were found that offered a limited number of published criteria. A study in Minnesota (*3*) offered some rules based on surface condition ratings of certain attributes. Washington DOT (*4*) documented recommendations with respect to traffic level [annual average daily traffic (AADT) less than 5,000] and percentage composition of heavy commercial vehicles (trucks made up less than 15 percent for AADT more than 2,000).

Material and Workmanship

The group of issues related to materials and workmanship concerns the best type of asphaltic materials to use and at what rate; the ideal size of aggregates; whether modified asphalts should be used; when to use cationic or anionic emulsions; the relative superiority of cementing materials such as RS-2, AE-90, and AE-150; and so on. As discussed earlier, this group lies within the scope of specifications provision; INDOT's specifications address these matters and, hence, were not of concern to this study.

PAVEMENT PERFORMANCE DATA AVAILABLE

A review of INDOT's files revealed that the department had records of the following pavement performance indicators:

- Road roughness numbers (RN). These measurements are obtained in the field by a PCA Roadmeter. RN measures the square of the number of ⅛-in. displacements of the autobody from the axle per mile.
- Pavement serviceability index (PSI). This index is computed from roughness data using a calibrated equation developed by INDOT's research division.
- Pavement serviceability rating (PSR). This is a visual average rating of the pavement condition based on a windshield survey. It is a combined measure of the comfort of the ride and the pavement surface distress.
- Skid resistance measurements (SkidN). These measures are obtained in the field in accordance with ASTM Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E274).

Of the four categories, RN data were found to be the most reliable. Moreover, RN and PSI were interchangeable because of the established mathematical relationship. PSI was used for the economic analysis because the available user cost information was based on road geometrics and condition measured in PSI. In the recommended guidelines, RN was used because it is the actual measured quantity in the field.

CHIP AND SAND SEALING IN INDIANA

To understand the state of practice of seal coating in Indiana, several methods were used; findings from each are documented.

Review of Historical Data

The period between 1984 and 1987 was chosen as a basis for studying the extent and variety of seal coating practice across Indiana. The annual INDOT road life records and surface change reports (*5*) contained information on the extent of the highway network and the major surface activities undertaken in a given year. INDOT seal-coated some 2,940 mi during the 3-year study period, averaging about 1,000 mi/year. This represents about a ninth of the 8,860 mi of rural roads, which is the candidate population for seal coating (*6*). Seal coating in Indiana during the study period split 50-50 between sand and chip seals. Most districts were observed to use both, but four out of six districts concentrated on one type. Even if two districts favored a given seal, there were variances from district to district because, during the study period, Indiana had four types of single and double, chip and sand seal combinations (see Table 1); INDOT has since revised the types to seven.

Staff Interviews and Meetings

Discussions with INDOT staff indicated that seal coating was normally used on low-volume roads, but exceptions were reported from the northern part of the state, where heavy traffic is normal. Seal coating was used to defer capital spending even on high-volume roads (as high as 13,000 AADT). When seal coating is applied on high-volume roads, strict construction and traffic controls are enforced. INDOT staff opinions were that seal coats last about 4 years in service for medium- to low-volume roads and that perhaps a maximum of 4 seal coats are possible before rehabilitation work is required. It was the consensus among field engineers that the overall traffic level, not truck traffic, is the determining factor for the life of the seal coat.

Expert Opinion Survey

A questionnaire survey was mailed to 14 INDOT staff members in the districts and central office. Ten complete responses (five from districts and five from central office) and one partially complete response were received and used in the final analysis. The survey indicated that the majority (90 percent) of respondents think that chip seals are effective and should continue to be used; a much lower number (64 percent) hold the same opinion about sand seals. In deciding to use chip seals, the primary factors considered are pavement condition and traffic; roughness and age are considered secondary factors. For sand seals, age, roughness, and traffic are the main decision factors. The trigger levels (roughness, for example) vary by surface type. The average life expectancy of sand seals was generally reported to be lower than that for chip seals. Life expectancies of chip and sand seals were observed to vary depending on the road's condition at the time of treatment application. This information proved valuable in the economic evaluation phase because the subsequent observational data did not yield reliable life expectancies.

TABLE 1 TYPES OF SEAL COATS USED IN INDIANA DURING STUDY PERIOD

TYPE	DESCRIPTION	COVER AGGREGATE SIZE NO.	RATE OF APPLICATION PER SQUARE YARD	
			AGGREGATE Pounds	BITUMINOUS MATERIALS Gallons at 60 deg. F
I	SINGLE APPLICATION	Boiler Slag, 11, 12, 13	15 - 25	0.20 - 0.40
II	FIRST APPLICATION	11	15 - 20	0.25 - 0.35
	SECOND APPLICATION	12, 13	15 - 25	0.20 - 0.30
III	FIRST APPLICATION	8, 9	25 - 40	0.35 - 0.50
	SECOND APPLICATION	11	10 - 20	0.25 - 0.40
IV	SINGLE APPLICATION	17, 14	10 - 25	0.10 - 0.25

ECONOMIC EVALUATION OF SEAL COATING

In economic analysis, it is assumed that the road has been built and the issue is whether to seal coat or not. The construction cost of the pavement was, therefore, considered sunk cost and hence excluded. Agency costs included in the analysis are those of maintenance, seal coating, and rehabilitating at the end of the life cycle. User vehicle operating costs were also included using the latest 1982 FHWA update study (*7*).

Economic Evaluation Framework

Seal coating is generally associated more with pavement distress than with roughness. Because of the completeness and greater reliability of the RN data and the availability of user costs in terms of PSI (which is highly correlated with RN), it was decided to use roughness data for the economic evaluation.

The underlying logic of the economic evaluation can be illustrated with the aid of Figure 2. The figure demonstrates that the agency maintenance and user operating costs increase with roughness. As shown, there is a decrease in agency and user costs when seal coating is applied. Subsequently these costs continue increasing until the end of the pavement's life cycle; the cycle was assumed to repeat perpetually. The maintenance costs used in the evaluation were correlated to pavement age and condition using Indiana's observational data sample (with RN changed into PSI). User costs were available from the source by road condition (measured in PSI), geometrics, and attributes. If the pavement received two or more seal costs, similar cost profiles were assumed and the total cycle was again assumed to repeat itself in perpetuity. The impact of seal coating on roughness was found to vary with the pavement's condition at the time of seal coating. This finding was confirmed in other studies (*8*).

Pavement performance was first predicted for every year and strategy assumed; maintenance, agency, and user costs were then assigned according to the condition experienced. All costs were finally discounted to the present worth using 6 percent interest rate and the equivalent uniform annual cost (in perpetuity) was computed.

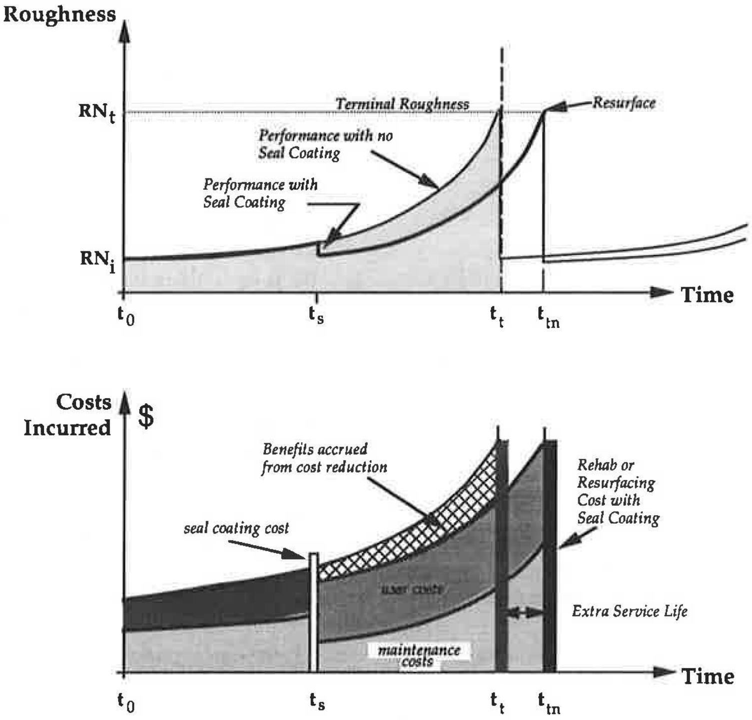


FIGURE 2 Agency and user cost profiles for seal coating strategy.

Deriving Basic Functions for Seal Coating Impacts in Indiana

In order to derive the required functions for carrying out the economic evaluation, a stratified two-stage sample was picked from 12 subdistricts. These data were used to derive three main functions:

1. The first function related PSI (before treatment) of the road sections (with AADT < 3,000) that have only been seal-coated to their current age (see Figure 3).

2. The second function related the PSI jump due to seal coating to the original PSI before seal coating. Due to their quality, the data for this function displayed significant noise (accumulation of errors attributable to items other than the effects of the parameter of interest); consequently, it was decided to carry out the analysis for the most optimistic scenario by assuming the maximum impact experienced. For initial PSI (value before action was taken) greater than 1.6, the jump was estimated as

$$\text{PSI jump} = 0.41\ (\text{PSI} - 1.6) \qquad (= \text{zero, PSI} \leq 1.6)$$

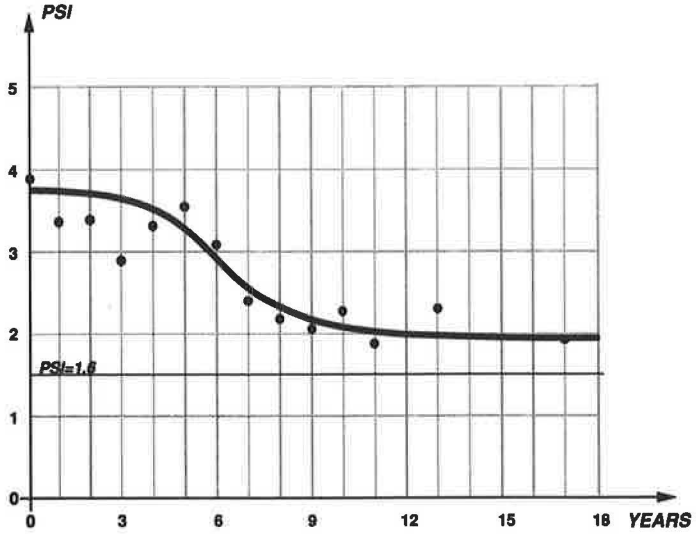


FIGURE 3 PSI as function of age for low-volume roads (AADT < 3,000; points plotted represent means; total observations = 22).

3. The third function related the routine maintenance cost to age of pavement (as shown in Figure 4).

Life expectancies of seal coats were extracted from the expert opinion survey, as mentioned earlier.

Analysis Results

Four scenarios were analyzed in this study, as shown in Figure 5. Each scenario (or strategy) was analyzed for four levels of AADT (2,500, 1,500, 1,000, and 500) and varying PSI trigger levels for action (varying from 3.5 to 2.5 PSI). The results (*9*) are discussed in the following.

Agency Costs

Theoretically, agency cost is expected to vary with traffic and road condition. However, as demonstrated in Figure 4, maintenance curve used is almost flat within the first 8 to 9 years. Moreover, within small ranges of traffic variation (such as the relatively low volumes tested), agency cost is not expected to be highly sensitive to AADT variations, particularly after discounting to present worth. The variation in agency cost can be expected to vary with one of two factors: variation in PSI action trigger level and variation in the number of seal coats applied in a lifetime. As calculated, agency cost tended to be insensitive to variation in PSI trigger levels. For example, agency cost dropped marginally (less than 2 percent) as the PSI trigger level was dropped within the 3.5 to 3.0 range, but is was insignificant (less than 0.5 percent) when the PSI range was less than 3.0.

As expected, agency cost tended to decrease with the number of seal coats but at lower pace. For example, without seal coating, the annual agency cost per lane mile was about $2,607, in perpetuity; with one seal coat (at PSI 3.00), it was $1,652; with two seal coats (at PSI 3.00), $1,187; and with four seal coats (at PSI 3.00), $744.

User Costs

User costs are expected to change with the variation of the same factors above plus the usage levels (AADT). As calculated, user costs tended to drop significantly after one seal coat but rose with the application of two or more seal coats. For example, for PSI = 3.00 and AADT = 2,500, user costs with no seal coating strategy were on the order of $82,208; with one seal coat, $80,153; with two seal coats, $81,816; and with four seal coats, $83,134. Hence, seal coating four times was estimated to cost the users more than not seal coating at all; the magnitude and rate of this rise after one seal coat varied by AADT. The explanation is perhaps that because applying more seal coats preserves mediocre condition when compared with resurfacing after one seal coat.

User costs tended to increase with the delay of seal coating action (action PSI level was lowered). The increase was very slow in general, about 1 percent. And user costs tended to drop directly with AADT level. For example, for assuming PSI = 3.0 as a trigger level and seal coating twice in a lifetime, user costs were estimated at about $81,816 for AADT = 2,500; $49,089 for AADT = 1,500; $32,726 for AADT = 1,000; and $16,363 for AADT = 500.

Total Costs

The variation of the total cost depends on the variation in its components. However, because the component costs varied inconsistently, the total costs tended to vary in the same inconsistency. As the action PSI trigger point is lowered, the increase in user cost was much higher than the marginal gains in agency cost savings. Hence, for this variable, total costs tended to follow user costs. Stated inversely, the better the surface condition at the time of application of the seal coat, the greater the overall benefits.

The total cost (agency plus user costs) tended to drop after one seal coat but increased with the increase in the number of seal coating applications for roads with AADT ≥ 1,000.

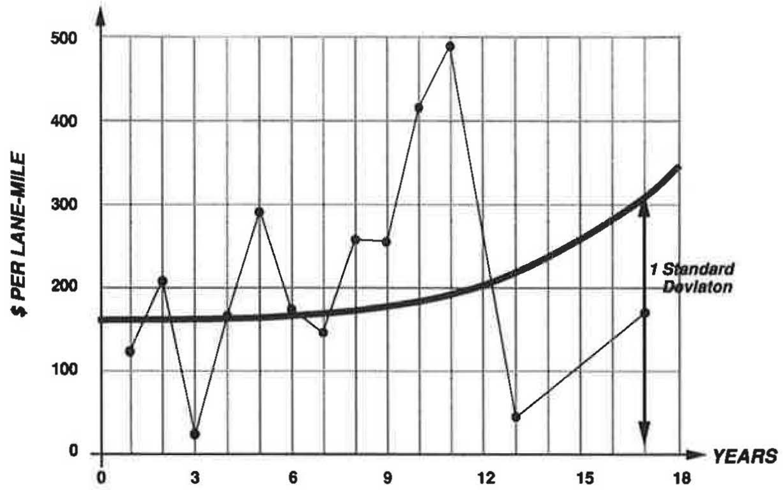


FIGURE 4 Only-basic routine maintenance cost as function of age (points plotted represent means; total observations = 22).

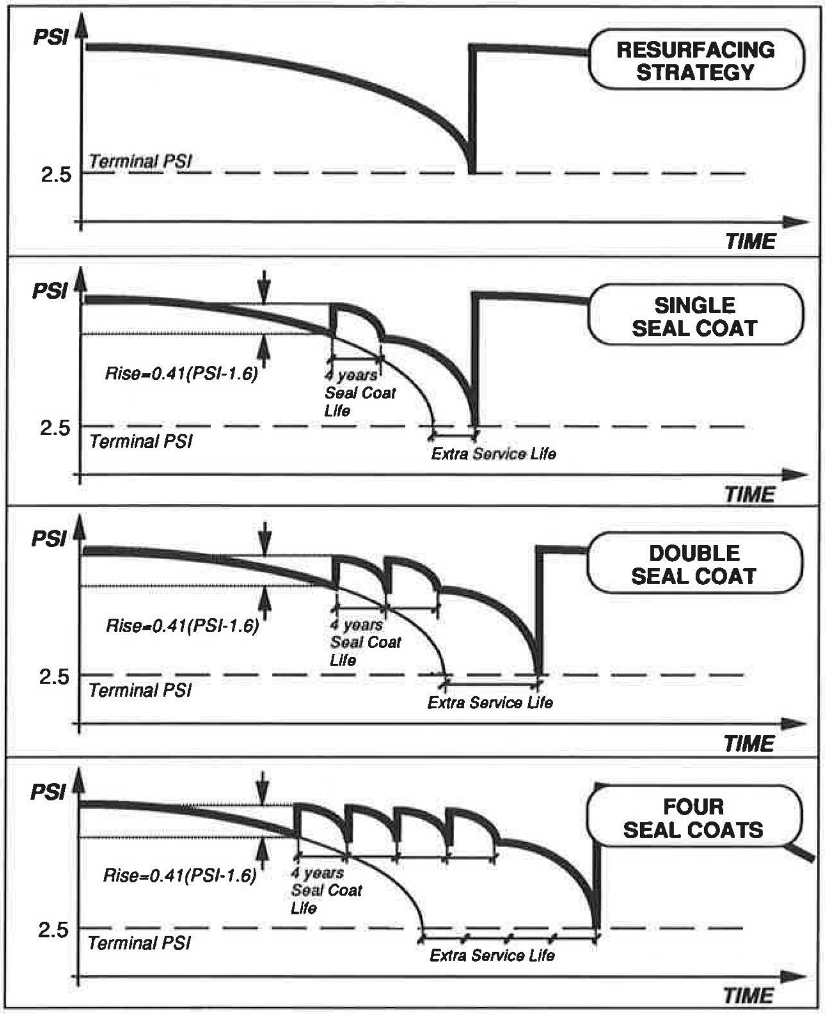


FIGURE 5 Alternative maintenance strategies considered.

For roads with AADT < 1,000, the total cost continued to decrease. This suggests that for very low volume roads, seal coating is a cost-effective solution and rehabilitation should be done only when absolutely necessary. For roads with higher traffic volume, seal coating once, perhaps twice as a maximum, was more desirable. Because agency cost did not vary within any given strategy, user costs were the determinants of trends with respect to total cost variation with traffic.

In summary, the economics of seal coating suggest several management policy lessons and directions:

- Seal coating does yield overall savings and hence should be retained as a legitimate option;
- The earlier the seal coating, the better (as long as it is not done too early);
- "The more seal coats, the better" is true for very low traffic (AADT ≤ 1,000) but not necessarily for roads with more significant traffic (AADT > 1,000).
- Taking into account both economic and practical considerations for timing, seal coating of higher traffic volumes (AADT > 1,000) would be ideally desirable at PSI = 3.0, whereas that for lower traffic volumes, at PSI = 2.7 (at 2.5, resurfacing would be required);
- When seal coating is to be used for buying time for the resurfacing option, it should not be used on roads with PSI ≤ 2.0; and
- Cost-effectiveness is dependent on one's perspective: agency, user, or total cost perspective (in this study, the total cost was used as a basis for evaluating cost-effectiveness).

POLICY GUIDELINES

This section summarizes the policy guidelines based on the findings of the research. The recommended guidelines can be grouped under three headings: general policy statement, specific management criteria, and guidelines on specific issues. A discussion of each follows.

General Policy Statement

Seal coating should be used on low- or medium-volume roads in the amounts necessary to correct surface deficiencies (but not structural) or to prevent the development of more-serious structural problems. Specifically,

- Seal coating may normally be applied on flexible or composite (asphalt overlay on PCC) pavements carrying 2,500 AADT or less; application to higher-volume roads can be made as long as adequate traffic controls are put in place to ensure sufficient time for curing;
- Seal coating should be considered on roads exhibiting slipperiness, bleeding, oxidization, raveling, spalling (composite pavements), erosion (dusting), or a permeable surface; and
- Seal coating may also be considered as an alternative measure to delay capital spending on roads with any traffic volume level.

Management Criteria

The recommended management criteria take the form of a decision tree. The tree, however, not only specifies the situations for which seal coating is the preferred solution from an engineering viewpoint, but it also provides priority rankings in case of fund shortages

Definitions of Seal Coating Priority Groups

In case of fund shortages, roadway sections that meet the low usage and bituminous surface criteria set in the general policy statement fall into one of four groupings based on these criteria:

- Priority Group 1—This group includes roads that need seal coating for safety reasons (SkidN < 30) and are subjected to significant usage (AADT $> 1,000$); roads that are in fair structural condition but exhibit aging signs [oxidation and mild alligator cracking (*10*)]; relatively new roads that have surface mix deficiencies and are subjected to high usage (AADT $> 1,000$); and roads that are raveling or showing signs of erosion or a permeable surface.
- Priority Group 2—This group includes roads that need seal coating for safety reasons (SkidN < 30) and are subjected to low usage (AADT $< 1,000$) and high-usage roads (AADT $> 1,000$) experiencing roughness confined to the surface but no structural distress.
- Priority Group 3—This group includes roads that cannot be resurfaced because of a shortage of funds for at least 2 to 2.5 years hence and roads that have a low level of serviceability (PSR $= 2.0$) but are not scheduled for capital work in the near future.
- Priority Group 4—This group includes roads that have roughness problems only (that is, acceptable PSR and no skid problems); roads that are suffering from structural problems and surface problems; and roads with curbed sections.

Decision Tree

A decision tree incorporating these criteria is shown in Figure 6. It can be used as a guide for interpreting the policy in management decisions relating to seal coating. The decision criteria were selected with consideration of the currently available information in INDOT: skid numbers, roughness numbers, age, and present serviceability ratings (measures of pavement distress). Using the pertinent information on a given section and following the appropriate path leads to a recommended technical solution and programming priority under financial constraints.

Ideally, these four groups of criteria will be augmented with detailed surface attributes and distress-related indicators or measures, when such parameters become available. The decision tree must be expanded to accommodate such new criteria.

Guidelines on Specific Issues

Guidance is offered on the applicability of seal coating for three specific issues.

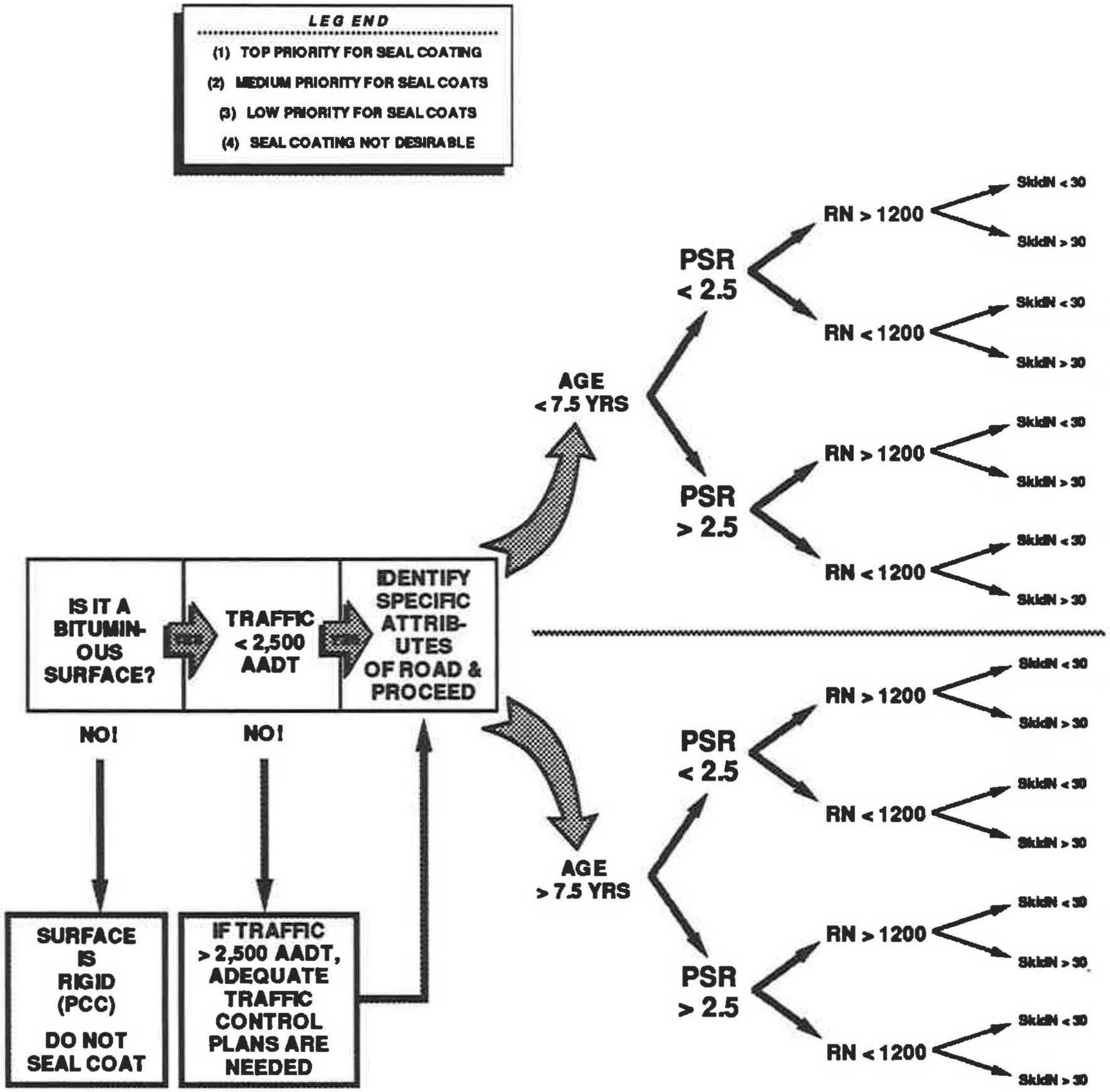
Sand or Chip Seal

Some rules for when chip seals and sand seals should be applied follow.

- Sand seals are more cost-effective for use on oxidized pavement; chip seals are more effective than sand seals on cracked pavement.
- Chip seals tend to have more bleeding problems than sand seals. Hence, for bleeding pavements, sand could be more effectively employed to bloat excess asphalt.
- If the source of poor skid resistance is the loss of fine matrix around the coarse aggregates, then sand sealing would be the more cost-effective.
- Chip seals are more effective than sand seals on spalled surfaces (overlaid pavements).
- Severe raveling or cracking can be treated more effectively by chip seals than by sand seals.
- Because sand seals are cheaper then chip seals and both seals bond together effectively, sand seals could be an option for the first seal during double sealing.
- In making the final choice of sand or chip seal, the availability and economics of quality materials would have to be considered.

Seal Coating on High-Volume Roads

The recommended general policy direction did not exclude the use of seal coating on high-volume roads provided adequate traffic controls were put in place to ensure adequate curing. The time required for curing depends on many factors, including the type of binder used and ambient conditions. Generally, two objections have been given to the use of seal coats on high-volume roads: the long closures of the roads to traffic and the damage created by flying stones. These objections may be addressed by the use of smaller, precoated chips. The amount of flying chips would then be reduced and, hence, the damage. The use of the two innovations reported earlier should also be tested for cost-effectiveness on this type of road.



PROBABLE AREA OF PROBLEM	PREFERRED STRATEGY FROM TECHNICAL VIEWPOINT: *FUNDS ARE NO PROBLEM*	RECOMMENDATION WHEN *CAPITAL IS A PROBLEM*	
		RELATIVELY HIGH USAGE (TRAFFIC > 1000 AADT)	RELATIVELY LOW USAGE (TRAFFIC < 1000 AADT)
SURFACE, SUBBASE AND/OR SUBGRADE	REHABILITATE	(4)	(4)
SUBBASE AND/OR SUBGRADE	REHABILITATE	(4)	(4)
SURFACE	SEAL COAT	(1)	(1)
SURFACE	SEAL COAT	(1)	(2)
SURFACE	SEAL COAT	(3)	(3)
SURFACE	SEAL COAT	(3)	(3)
SURFACE	SEAL COAT	(1)	(1)
NORMAL WEAR	ROUTINE MAINTENANCE	(4)	(4)
SURFACE, SUBBASE AND/OR SUBGRADE	REHABILITATE OR RECONSTRUCT	(4)	(4)
SUBBASE AND/OR SUBGRADE	REHABILITATE	(4)	(4)
SURFACE	RESURFACE OR SEAL COAT	(1)	(2)
SURFACE	RESURFACE OR SEAL COAT	(1)	(2)
SURFACE	RESURFACE	(3)	(3)
SURFACE	SEAL COAT	(2)	(3)
SURFACE	SEAL COAT	(2)	(3)
NORMAL WEAR	ROUTINE MAINTENANCE	(4)	(4)

FIGURE 6 Recommended decision tree for seal coating decisions.

Seal Coating on High-Truck-Usage Roads

No structural strength is attributed to seal costs, so high volumes of heavy loads would be expected, from an engineering perspective, to cause greater damage. Seal coating on roadways carrying high volumes of trucks is not expected to last long enough to justify the investment. As such, it should be discouraged in practice.

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Rational Method for Selecting Maintenance Treatment Alternatives on the Basis of Distress Structural Capacity and Roughness

GABRIEL ZOLTAN, ARIEH SIDESS, AND HAIM BONJACK

Primary concerns of a pavement maintenance management system (PMMS) include determining the kind of treatment appropriate to periodic maintenance and classifying sections by priority order. A rational method is presented for selecting maintenance treatment solutions based on (a) pavement performance as exhibited by indexes of visual distress rating by the Washington survey method; (b) structural capacity evaluated by nondestructive testing using a falling weight deflectometer; and (c) roughness determined by the present serviceability rating. Unlike PMMS applied in some parts of the world, this system refers individually and quantitatively to each index separately, considering its engineering significance and allowing calculation of the necessary structural strengthening. The method is based on the classification of road network sections by a decision tree. At each intersection, classification is performed per index suitable to the criteria under consideration. After the sections are classified, each section is assigned its appropriate treatment out of a treatment inventory available at the maintenance department. Determination of the treatments' economic feasibility and order of priority is based on the net present value and the internal rate of return of cost flows of periodic and routine maintenance and vehicle operation costs.

The main objects of a pavement maintenance management system (PMMS) include determining the appropriate treatment and maintenance solution that will be applied at each road network section and classifying the sections by treatment priority order on the basis of economic and engineering considerations (*1–3*). Determination of the necessary treatment solution is based on the current performance rate of the section and its anticipated traffic data. Road authorities in the world are using various criteria indexes to evaluate pavement performance. In general, these indexes may be divided into four elementary groups as follows (*3*): visual distress, roughness (riding comfort), structural capacity, and pavement surface friction (safety).

The number of indexes used and the integration between them differs from system to system. For instance, in the Paver (*4*), Washington (*5*), Texas (*6*), and California (*6*) methods, one or two indexes were used (visual distress and roughness); in the Macpon (*7*), Belgian (*8*), Kentucky (*9*), and Swiss Neuchatel (*10*) methods, all four indexes were used. For the purpose of decision making, that is, section classification and type of treatment determination, the above PMMSs base themselves directly on the measured values or on empiric values obtained from normalization and weighting of the measured values (*11*). The advantage of this method is its simplicity. However, there is also a striking disadvantage in the nonrational association between the rehabilitation solution and the weighted value used for its selection. In other words, the weighted value does not express the uniqueness of every index in the category that the index specializes in. Therefore, this approach will render identical solutions to two sections with an identical weighted index value, which is obtained out of different combinations of elementary index values, whereas if the diagnosis had been performed by observing every index and its engineering implications individually, the maintenance solution of the sections in question might have been different.

This paper discusses development and application of an engineering-economic approach that enables the following: (a) classification of road network sections according to their performance characteristics; (b) determination of a suitable maintenance solution at the network level, at which the overall problems pertinent to each section are analyzed; and (c) classification of sections by treatment priority order on the basis of internal rate of return. The work was performed within the framework of an economic engineering evaluation project comprising 900 km of roads in Israel's main highway network, which was divided into 170 sections. The elementary data on which work was based are (a) visual distress data, collected according to the Washington approach; (b) deflection basin data measured by a falling weight deflectometer (FWD); and (c) roughness data obtained by means of a car road meter.

PROPOSED APPROACH DESCRIPTION

Basic Assumptions and Principles

The system, which was developed within the present framework, consists of three stages: (a) classifying the network sections by their performance, (b) choosing the appropriate maintenance treatment, and (c) classifying sections according to treatment priority order.

The classification and characterization methodology based on performance follows these guidelines:

1. Systematic approach and simplicity—Clearly defined principles, activities, and indexes will enable the computer-

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ized handling of a massive data amount as well as the manual handling after receipt of basic data, even by operators not equipped with theoretical knowledge.

2. Individualistic and modular utilization of indexes—Each criterion index will be examined by stages, at each stage only one index with reference to its particular specialization.
3. Rational approach—This is the rational reference toward indexes while their engineering significance and their measured numerical value are examined.
4. Flexible structure—Modular structure of the system enables change execution at different stages without disrupting the framework and the general procedure.
5. Versatile use—It is possible to use the system in an identical procedure for an individual sample data level, accumulated data at the section level (project level), and overall data at the network level.

The basic principles of the maintenance treatment selection procedure are

1. Treatment selection according to failure character—A comprehensive, fundamental treatment procedure is chosen to suit the section's problem scope as expressed by the various specification indexes.
2. Quantitative definition of the required structural strengthening determination of a quantitative rehabilitation solution—The section possesses a necessary structural strengthening thickness (overlay thickness).
3. Use of practical work procedures—The proposed solutions are based on work procedures, equipment, and local maintenance teams' skills.
4. Economic evaluation accommodation—This treatment solution definition enables its easy conversion into financial values for economic evaluation and classification according to treatment priority order.

Classification Procedure Description

Figure 1 shows in detail the classification methodology and treatment selection procedure in the form of a decision tree. Classification consists of three integrated subsystems; in each the significance and effect of visual distress, structural capacity, and roughness are examined.

Visual Distress Subsystem

The damages observed on the pavements' surface are the parameters that mainly prescribe the type and character of routine corrective maintenance of the section. For major (periodic) maintenance and rehabilitation, these damages determine the surface preparatory treatment type that precedes the overlay strengthening. The data of a distress survey and DR parameters based on them are used in three stages:

1. Differentiation between visually satisfactory and other sections—This differentiation is made by the criterion value of DR = 80. This and higher values of DR ensure that there is no fatigue distress in the section. In this case there is at the most a limited amount of other distresses, such as transversal and longitudinal cracks and patching at a low to medium severity level. The satisfactory classified sections do not demand any surface preparation treatment, and the requirement for overlay is dictated by the other two indexes. Such a differentiation is also needed because of public or institutional

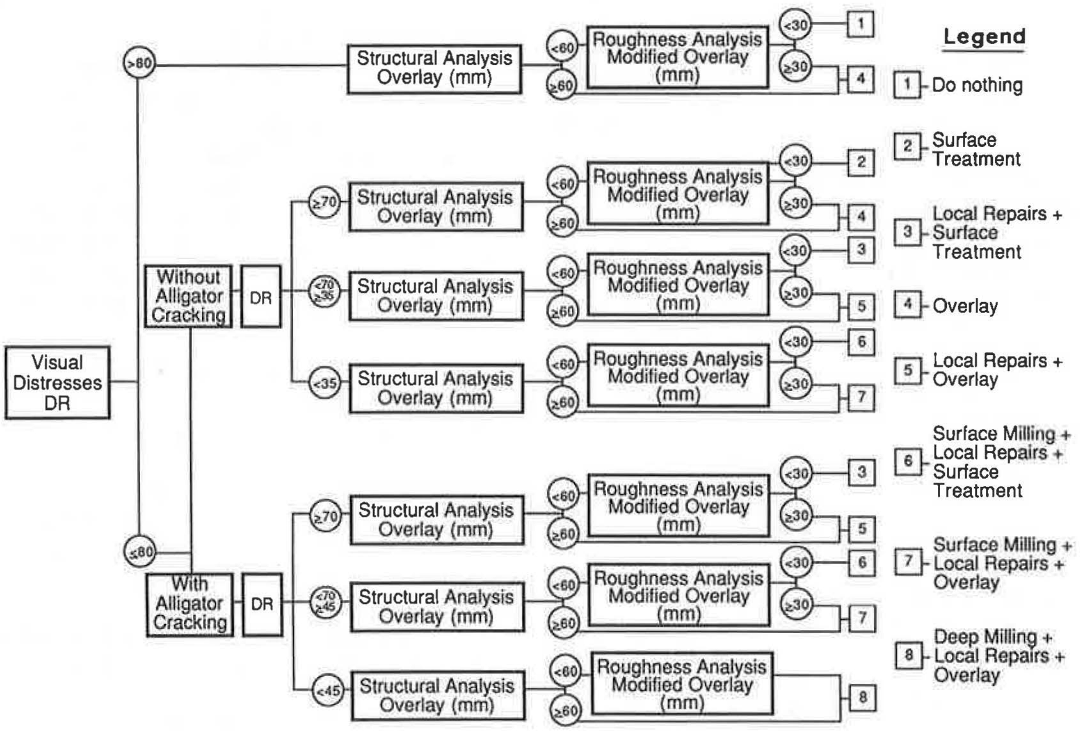

FIGURE 1 Decision tree for alternative maintenance treatments.

TABLE 1 CLASSIFICATION OF SECTIONS FOR SURFACE PREPARATION

Section	Treatment
With Fatigue Distress	
Damage Level	
Low: DR > 70	Not required
Medium: 35 ≤ DR < 70	Local repairs
High: DR < 70	Surface milling and local repairs
Without Fatigue Distress	
Damage Level	
Low: DR > 70	Not required
Medium: 45 ≤ DR < 70	Surface milling and local repairs
High: DR < 45	Deep milling and local repairs

pressure that might arise against maintenance activities in apparently well maintained sections even though the maintenance is technically or economically justified.

2. Differentiation between sections with fatigue cracking and other sections—This differentiation is required because a massive presence of fatigue cracks might suggest failure or structural capacity problems related to the asphaltic layer. In such instances the surface preparation treatment must be more intense and on a greater scale. This classification is directly based on quantity and severity level of fatigue cracking. The criterion value is determined by a weighted percentage (sum of individual samples' deduct values divided by number of samples) of fatigue cracking of 20 percent at the section, equal to the deduct value of 13 points by the Washington method (*5*) (100 percent fatigue cracking at a high level rate is equivalent to 65 deduct points).

3. Differentiation between sections according to damage level—Two section groups, one defined "with fatigue cracking" and the other "without fatigue cracking," are classified into three subgroups, each according to the damage rate. Such classification is necessary to define the surface preparation type and level. The criterion values for this classification are given in Table 1. The surface preparation treatment and DR classification criterion values are higher for the sections with fatigue distress because of the special structural meaning assigned to fatigue distress.

Structural Capacity Subsystem

One of the principle subsystems of the overall scheme is the structural capacity system, designated to produce a rehabilitation solution for the whole road network. The development of such a subsystem must be able to be incorporated within the general overall PMMS, to present fast and reliable rehabilitation solutions at the network level, and to use input data that are relatively easy to obtain.

As presented by Yariv-Civil Engineering Ltd., (*12*) such a subsystem–which is based on nondestructive testing (NDT) of deflection basins measurements and on the rational approach–was developed and applied. The rational approach characterizes the pavement response to the major distress criteria such as fatigue and rutting. A detailed description of the subsystem can be found in works by Yariv-Civil Engineering and Sidess et al. (*12,13*). To complete the representation of the decision tree, a brief description of the subsystem principles is given:

- According to the measured deflection basins and moduli derivation of the pavement layers and subgrade, criteria were established to classify the subgrade and pavement as weak, medium, or strong. Classification of subgrade was based on the seventh deflection $-D_6$ at 1.80 m from the load plate; classification of pavement was based on the surface curvature index. SCI $= D_o - D_1$, where D_o is central deflection and D_1 is the deflection at 0.3 m. The criteria values for the pavement and the subgrade classification are shown in Table 2. They relate to the measured values, corrected to standard conditions of load and temperature. All the deflection basins were corrected according to a standard load of 75 kN (16.5 kip) in a linear manner as follows:

$$D_i^{\mathrm{cor}} = D_i^m \times \frac{75}{P_m} \tag{1}$$

where

D^{cor}_i = corrected deflection for standard load of 75 kN for ith sensor,
D^m_i = measured deflection at P_m load for ith sensor, and
P_m = load at measurement time.

The correction function of the central deflection (D_o) to standard temperature of 30°C (86°F) was carried out by the following equation:

$$F_T(D_o) = 1.694 - 3.155 \times 10^{-2}\, T_p + 3.286 \times 10^{-4}\, T_p^2 - 1.667 \times 10^{-6}\, T_p^3 \tag{2}$$

TABLE 2 SUBGRADE AND PAVEMENT CLASSIFICATION BASED ON NDT (micron)

Pavement	Subgrade Classification		
Classification	Weak	Medium	Strong
Weak	$D_6 \geqslant 105$ SCI $\geqslant 750$	$55 \leqslant D_6 < 105$ SCI$\geqslant 750$	$D_6 < 55$ SCI $\geqslant 750$
Medium	$D_6 \geqslant 105$ $350 \leqslant$ SCI < 750	$55 \leqslant D_6 < 105$ $350 \leqslant$ SCI < 750	$D_6 < 55$ $350 \leqslant$ SCI < 750
Strong	$D_6 \geqslant 105$ SCI < 350	$55 \leqslant D_6 < 105$ SCI < 350	$D_6 < 55$ SCI < 350

where T_p is the asphalt layer temperature at time of measurement in degrees Celsius.

- According to these classification criteria, a parameter called structural index (SI) was defined. This index within the 0–1.0 range expresses the performance condition of the rehabilitated pavement in terms of remaining life. According to principles shown by Yariv-Civil Engineering and by Sidess et al. (*12,13*) the index was determined depending on pavement and subgrade categories as follows:

For a weak subgrade ($D_6 \geq 105\ \mu\text{m}$),

$$SI_W = \left[\begin{array}{ll} 0.2 & (SCI \geq 750\ \mu\text{m}) \\ 2.361 \times 10^5\, SCI^{-2.112} & (350 \leq SCI < 750\ \mu\text{m}) \\ 1.0 & (SCI < 350\ \mu\text{m}) \end{array}\right. \quad (3)$$

For a medium subgrade ($55 \leq D_6 < 105\ \mu\text{m}$),

$$SI_M = \left[\begin{array}{ll} 0.25 & (SCI \geq 750\mu\text{m}) \\ 4.243 \times 10^4\, SCI^{-1.819} & (350 \leq SCI < 750\ \mu\text{m}) \\ 1.0 & (SCI < 350\ \mu\text{m}) \end{array}\right. \quad (4)$$

For a strong subgrade ($D_6 < 55\ \mu\text{m}$),

$$SI_S = \left[\begin{array}{ll} 0.3 & (SCI \geq 750\ \mu\text{m}) \\ 1.046 \times 10^4\, SCI^{-1.50} & (350 \leq SCI < 750\ \mu\text{m}) \\ 1.0 & (SCI < 350\ \mu\text{m}) \end{array}\right. \quad (5)$$

It must be emphasized that there is no relationship between the numerical value of SI in the different subgrade categories. Identical SI values in different subgrade categories are not equivalent and do not express identical strength of the pavement.

- The overlay thickness design curves depending on the SI were developed on the basis of the rational approach, which relates the pavement response to major deterioration criteria such as fatigue and rutting. The adopted load distribution range is 20–180 kN (*12,13*). The effect of the mixed load distribution was taken into account according to Miner hypothesis (*14*). The fatigue criterion was adopted according to the model proposed by Finn et al. (*15*) with some modification recommended by Uzan (*16*). The model of Verstraten et al. (*17*) was adopted as a rutting criterion.

As an example, Figure 2 shows overlay thickness design curves depending on the SI for a medium subgrade classification. The overlay thickness is presented for various equivalent 130 kN single axle load applications, the design axle load in Israel. The other design curves for a weak and strong subgrade are shown by Yariv-Civil Engineering and Sidess et al. (*12,13*).

Application of the subsystem for determination overlay thickness of rehabilitated pavement consists the following stages:

1. Deflection basin measurement and asphalt temperature at the time of measurement.
2. Correction of the central deflection D_o to the standard load level and temperature (Equations 1 and 2) and of deflections D_1, D_6 to the standard load.
3. Calculation of the SCI parameter according to the corrected deflections D_o and D_1.
4. Subgrade and pavement classification according to Table 2 and SI calculation (Equations 3–5).
5. Traffic analysis for design period.
6. Overlay thickness design according to subgrade classification, SI, and traffic analysis (for instance, Figure 2).

Roughness Subsystem

Roughness of the pavement surface is an expression of riding comfort (service level) and serves as an indication for user costs. Correction of roughness improves the service level and decreases operation costs, regardless if roughness treatment responds also to the problems expressed by the other indexes. The roughness correction is performed by a leveling layer, the thickness of which adds up to the required layer thickness

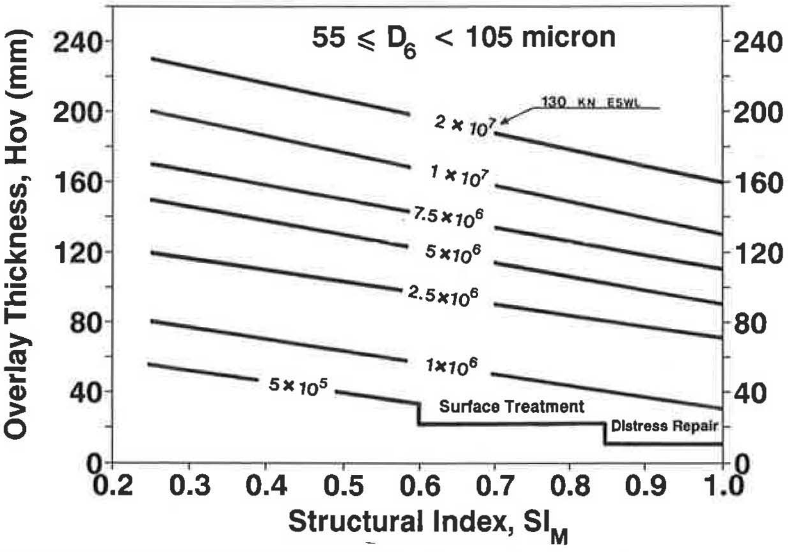

FIGURE 2 Overlay thickness versus SI_M for medium subgrade.

due to structural capacity. Calculation of the added thickness is based on the following assumptions:

- The added thickness is proportional to the present serviceability rating (PSR) value of the section, where the poor sections (PSR ≤ 1.5) receive the maximum of 30 mm and satisfactory sections (PSR ≥ 3.5) are left with the original thickness.
- The required thickness addition, to improve roughness of pavement surface, is calculated only for sections whose required structural overlay thickness is less than 60 mm on the basis of the assumption that a layer of 60 mm and above can act also as leveling layer for a solution to a roughness problem.
- The total layer thickness (leveling layer + structural layer) after the roughness repair should not be less than 40 mm.

Decision Tree Description

In the detailed classification process (Figure 1), each section is being tested at up to five decision cross sections, at which the criterion indexes are examined by stages (at each intersection, only one index). When the process is completed, the sections are divided into 20 subgroups and each subgroup is assigned the appropriate treatment procedure. The first classification stages are executed with reference to the visual distress index. Initially, the sections are channeled into three main paths: (a) satisfactory sections, (b) sections with fatigue cracking, and (c) sections without fatigue cracking. Then each of the last two paths is divided into three subpaths: low-level damage, medium-level damage, and high-level damage. In the next stage, the structural capacity index is examined. At each of the seven subpaths, structural capacity evaluation is performed and the required structural strengthening (overlay) thickness is calculated. Finally, the roughness index is evaluated and, if necessary, the overlay thickness is updated. The roughness index evaluation is performed only on sections whose strengthening thickness is less than 60 mm. The subgroups with a required layer thickness of less than 60 mm are divided into two paths with reference to 30-mm criterion thickness.

Treatment Types

For each of the 20 groups obtained at the end of the classification process a suitable maintenance treatment type is assigned out of the treatment inventory available to the maintenance department. The list contains eight treatment types, based on the following activities: (a) local repairs (patching and crack sealing), (b) milling (surface or deep milling), (c) surface treatment (slurry seal), and (d) overlay (thickness as determined in structural evaluation process). These procedures were chosen because they are most common in a local maintenance system.

The treatment types, which are derived from an integration of the basic maintenance procedures and proposed as maintenance treatment solutions to the different decision tree paths, are as follows:

- Treatment 1: Do nothing.
- Treatment 2: Surface treatment; recommended if the resulting layer is less than 30 mm.
- Treatment 3: Local repairs and surface treatment; local repairs consist of in-advance repairs at weak spots that may become sites of failure after surface treatment.
- Treatment 4: Overlay.
- Treatment 5: Local repairs and overlay.
- Treatment 6: Surface milling, local repairs after milling, and surface treatment; milling is a preparation treatment at the massive cracking areas that can not be repaired by local patching.
- Treatment 7: Surface milling, local repairs after milling, and overlay.
- Treatment 8: Deep milling, local repairs after milling, and overlay.

Some remarks on treatment types follow.

If milling is required, surface milling is sufficient for sections without fatigue distress.

At sections where fatigue distress was diagnosed, surface preparation treatment in the form of local repairs or milling (or both) before the overall treatment is needed.

Deep milling of the asphaltic layer requires adding to the designed thickness a reinforcement thickness equivalent to strength losses due to the milled layer. The proportion of the milled layer thickness to the layer addition is 1:2 (to layers up to 100 mm thick). This proportion is based on translation proportions between a cracked asphalt layer (elasticity modulus of 300 MPa) and a new asphalt layer (elasticity modulus of 200 MPa). For general calculation it is assumed that the mean depth of deep milling will be 40 to 50 mm and therefore the addition will be 20 mm.

For a low distress level and the absence of fatigue distress, no surface preparation treatment is necessary.

A large-scale distress in the pavement causes unevenness to its surface and therefore requires surface preparation treatment in the form of scraping and local repairs in any case, regardless of the structural requirements.

By definition, surface treatment is applicable only in case of no or minimal distress and is not appropriate when a structural strengthening is required.

ECONOMIC EVALUATION AND PRIORITY DETERMINATION

The proposed approach enables one to carry out a detailed economic evaluation and provide indicators for setting up priorities and selecting maintenance treatment alternatives. Principle parameters taken into account are periodic maintenance treatment or rehabilitation costs, routine maintenance costs, and vehicle operation and travel time costs of road users. The difference between future costs "with project" and "without project" was calculated for the planning period; on this basis the net present value (NPV) and the internal rate of return (IRR) were determined for each road section. The priority order of sections was based on IRR.

Future vehicle operation costs flow depends on the anticipated annual PSR. The future routine maintenance flow depends upon distress development in the coming years. To refer to future vehicle operation and routine maintenance cost flows, assumptions were made concerning the character of the deterioration curves. The basic assumption is that cost savings in the first year as a result of the proposed treatment will be kept the same in future years. In other words, it was postulated that the degeneration rate of a treated road, and the same road if not treated, will be such that the distress difference between the two conditions remains constant. To validate this assumption, sensitivity tests were performed to express different deterioration rates of the parameters taken into account.

Maintenance Treatment Costs

The periodical maintenance treatment costs were derived from the solution selected for each road section by the Decision Tree and based on local unit costs. The routine maintenance costs were estimated on the basis of visual distress surveys (DR), according to the Washington method. Because the experience of the authors shows that DR value is not sufficient to determine a section's routine maintenance cost, distress components derived from the DR method were used. Treatment cost was estimated separately for each distress according to its severity. The quantity of required patching was associated also with the road traffic level in a nonlinear function type AX^b ($b < 1$).

Vehicle Operation Costs

Road users' cost (vehicles' operation costs + travelers' time cost) were calculated with the World Bank's HDM-III model (*16,17*) using coefficients found in studies made in Brazil. In this model the roughness input data are in terms of international roughness index (IRI), whereas in the present study the PSR index was used. So that the HDM-III model could be used, the PSR data were transformed into IRI by means of a conversion curve shown in Figure 3 (*18*). The economic operation costs for representative vehicles were calculated for three options: (a) variable costs only (gasoline, oil, tire, and vehicle maintenance), (b) total cost excluding travel time, and (c) total cost including travel time. The obtained operation cost functions for an average representative vehicle are as follows (see Figure 4):

$$Y_1 = 895 - 508(\text{PSR}) + 234(\text{PSR})^2 - 50.6(\text{PSR})^3 + 4.1(\text{PSR})^4 \qquad (6)$$

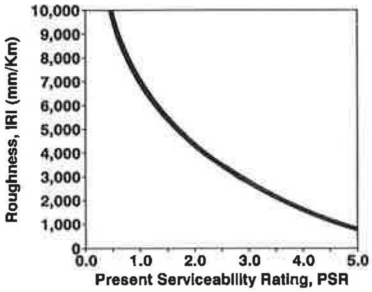

FIGURE 3 Conversion curve between IRR and PSR.

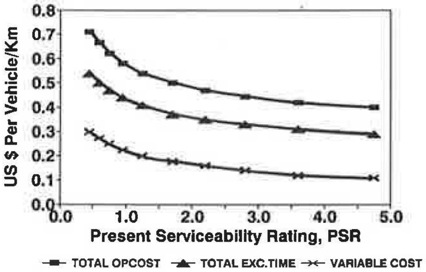

FIGURE 4 Average vehicle operation cost by PSR level.

$$Y_2 = 668 - 355(\text{PSR}) + 158(\text{PSR})^2 - 33.8(\text{PSR})^3 + 2.7(\text{PRS})^4 \qquad (7)$$

$$Y_3 = 398 - 267(\text{PSR}) + 118(\text{PSR})^2 - 25.1(\text{PSR})^3 + 2.0(\text{PSR})^4 \qquad (8)$$

where

Y_1 = total economic vehicle operation costs (OPCOST in Figure 4) including passenger travel time per 1000 vehicle-km (in U.S. dollars).
Y_2 = total economic vehicle operation costs excluding passenger travel time (EXC.TIME) per 1000 vehicle-km (in U.S. dollars).
Y_3 = total variable economic vehicle operation costs (gasoline, oil, tire, and vehicle maintenance) per 1000 vehicle-km (in U.S. dollars).

It is assumed that under the local conditions a PSR target of 3.5 after road treatment completion is achievable. Yearly economy on operation costs due to periodic maintenance treatment was calculated as the difference between operation costs at the present roughness rate and that obtained at PSR = 3.5.

APPLICATION OF METHOD

The method was applied to 170 road sections totaling 900 km (*12*). An example of the evaluation results for the 10 sections with the highest IRR is shown in Table 3. Through this table it is possible to understand in brief the decision-making process as related in this paper.

Columns 1 and 2 contain the identification data of the road sections. By means of DR data in Column 3, the sections were first classified according to the DR value above and below 80. The second classification cycle was performed according to fatigue distress percentage in Column 4 into sections having fatigue distress percentage above and below 20 percent. The third cycle was again performed by sorting sections into three DR subgroups. By data of the seventh deflection D_6, SCI, and vehicle traffic level shown in Columns 5, 6, and 7, respectively, the recommended overlay thickness is calculated. The overlay thickness is modified (Column 8) according to roughness results (Column 9) and the treatment type is de-

TABLE 3 EXAMPLE OF PARAMETERS AND RESULTS (FIRST TEN RANKED SECTIONS)

No.	Section No.	Average DR	Fatigue Cracking (%)	Average D6 (micron)	Average SCI (micron)	AADT (thousands)	Overlay Thickness (mm)	Average PSR	Treatment Code	Length (km)	NPV ($ millions)	IRR
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	105	53	18	75	302	29	60	1.73	5	6.4	10.3	819
2	103	71	11	67	278	22	60	1.66	4	1.6	2.2	734
3	59	53	10	39	190	11	40	1.97	5	4.4	2.1	723
4	21	33	26	40	206	40	100	1.97	8	2.8	4.8	479
5	22	50	13	28	116	34	60	2.40	5	1.7	1.7	477
6	44	64	5	53	270	19	60	1.92	5	1.5	1.4	460
7	52	52	16	68	266	35	100	2.09	5	3.7	5.3	452
8	45	56	7	51	313	42	90	2.36	5	4.3	5.4	451
9	11	36	24	80	349	38	130	1.81	8	4.6	9.1	433
10	70	31	24	46	150	51	110	2.32	8	1.1	1.8	424

termined (Column 10) according to the modified overlay thickness and the decision tree branch by which the section was classified.

The economic evaluation results, NPV and IRR, are shown in Columns 12 and 13. These indicators are the outcome of periodic and routine maintenance and vehicle operation cost flows with and without project. The planning period considered was 5 years.

In tests between the required structural strengthening thickness and DR and PSR values, no correlation was found (see Figure 5 and 6). At the same time, no significant correlation between IRR, PSR, and traffic levels were found (Figure 7 and 8). This phenomenon emphasizes the opinion of the authors that the use of only one or two parameters does not permit a reliable decision about the required treatment type to be used, nor does it render information about the feasibility or priority order for section treatment.

SUMMARY AND CONCLUSIONS

Applying the proposed method to the analyzed road network points to several conclusions:

1. To obtain reliable maintenance treatment decisions, the engineering evaluation must be based on three parameters—

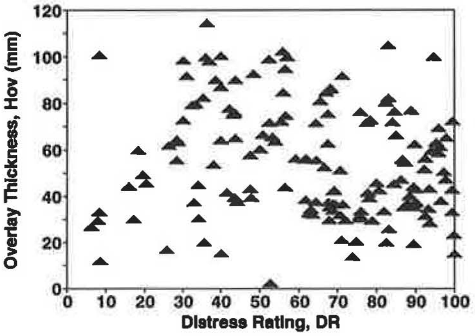

FIGURE 5 Correlation between overlay thickness and DR.

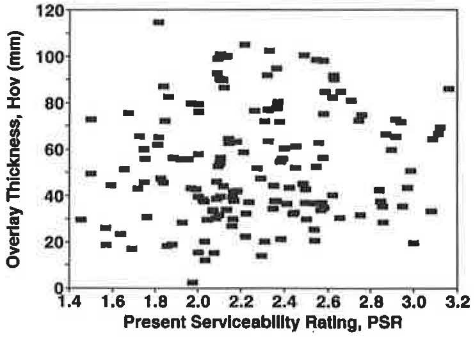

FIGURE 6 Correlation between overlay thickness and PSR.

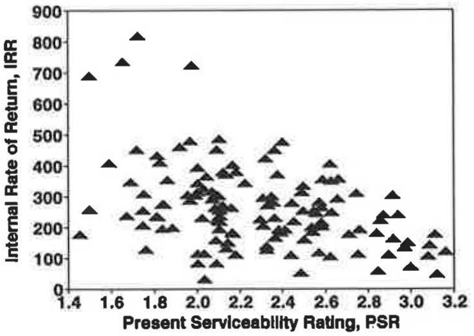

FIGURE 7 Correlation between IRR and PSR.

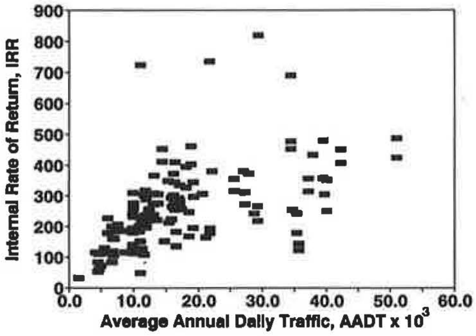

FIGURE 8 Correlation between IRR and AADT.

structural capacity, visual distress, and roughness level—in addition to traffic level. In testing correlation between final results and each parameter individually, none was found, proving the inability to rely on only two of them. In the same way, traffic or roughness level are not sufficient to determine investment priorities.

2. The decision tree method by which each index is evaluated at different intersection enables one to emphasize the meaning of each index within the area of its specialization.

3. To be of use, the DR index must be traced into distress factors to differentiate between fatigue distress and other distresses. This index does not contain rutting data, drop of shoulders, or quantitative distress measurement.

4. In a system with a relatively short planning horizon (5 years in this example) and great difference of roughness rate among the various sections, the deterioration curve shape has little influence for priority determination. The reason for this is that the initial absolute saving gaps between a condition "with project" and "without project" are much more dominant than their development rate in the future.

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Summary Models of Paved Road Deterioration Based on HDM-III

WILLIAM D. O. PATERSON AND BUSBY ATTOH-OKINE

Two generalized models predicting roughness progression in flexible pavements are developed from the comprehensive and widely validated set of incremental and interactive pavement distress functions in the Highway Design and Maintenance Standards model. These are summary models intended for use in pavement management applications and as a performance model for pavement design. The first retains most of the powerful capability of the original incremental model and has a very close fit to it, using traffic loading, strength, age, environment, rutting, cracking, and patching to predict roughness at any pavement age. Variants, which can be used when one or more of the distress parameters are missing, are also presented. The second model is simpler and generally structural, omitting surface distress parameters and compensating for this through the primary structural, traffic, age, and environmental factors. It is adequate for use where moderately good maintenance standards are being applied but has an error six times larger than the full model when extended over the full range of distress.

Developing balanced expenditure programs for a highway network requires predicting the rate of deterioration of the pavement and the nature of the changes in its condition so that the timing, type, and cost of maintenance needs can be estimated. A deterioration model, or pavement performance model, is therefore a key component of the analysis supporting decision making in pavement management.

If the model is to be useful for evaluating the primary options available in maintenance and rehabilitation, it must show explicitly the primary effects of traffic, pavement strength, age, distress, and environment on the trend of condition. Then the tradeoffs between the intervention options of minimal maintenance, patching, resurfacing, or strengthening at different times and condition levels can be properly compared. This is particularly true if the model is to be used also for estimating the cost-share of various classes of road user. Many models of roughness or cracking developed from local performance data, however, are either time-based models or forced to be traffic-based because they can capture only limited effects. Such models are generally incapable of distinguishing all the desired factor impacts and are applicable only in limited conditions.

The search for a mechanistic pavement performance model has been elusive because the causes of pavement deterioration are complex; interaction between different modes of surface distress and maintenance inputs influences the progression of rutting and roughness, as do aging and the environment. The Road Deterioration and Maintenance submodel of the World Bank's Highway Design and Maintenance Standards model (HDM-III) (*1*) is a comprehensive model that comes close to this goal because it quantifies these interactions and predicts all modes of distress and the impacts of maintenance. Formulated on mechanistic principles and developed from a broad empirical data base of a major international study, the model has been widely validated on data from several countries and has proved to be highly transferable (*2*). It provides detailed life-cycle simulation of physical conditions within a full economic evaluation model and has been applied in pavement management, highway planning, and highway economic evaluation in more than 40 countries.

The submodel quantifies all the primary effects, including the concurrent effects of trafficking and aging through an incremental recursive approach, calculating the change in each mode of distress sequentially for each year of the analysis period. Such a simulation approach requires an appreciable amount of computing time and capacity when applied to a large number of pavement sections and technical options, which is typical for network-level programming of maintenance and rehabilitation. Applications of HDM-III to pavement management for thousands of pavement sections have been limited by this time requirement. Thus, there is a strong need for simpler algorithms that approximate the primary effects captured by the full recursive model and permit rapid prediction of pavement roughness from a small number of primary parameters. If the reduced predictive accuracy has a negligible impact on the technical strategies chosen, then the improvement in computational speed will be highly advantageous.

The purpose of this study, therefore, was to develop summary algorithms for predicting pavement roughness that would be universally applicable and serve as a primary performance model for pavement management forecasting or a pavement design method. Basing these on the HDM-III model makes the results of that major international pavement research more generally available and produces a simple alternative for HDM users that is largely compatible with the full HDM-III model. The summary model is to predict absolute roughness rather than incremental roughness so it can serve as an independent performance model.

MODELING APPROACH

In the HDM-III submodel, the pavement condition and change in condition are predicted year by year in the model for each mode of distress in the following sequence:

1. Surface age for initiation of all cracking (of width 1 mm and wider) and the increment in area of all cracking (if either

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the age is greater than the initiation age or cracking is already present), which are functions of surface type, annual equivalent single axle loads (ESALs), and pavement strength;

2. Initiation and increment in area of wide cracking (wider than 3 mm) similar to Item 1;
3. Initiation and increment in area of raveling (loss of surface stone), which are functions of age and annual heavy vehicles (not ESALs);
4. Initiation and increment in total area of all potholes, which are functions of existing surface distress and annual vehicles (not loading);
5. Increment in rut depth (mean and standard deviation), which is a function of the strength, ESALs, age, cracking, precipitation, rehabilitation status, and pavement compaction; and
6. Increment in roughness, which is a function of strength, ESALs, age, environment, cracking, roughness, and changes in rut-depth standard deviation, cracking, potholing, and patching.

The net results of this deterioration simulation are curves for roughness that show a fairly distinctive two-phase character, as seen in Figure 1. Before cracking, the rate of roughness progression is relatively slow. It is driven by deformations related to structure and components related to environment but not to traffic. Under light loadings the structural component is extremely small but roughness continues to develop through the environmental-age component (moisture and temperature cycles). Under traffic loadings that are heavy relative to the pavement strength, the structural component is large. After cracking has initiated, the rate of roughness progression increases—and increases still faster when potholing begins. Patching largely, but not completely, compensates for the latter increase. More-detailed presentations may be found in works by Paterson (*2,3*).

At the time of the original model development, various summary model forms, including those of existing pavement performance models, were evaluated on the field data, but all were found to be significantly inferior to the detailed recursive model that was finally developed. The various statistical reasons for this include

- The state of surface distress was found to have a profound effect on the roughness but was not included in traditional model forms and complicated the form substantially;
- Age had a significant effect on roughness progression, through environment and largely non-traffic-related effects in addition to traditional parameters;
- Both age and cumulative traffic loading affected the roughness but were themselves correlated, and the loading effect was diminished by the compensating influence of pavement strength, which made it difficult statistically to distinguish the main effects; and
- Cross-sectional effects in the data, related to the different time windows and condition of the various pavements, confounded the estimation of traditional models such as the AASHTO performance model by producing a rate of deterioration that apparently decreased over time but conflicted with the shape of the time-series data for the individual sections that showed distinctly increasing rates.

Although a model was derived from the field data after resolving those problems [see a detailed discussion by Paterson (*2*)], experience has shown that it contains significant bias and has limited reliability compared with the full stimulation model.

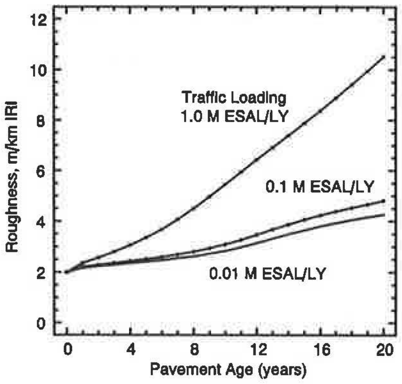

FIGURE 1 Example of roughness data generated by HDM-III [SNC = 3.0; wet, nonfreeze climate (m = 0.023); LY = land year].

GENERATION OF DATA

In this study, therefore, an alternative approach is used to develop a summary model. The approach uses the full empirical simulation model to generate roughness data for a wide range of the primary parameters and estimates the summary models by fitting the generated data. The confidence in the generated data stems from the extensive verification of the empirical simulation model and the comprehensiveness of the interactive form of the primary parameters. Roughness data were generated using the new RODEMAN version of the HDM-III model. RODEMAN is a menu-driven PC version of the Road Deterioration and Maintenance submodel of HDM-III that produces the same detailed pavement results as HDM-III. It also includes simplified vehicle operating cost and other cost functions, which enable it to calculate the main economic parameters, although in less detail than HDM-III.

The actual data generated are discrete annual values of each parameter and distress mode as indicated in Table 1. Data were generated for an array of two pavement types (asphalt concrete and surface treatment flexible pavements) and three primary variables (pavement strength, annual traffic loading, and environment) for the side ranges shown in Table 2. Loadings ranged from 10,000 to 3 million ESAL/lane-year; strength from 2 to 8 modified structural number; and environment from arid, nonfreezing (m = 0.005) to wet, freezing (m = 0.10), where m is an environmental parameter. There were 33 combinations and a 20-year analysis period, resulting in 693 observations for each pavement type. For this application, the patching of all potholes as they appeared was adopted as the basic maintenance strategy.

TABLE 1 EXAMPLE SUBSET OF DATA GENERATED BY HDM-III MODEL

SNC	Traffic veh/d	Loading M ESAL/ lane-yr	AGE (yr)	CRA (%)	CRW (%)	CRX (%)	RAV (%)	PHA[1] (%)	RDM (mm)	RDS (mm)	RI IRI[2]	PAT (%)
3	1000	0.10	0	0	0	0	0	0	0	0	2.0	0.00
			1	0	0	0	0	0	3.0	1.3	2.21	0.00
			2	0	0	0	0	0	3.4	1.4	2.28	0.00
			3	0	0	0	0	0	3.8	1.5	2.36	0.00
			4	0	0	0	0	0	4.1	1.5	2.43	0.00
			5	0	0	0	0	0	4.3	1.6	2.51	0.00
			6	2	0	1	0	0	4.5	1.6	2.59	0.00
			7	6	0	4	0	0	4.7	1.7	2.69	0.00
			8	12	3	8	0	0	4.9	1.7	2.80	0.00
			9	20	9	16	0	0	5.1	1.7	2.94	0.00
			10	31	19	26	0	0	5.2	1.7	3.09	0.00
			11	44	31	39	0	0	5.4	1.8	3.27	0.00
			12	59	45	54	0	0.03	5.6	1.8	3.47	0.03
			13	72	61	68	0	0.04	5.8	1.9	3.68	0.07
			14	82	74	80	0	0.06	6.0	1.9	3.87	0.13
			15	90	85	89	0	0.07	6.2	2.0	4.06	0.20
			16	95	93	95	0	0.08	6.4	2.0	4.23	0.28

1. Potholing area data shown is prior to patching, and is reduced to zero annually by the patching.
2. 1 m/km IRI = 63.36 inch/mi IRI.

Note: AGE = age of pavement since resurfacing, yrs; CRA = area of all cracking, %; CRW = area of wide cracking (3mm and wider), %; RAV = area of ravelling, %; PHA = area of potholing, %; RDM = mean rut depth, mm; and other terms are defined with eq. (1).

TABLE 2 COMBINATION AND RANGES OF PRIMARY PARAMETERS USED TO GENERATE CONDITION DATA

				Traffic Loading (million ESAL/lane-yr)					
Environment	Surface Type	SNC	Surface Thickness	0.01	0.03	0.10	0.30	1.0	3.0
DNF (0.005)	AC	2	30	x		x			
		3	50	x		x		x	
		5	80		x		x		x
		8	100		x		x		x
	ST	2	12	x		x			
		3	12	x		x		x	
		4	15		x		x		x
		6	18		x		x		x
WNF (0.023)	AS FOR DNF ABOVE								
WF (0.100)	AS FOR DNF ABOVE								

Note: DNF = Dry, non-freeze; WNF = wet, non-freeze; WF = wet, freeze; ESAL = equivalent standard axle loadings (8,200 kg); AC = asphalt concrete; ST = surface treatment.

FORMULATION OF MODELS

In modeling the performance over the pavement life cycle, the important features to capture are the two phases of deterioration rate, before and after cracking, and the different mechanisms causing roughness. This complicates the formulation, but the original HDM-III algorithm does this well by relating the change in roughness to three separate mechanisms, namely,

incremental roughness = structural deformation (function of modified structural number, incremental traffic loadings, extent of cracking and thickness of cracked layer, incremental variation of rut depth) + surface defects (function of changes in cracking, patching and potholing) + environmental and non-traffic-related mechanisms (function of pavement environment, time and roughness)

Thus, the starting point for the new model formulation was the multiparameter incremental model for roughness (*2,3*) in HDM-III and RODEMAN. The integral form of the model with respect to time is the following:

$$\mathrm{RI}_t = e^{mt}[\mathrm{RI}_0 + 134\,\mathrm{SNCK}^{-4.99}\,\mathrm{NE}_t] + 0.114\,\mathrm{RDS}_t + 0.0066\,\mathrm{CRX}_t + 0.16\,\mathrm{PHV}_t + 0.01\,\mathrm{PAT}_t \qquad (1)$$

where

RI_t = roughness at pavement age t [m/km international roughness index (IRI)];
RI_0 = initial roughness (m/km IRI);
NE_t = cumulative ESALs at age t (million ESAL/lane);
t = pavement age since rehabilitation or construction (years);
m = environmental coefficient (0.023 for wet, non-freeze climate in the original estimation);
SNCK = $(1 + \mathrm{SNC} - F\,\mathrm{HS}\,\mathrm{CRX}_t)$;
SNC = structural number modified for subgrade strength;
F = coefficient that was 0.0000758 in original incremental model and that in integrated form is ap-

proximated by half that value, that is, 0.00004 (see discussion in text);

HS = thickness of bound layers (mm);

CRX_t = area of indexed cracking at time t (%), in which the areas of each class of cracking are weighted by the crack width (2 mm for narrow cracks and 4 mm for wide cracks);

RDS_t = standard deviation of rut depths (mm);

PHV_t = volume of potholing (m^3/lane-km);

PAT_t = area of patching (%).

The first terms, comprising the function in brackets, represent the primary performance function comparable to the AASHTO and other performance models in which roughness progression is purely structural and a function of cumulative traffic loading. However, unlike the AASHTO-type models, that term is a linear function of cumulative loadings because the acceleration of deterioration that occurs over time is reflected in the separate distress parameters of the model. Other noteworthy features are the time-environment factor, e^{mt}, which introduces a nontraffic time-related component, and the distress terms that introduce a component, quantifying the superficial effects that can be altered through surface maintenance. The mathematical form of Equation 1 is a slight approximation because the rutting, cracking, and pothole distress terms are traffic-dependent and ideally would have been within the bracketed function if the original incremental relationship had been purely integratable with respect to time. This is a second-order effect that would have required only a small change to the original incremental form and would not change the original coefficient estimates by much. The SNCK term is also an approximation because the cracking parameter, CRX, is both time-dependent and discontinuous, for which there is no simple integral form. A value half of the original estimate is a theoretically close approximation, that is, 0.00004.

The pavement performance model estimated directly from the field data in the original study (*2*) focused just on the structural-time term, as follows:

$$RI_t = e^{0.0153t}[RI_0 + 725\,(1 + SNC)^{-4.99}\,NE_t] \qquad (2)$$

The exclusion of the distress terms was a convenience for the user to avoid having to estimate all modes of distress. A subsequent estimation from generated data using the technique of this paper found values of 0.035 and 190 for the first two coefficients and kept the third coefficient fixed as −5 (*4*). In these cases, the models are compensating for the exclusion of the other terms by major changes in the time-environment coefficient m, from −30 percent to +50 percent compared to the original value of 0.023. Because this misrepresents the distress effects as environmental, it is not satisfactory.

Two sets of models were formulated for the new estimation. In the first, the integrated form of Equation 1 was estimated to verify the coefficients and produce an absolute value version of the multiparameter roughness model that could be used in studies in which distress data could be used but only a single relationship was desired. Some variants of this, omitting certain terms, were developed for cases where not all distress parameters are available. In the second set, all effects were concentrated in the general structural performance terms, similar to Equation 2, and the variants sought to improve the fit and reliability of the model without introducing other distress measures. The models tested are given in Table 3. The statistical analysis program used was Statgraphics Versions 5 (*5*).

MODEL ESTIMATES

The results of linear multiple regression for the full generalized model, including distress parameters, are presented in Table 4. Here it can be seen that Model A.1, which is a direct estimate of Equation 1, has coefficients extremely close to the original estimates and extremely tight, with standard errors of less than 0.2 percent of the coefficient. Only the patching coefficient is different in relative terms, but this is still numerically very small. The error of estimate of 0.084 m/km IRI of the model is very small, and the fit (R^2 = .9997) is extremely tight. These inferences were confirmed by a study of the residuals. Thus, Model A.1 is considered a highly satisfactory absolute form of the original incremental model. It will have primary uses in deterioration prediction when cracking and rutting distress data are available and in design when the maintenance intervention level can be defined in terms of these parameters.

Other variants of the model in Table 4 represent cases in which some of the distress parameters are omitted. Model A.4 shows that the mean rut depth can be substituted for the standard deviation of rut depth with very little loss of precision. This is primarily because of the functional similarity of the algorithms in the HDM-III model, but there is also a generally high correlation in practice except where there are substantial differences in rut depth and variability between wheel tracks. Model A.2, which omits the rut-depth term, is

TABLE 3 SIMPLIFIED MODEL FORMS OF ROUGHNESS PROGRESSION

Name	Parametric Form		Range
A	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC- F\,HS\,CRX)^{-5}\,NE_t]$ $+ c\,RDS_t + d\,CRX_t + e\,PAT_t$	$0 \leq CRX \leq 100$
B	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC)^{-5}\,NE_t] + c\,RDS_t$ $+ d\,CRX_t + e\,PAT_t$	$0 \leq CRX \leq 100$
C	RI_{unc}	$= e^{mt}\,[RI_0 + a\,(1+SNC)^{-5}\,NE_t]$	$0 \leq CRX \leq 5$
D	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC- F_i\,HS\,CRX)^{-5}\,NE_t]$	$0 \leq CRX \leq 100$
E	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC)^{-5}\,NE_t]$	$0 \leq CRX \leq 100$
F	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC- f\,HS\,CRX)^{g}\,NE_t]$	$0 \leq CRX \leq 100$
G	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC)^{g}\,NE_t]$	$0 \leq CRX \leq 100$
H	RI_t	$= e^{mt}\,[RI_0 + a\,(1+SNC)^{g}\,NE_t^{b}]$	$0 \leq CRX \leq 100$

Note: Variable names are defined in text with Eq. (1).

TABLE 4 ESTIMATES FOR GENERAL MODEL WITH DISTRESS PARAMETERS

Model	erg0	esnk4	RDS	RDM	CRX	PAT	SEE.	R^2	D-W
A.1	0.980 *0.001*	132 *0.6*	0.143 *0.003*	-	0.0068 *0.0001*	0.056 *0.0003*	0.084	1.000	0.335
A.2	1.013 *0.002*	147 *1.0*	- -	- -	0.0090 *0.0001*	0.058 *0.0005*	0.147	0.999	0.241
A.3	0.990 *0.005*	187 *2.9*	-	-	0.0118 *0.0004*	-	0.471	0.989	0.233
A.4	0.984 *0.001*	130 *0.7*	-	0.046 *0.001*	0.0067 *0.0001*	0.057 *0.0003*	0.088	1.000	0.311
A.5	0.955 *0.006*	173 *3.8*	-	0.107 *0.005*	-	-	0.515	0.987	0.196

Notes: All coefficient estimates have significance level better than 0.00005. Italics = standard error of coefficient. SEE = standard error of estimate, m/km IRI. R^2 = adjusted coefficient of determination. D-W = Durbin-Watson statistic. Number of observations = 1274 for all models. $\text{erg0} = e^{m'} RI_0$. $\text{esnkf} = e^{m'} SNCK_f$. F = f/100,000. RDM = rut depth mean. $SNCK_f$, RDS, CRX, and PAT are defined with eq.(1). For model A.1 R^2 = 0.9997, and for Model A.4 R^2 = 0.9996.

also strong and shows that good predictions of roughness can be made by adding at least the amount of cracking to the structural factors in the prediction. Omitting the patching terms as well (Model A.3), however, greatly increases the error of prediction in the upper range when potholing and patching become substantial contributors to the level of roughness. Finally, adding just rut depth to the structural terms of the primary model, as shown in Model A.5, is satisfactory but poorer than any of the other forms. These observations are confirmed by the statistical analysis of variance of the variables in Model A.1.

Model forms that concentrate on the primary structural function and attempt to compensate for the development of surface distress and the effects of surface maintenance within the traffic, strength, or time terms are shown in Table 5. In Models D.0 to D.2, the impact of the degradation in effective structural number due to cracking (represented by the SNCK term with different values of the coefficient *F* of Equation 1) is shown, when all other distress parameters are omitted. In these instances, the power values of the structural number and traffic loading terms were held at the original values. It is evident that the fit is good but clearly poorer than the A-models in Table 4; standard errors range from 0.57 to 0.67 m/km IRI. The best of these models in fact is Model D.0, which shows no degradation effect in the structural number.

Study of the residuals shows that the predictions of the D-models degrade for extremes of heavy or light trafficking relative to the pavement strength. Model H shows, however, that the nonlinear form does not produce a better result than Model D.0, which has the original power values for strength and loading. But it does indicate a greater sensitivity of deterioration to the relative levels of pavement strength and traffic loading and approaches closer to the rho and beta parameters of the AASHTO performance model. This appears to indicate that the strong nonlinearity in the form of the AASHTO performance model of serviceability is primarily a surrogate for the concurrent development of other modes of pavement distress that accelerate the evolution of pavement roughness. The results for Model C, which was estimated from only observations with cracking less than 5 percent, show that the primary structural term in Model A.0 and Equation 1 is adequate and valid for predictions for the period before cracking occurs.

The results presented were derived from the combination of all the data generated according to Table 2. Separate estimations were also made for each pavement type, but the differences due to pavement type were not significant. The incidence of potholing and patching was highest in the surface treatment data, however, so that had a dominant influence in determining the coefficient of the patching term. This does

TABLE 5 ESTIMATES OF MODELS WITH PRIMARY STRUCTURAL FUNCTION ALONE

Model	erg0	$esnk_0$	$esnk_4$	$esnk_8$	g	h	SEE	R^2	D-W
C^a	1.024 *0.002*	211 *4.8*	-	-	-5^b	1.0^b	0.106	0.999	0.403
D.0	1.039 *0.005*	273 *3.2*	-	-	-5^b	1.0^b	0.571	0.984	0.211
D.1	1.046 *0.005*	-	236 *3.0*	-	-5^b	1.0^b	0.606	0.982	0.218
D.2	1.056 *0.006*	-	-	201 *2.9*	-5^b	1.0^b	0.665	0.979	0.222
H	1.111 *0.012*	237 *73*	-	-	-6.89 *0.30*	1.786 *0.056*	0.878		

Notes: General notes as for Table 4. a. Valid for indexed cracking less than 5 percent of area, from 651 observations. b. Value of parameter was fixed as for original incremental model, not estimated. g,h. Coefficients are defined in Table 3, Models F, G, and H.

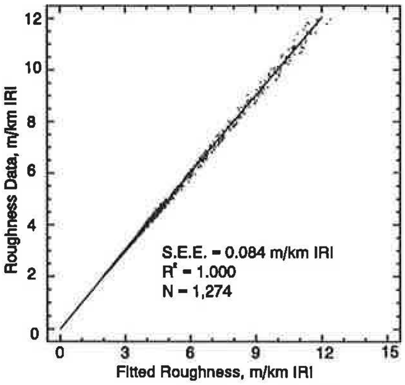

FIGURE 2 Fit of best generalized model: full model with distress parameters (Model A.1; data include all values of pavement type, strength, loading, and environment).

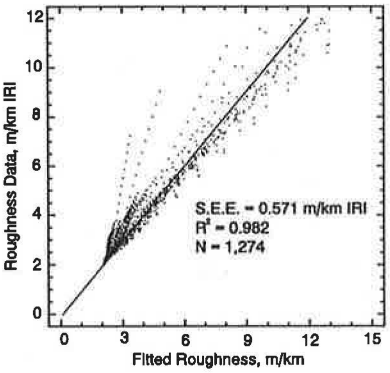

FIGURE 3 Fit of alternative simple model: basic structural model (Model D.0; data include all values of pavement type, strength, loading, and environment).

not imply that the deterioration models are overinfluenced by surface treatment pavements at all. In fact, it indicates that satisfactory performance predictions of roughness for a variety of pavement types and environments can only be made when the concurrent evolution of other modes of surface distress is included in the predictive model.

The predictive fits of the two best generalized models are shown in Figure 2 for the full model with distress parameters (Model A.1) and in Figure 3 for the basic structural model (Model D.0). It is evident that Model A.1 is superior in explaining the trends for the large majority of the factor combinations. The differences between the two that give rise to the respective predictive errors are apparent in the few combinations in which Model D.0 is under- or overestimating the rate of deterioration.

PREFERRED MODELS AND THEIR USES

For applications in which data or predictions of rutting, cracking, and patching are available, the recommended model is A.1, namely,

$$RI_t = 0.98\, e^{mt}[RI_0 + 135\ SNCK_4^{-5}\ NE_t] + 0.143\ RDS_t + 0.0068\ CRX_t + 0.056\ PAT_t \quad (3)$$

where $SNCK_4 = 1 + SNC - 0.00004\ HS\ CRX_t$ for $HS\ CRX_t < 10{,}000$.

Predictions of this model for three levels of traffic loading are shown in Figure 4, alongside the observed data that were used to generate the model. The adherence of the predictions to the data is seen to be strong, and the model thus shows the change in roughness progression that occurs once surface distress commences, similar to the original model. Any valid predictions of cracking and rutting can be used with the model in Equation 3, because the underlying model is based on observed cracking and rutting behavior and is not dependent on any particular set of cracking and rutting models. Inputs from field data or other cracking and rutting models will produce a trend that differs slightly from that shown in the figure (which assumes the HDM-III and rutting predictions) and that will be more valid if the cracking and rutting inputs are more valid for the application than those from HDM-III.

When a general model is required without knowledge of the surface distress, then the preferred model is D.0, as follows:

$$RI_t = 1.04\, e^{mt}[RI_0 + 263\ (1 + SNC)^{-5}\ NE_t] \quad (4)$$

Some predictions of this model are shown in Figure 5, also against the generating data to illustrate the adherence of the model to the originating trends. As the model does not account well for the impact of high levels of surface distress, it should preferably be applied for predictions of roughness of flexible pavements that are maintained at low amounts of cracking, up to about 30 percent (from a comparison of the model with the originating data).

Some guidance on quantifying the environmental effects is important to applications of Equations 3 and 4. First, the SNC value represents the in situ strength of the materials, not a conservative soaked value or design value. Thus, impacts of drainage on the California bearing ratio of the subgrade and

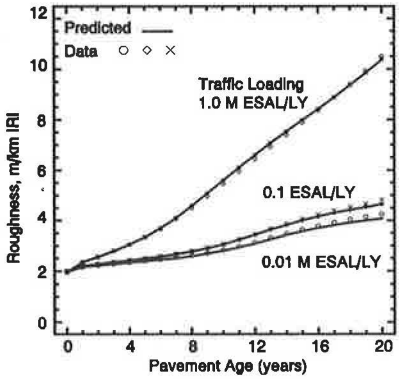

FIGURE 4 Example of predictions from full model with distress parameters (Model A.1; SNC = 3.0; *m* = 0.023; LY = lane year).

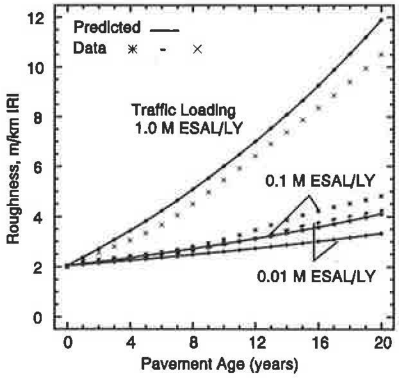

FIGURE 5 Example of predictions from basic structural model (Model D.0; SNC = 3.0; *m* = 0.023; LY = lane year).

modulus of the pavement layers should be included in the SNC value. Where strong seasonal variations occur, a weighting (by inverse fifth power) must be applied to obtain the effective annual average SNC value. Second, the value of the coefficient m varies with climate approximately as follows:

- Dry, nonfreeze: $m = 0.005$ to 0.015 (0.010),
- Dry, freeze: $m = 0.010$ to 0.035 (0.020),
- Wet, nonfreeze: $m = 0.015$ to 0.030 (0.023), and
- Wet, freeze: $m = 0.030$ to 0.150 (0.070).

The value increases with increased rainfall or diurnal temperature differences, and the values in parentheses are typical for the zonal classification. As it really represents the immediate environment of the pavement, the presence of specific environmental design provisions, such as material selection to avoid frost-susceptibility, free-draining materials, and sealed shoulders, are expected to be reflected by a reduction in the effective value of m. However, valid empirical guidance quantifying these effects is not available at this stage. The origin and further discussion of the m-values may be found in work by Paterson (*2*).

CONCLUSION

The strongest summary performance model is that in Equation 3, which predicts the roughness of flexible pavements from traffic loading, pavement structural number, age, environment, rutting, cracking, and patching. It is a successful and very strong representation of the original incremental, multidistress model incorporated in HDM-III. It is useful when distress data are available in a road data base through road monitoring and when other predictive models for rutting, cracking, and patching are available. The validity of the predictions does not depend on the use of the distress models of HDM-III. The accuracy of fit was 0.08 m/km IRI for the generated model, but the predictive accuracy overall for empirical data was about 0.55 m/km IRI (including the predictive error of the original model).

It is generally applicable to flexible pavements but most accurate for those maintained before the area of cracking exceeds about 30 percent. The overall predictive error for empirical data (taking into account the predictive error of the original model) is likely to be about 1.0 m/km IRI over the full range up to 12 m/km IRI, but the error reduces to about 0.6 m/km IRI for maintained pavements. Other model forms similar to the AASHTO performance model did not fit the generated data as well as the generalized model because of their exclusion of the nontraffic, environmentally related components.

The two summary models are suited for applications to pavement performance prediction in pavement management systems and economic evaluation analyses. On the basis of the wide validation of the originating HDM-III road deterioration model, especially its mechanistic component form, these simplified models too can be expected to be valid in most countries and environments. They are limited to flexible and semirigid pavements and are primarily valid for pavements with asphalt less than 150 mm thick. In addition to the published climatic effects on the environmental coefficient, drainage provisions may reduce the coefficients through their effect on the microenvironment of the pavement.

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Identifying Error-Generating Factors in Infrastructure Condition Evaluations

FRANNIE HUMPLICK

Infrastructure surface inspection and condition rating systems used today range from detailed automated inspections that use photographic and laser technologies to manual inspection that uses the human eye. The capability of these systems in measuring distressed areas varies because of several factors, including the principle of measurement, type of inspection strategy, manner of data reduction, and objectivity of data collection. The characteristics of the objects being measured and the surroundings in which they are inspected also affect the results. The types of errors affecting inspection results are presented, as is a set of hypotheses derived from theoretical expectations of the effect of the mentioned factors on the accuracy of inspection systems. These hypotheses are tested using data from state-of-the-art inspection systems. The conclusions are useful for designing, improving, and choosing systems and for adjusting inspection results for improved accuracy.

A variety of infrastructure inspection systems currently exists (*1–8*). These systems range from detailed automated inspections using photographic and laser technologies to manual inspection using the human eye. The capabilities of such systems in locating, recognizing, discriminating, and distinguishing among distresses, as well as scaling their size, extent, and severity, depend on a variety of factors. These include the principle of measurement, type of inspection strategy, and manner of data collection and reduction. Inspection results are also affected by the characteristics of the objects being measured, which create confounding measurement scenes for the inspection systems and hence limit their accuracy. Finally, the surroundings of the measured objects also affect inspection.

This paper discusses the types of error affecting the results of inspection and presents a set of hypotheses derived from theoretical expectations of the effect of characteristics on the accuracy of inspection systems. These hypotheses are then tested using data from a FHWA study entitled *Improved Methods and Equipment to Conduct Pavement Distress Surveys* (*4*). This data set will be referred to as the FHWA data set.

CLASSIFICATION OF ERRORS OF INSPECTION

A typical inspection process consists of the facility under inspection and the inspection system. Inspection errors originate from the inspected facility and inspection system, as well as the interface between them.

Intrinsic or Inherent Errors

Intrinsic errors are inherent in the inspected facility and the inspection system. They can be observed in laboratory or experimental conditions when all known influencing factors are controlled. In such situations the same object measured repeatedly by an inspection system can result in almost the same measured quantity or in highly varying measured quantities.

The first case is characteristic of most mechanical gauges, such as roughness measurements on highway pavements, where the required response from the inspection system is well known and can be determined. The second situation occurs when the response from the inspection system is not well behaved—that is, the results of measurement in the second situation are so varied that one cannot predict the underlying true value of the object without further knowledge about the distribution of the measured values or the causes of the discrepancies. The latter case is the most common in infrastructure condition evaluation, because the inspection systems used have multiple components, some of which are not well tested or designed. It has been observed that repeated measurements by the same system are highly variable, and the measured values by different systems of the same sections are even more variable (*4*).

The following forms of error can cause differences in observed results. These have been adapted from a work by Finkelstein (*9*) and are generalized to account for the inspection systems common in infrastructure condition evaluations:

1. Zero error occurs when the inspection system outputs a value even when there is no event present. For example, if a video inspection of an undistressed pavement results in a fixed or variable value of measured distress each time it is used, it is an indication of zero error. That is, the video inspection will always result in a value of distress even when it is not present. The analogy to a gauge-type measurement is that the gauge will have a misplaced zero position, reading a fixed value even before it has been applied in measurement.
2. Dynamic error results during the operation of an inspection system such as inspecting pavement surfaces with truck-mounted photographic equipment. The error in data recording with respect to location on the pavement is considered a dynamic error. The measured value of distress in this case is a function of the speed of the data acquisition (e.g., the shutter speed of a camera) and the speed of the operating vehicle.
3. Quantization or categorization errors result from inspection systems in which the measurement response changes

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in discrete steps. For example, digitizing intensity values on a piece of film for future processing, by grouping observations into ranges of intensity values and assigning a value to each range, can result in a quantization error. This error type also appears in visual inspections in which distresses are categorized into ranges, resulting in a categorization error.

4. Viewing limitation errors occur when an inspection system can view only a fixed portion of the inspected facility or when detection capability is limited to a certain range. For example, a photographic camera has a fixed field of view and resolution that can be achieved in practice, limiting the size of the facility surface that can be viewed and the size of the objects detected. An analysis of the impacts of such error types can be found elsewhere (*10*).

Influence Errors

Influence errors arise during interaction between the inspection and inspected systems. They are caused by factors in the inspection environment that were not controlled in an experimental setting during the design of an inspection system. For example, a film of water on a pavement surface being inspected after a rainy period can change the reflectance property of the pavement surface. The system may not be designed to account for such a change.

Such errors can arise at the output interface and include the data reduction format used and whether individual, range estimates, or average values are reported. Alternatively, factors at the input interface such as the sampling strategy, characteristics of the inspected facility (the nature of the measured surface), and properties of the measured objects (pattern of occurrence or dimensions of the objects) may cause errors. Finally, uncontrolled influences due to departure from design or calibrated conditions can result in influence errors.

Uncontrolled influences affecting the results of inspection arise from the inspection environment. These include (a) mechanical vibrations during data collection affecting the relative position of measuring equipment; (b) electrical and thermal changes influencing the behavior of measuring equipment; (c) events occurring on the inspected facility that were not planned for or are not part of the measurement, such as shadows, oil spots, and debris, which can confound automated systems; and (d) fatigue due to long hours of operation of equipment or humans, inducing measurement errors.

Intrinsic and influence errors have a systematic component and a random component. Systematic errors are errors that can be predicted from past knowledge or use of an inspection system. For example, if it is known that a human inspector tends to add a fixed amount to all measurements, the measurements can be corrected by this amount, which can be obtained from past observations. Random measurement errors, on the other hand, are those that cannot be predicted on an individual measurement basis but that can be statistically estimated from multiple measurements. These errors are due to short-term variations of factor influencing measurement. For example, if an inspection system is repeatedly measuring the same object using the same procedures, the scatter of the measured value, caused by temporal and local variations in influences, is a realization of random measurement errors.

Formulation of Measurement Problem

The difference between the result of measurement and the true value of the measured quantity is an error of measurement. This error can be defined as ε_t where

$$\varepsilon_t = d - d^* \tag{1}$$

and d and d^* represent the measured and true values, respectively.

The true value is the quantity that would be obtained from a perfect measurement. Because all measurements are subject to error, the true value is essentially unknown (latent). For calibration purposes, an approximation obtained from a measurement deemed close enough to the true value for all practical purposes is often used. For example, inspection systems of moderate accuracy are calibrated against systems of high accuracy, by using the measured quantities by the high accuracy system as the "true values" [see Jeyapalan, Cable, and Welper (*11*)].

Let us denote the values of the error-generating factors that an inspection system encounters during measurement by θ. Measuring an object with a true value d^* with these settings of the error-generating factors results in

$$d = f(d^*, \theta) \tag{2}$$

If we denote the values of the error-generating factors for which an inspection system has been designed and calibrated for by θ^r, then the influence error can be defined as the difference between the resulting measurements from the actual and design settings. This can be expressed as follows:

$$\varepsilon_{\text{inf}} = f(d^*, \theta) - f(d^*, \theta^r) \tag{3}$$

where ε_{inf} measures the departure from design conditions.

From the definition of total error in equation 1, and denoting the error due to the measuring principle as ε_m, we have

$$\varepsilon_t = \varepsilon_{\text{inf}} + \varepsilon_m$$

These errors are additive, because they are assumed independently of each other; that is, removing the effect of an influence error (such as the resolution limitation of a camera) does not result in a change in the error due to the measurement principle, which remains essentially the same (indirect). Hence, the effect of the influence errors and the errors due to the measurement principle are additive.

A measurement principle can be direct or indirect. In direct measurement the inspection system receives as input properties of the measured object (e.g, the length and width of a crack) and gives as output a measure of these properties (e.g., length in feet). Visual inspection by human beings is a form of direct measurement. In indirect measurement the inspection system receives as input the properties of the measured object, which it senses depending on the measurement principle employed, and converts the information into signals that represent a proxy for these properties. These signals are then processed. The processing transforms the sensed proxy into measures or properties of the original object through some mapping function, which then outputs the measured value.

A rule system relates the numerical value obtained as output to the value of the properties of the object input. Transferring intensity values on a piece of film to measures of distress is an example of such mapping.

This process leads to two components of errors due to measuring principle: data acquisition and data processing. In general, a measurement process can be represented as

$$d^{*} \rightarrow \delta \rightarrow d$$

where δ is a proxy of the actual object realized as an intermediate step. Thus, the data acquisition error can be expressed as

$$\delta = h_1(d^{*}) + \bar{\varepsilon}_{da} + \bar{\varepsilon}_{\text{inf}} \tag{4}$$

and the data processing error can be expressed as

$$d = h_2(\delta) + \varepsilon_{dp} \tag{5}$$

where h_1 (.) is a transformation mapping the true distress value into a proxy (the measured value of distress) and $h_2(.)$ maps the measured proxy of distress into the inspection output.

Substituting Equation 4 into Equation 5 gives

$$d = h_2[h_1(d^{*}) + \bar{\varepsilon}_{da} + \bar{\varepsilon}_{\text{inf}}] + \varepsilon_{dp}$$

Assuming without loss of generality that the mapping from d^{*} to δ is linear,

$$h_1(d^{*}) = \alpha + \beta d^{*} = x \qquad \text{then}$$

$$h_2(\delta) = \left(\frac{\delta - \alpha}{\beta}\right) \qquad \text{and}$$

$$d = \frac{h_1(d^{*}) + \bar{\varepsilon}_{da} + \bar{\varepsilon}_{\text{inf}} - \alpha}{\beta} + \varepsilon_{dp}$$

$$= \frac{\alpha + \beta d^{*} - \alpha + \bar{\varepsilon}_{da} + \bar{\varepsilon}_{\text{inf}}}{\beta} + \varepsilon_{dp}$$

$$= d^{*} + \frac{\bar{\varepsilon}_{da}}{\beta} + \varepsilon_{dp} + \frac{\bar{\varepsilon}_{\text{inf}}}{\beta}$$

Defining the following,

$$\varepsilon_{da} = \frac{\bar{\varepsilon}_{da}}{\beta}$$

$$\varepsilon_{\text{inf}} = \frac{\bar{\varepsilon}_{\text{inf}}}{\beta}$$

we obtain

$$\varepsilon_t = d - d^{*} = \varepsilon_{da} + \varepsilon_{dp} + \varepsilon_{\text{inf}} \tag{6}$$

where $\varepsilon_m = \varepsilon_{da} + \varepsilon_{dp}$, and ε_{da}, ε_{dp}, and ε_m are data acquisition, data processing, and measurement principle errors, respectively.

Not all the errors in Equations 1 through 6 can be determined determined empirically. Particularly, the error due to data acquisition in Equation 5 cannot be ascertained because it is related to an intermediate value whose relationship to the original input is not well studied. For photographic inspection of pavement surfaces, the relationships between the intensity values on a piece of film and the true values of distress on a surface are not well studied. Procedures to identify and classify these intensity values in terms of distresses occurring on a surface are still in an experimental stage (*6*). Therefore it is not possible to determine the data acquisition error for such systems from analytical deductions.

A generalized measurement error model specified from the error structures derived thus far is expressed as

$$d_{ijk} = f(d^{*}_{ik}, \theta_i, \theta_j, \theta_k) \tag{7}$$

where

d_{ijk} = measured distress on Section i of Distress Type k by Inspection System j;
$f(.)$ = function representing the relationship between the measured distress, the true value of distress, and factors affecting the measurement;
d^{*}_{ik} = unobserved true value of distress of Type k on Section i; and
θ_i, θ_j, and θ_k = vectors representing error-generating factors from the inspection environment (section), inspection system (technology), and the measured objects (distresses), respectively.

Without loss of generality, we can express the function $f(.)$ in Equation 7 in a linear form with respect to the true distress, as is commonly done for calibration purposes.

$$d_{ijk} = \alpha_{jk} + \beta_{jk} d^{*}_{ik} + \varepsilon_{ijk} \tag{8}$$

where α_{jk}, β_{jk}, and ε_{ijk} are the systematic additive, systematic multiplicative, and additive random error of Inspection System j while measuring distress of Type k in Section i.

HYPOTHESIS DEVELOPMENT AND TESTING

The function in Equation 8 was estimated for a variety of inspection systems, distress types, and pavement sections using the FHWA data set (*4*). Details of the estimation can be found in unpublished data by Ben-Akiva and Humplick. Empirical results from the estimation are used in this section to investigate the effects of error-generating factors on measurements results.

The FHWA data set included measurements by seven inspection systems measuring distresses in experimental units consisting of three pavement types (flexible, composite, and rigid) and three condition levels (good, moderate, and poor). The inspection systems are a manual mapping method, detailed visual surveys using manual recording, automated data logging, the GERPHO device (Photo1), the PASCO Roadrecon survey vehicle (Photo2), the ARAN survey vehicle (Video), and the Laser RST device (Laser). These systems will be referred to as "Mapping," "Manual," "Logging," "Photo1," "Photo2," "Video," and "Laser." For a detailed description of these systems and the type of measurements they performed, see the work by Hudson et al. (*4*).

Hypotheses About Inspection System Characteristics

Two hypotheses were developed to test for the impact of inspection system characteristics on measurement accuracy.

Effects of Inspection Strategy and Data Reduction Format

The manner in which data is reduced and the percentage area of pavement inspected are expected to affect measurement results. An inspection system that either views less than 100 percent of the pavement surface or reports average values or ranges of distress values for a given section is expected to be less accurate than one that views 100 percent of the section and measures individual distress elements. The inspection systems used in the FHWA study can be grouped as

- Total area observed, individual measures of distress made, or both (Manual, Photo1, and Photo2); and
- Sample area observed and range or average values reported (Mapping, Logging, Video, and Laser).

The distinction is made because the inspection systems falling into the second group require some kind of estimate to obtain the total value of distress on a section. The types of errors in the inspection results of the second group that are not present in those of the first group may be due to extrapolating from a small sample size, or averaging by eye. To test whether there is a difference between the inspection results of the systems in the two groups, the following hypothesis is set up

$$H_0 : \beta_{\mathrm{Man}} = \beta_{\mathrm{Photo1}} = \beta_{\mathrm{Photo2}} \quad \text{and}$$

$$\beta_{\mathrm{Map}} = \beta_{\mathrm{Log}} = \beta_{\mathrm{Video}} = \beta_{\mathrm{Laser}} \qquad (9)$$

Effect of Data Collection Process

The type of data collection process employed, whether objective or subjective, is expected to affect the accuracy of the results of measurement. In particular, inspection systems making objective measurements are expected to have smaller random biases than those based on subjective rankings, unless they suffer from interpretation problems; then the random biases would be large. For example, objective measures of alligator cracking obtained by a photographic technique should have less variation than those obtained from an eye estimate by a human inspector. However, one expects more classification errors to affect the photographic technique, because there is no visual verification of the types of distress present. Subjective evaluations are expected to measure the extent of distress less accurately, but they suffer less from interpretation and classification errors. Inspection systems using both subjective and objective measures should, therefore, have more accurate results.

From the descriptions of the inspection systems in the FHWA study, they can be grouped as follows:

- Objective: Mapping, Photo2;
- Subjective: Manual, Logging; and
- Objective and subjective: Photo1, Video, Laser.

The Photo1, Video, and Laser used both subjective and objective measures to estimate the level of distress on a section. The following hypotheses are set up:

$$H_0 : \beta_{\mathrm{Man}} = \beta_{\mathrm{Log}}$$

$$\beta_{\mathrm{Map}} = \beta_{\mathrm{Photo2}}$$

$$\beta_{\mathrm{Photo1}} = \beta_{\mathrm{Video}} = \beta_{\mathrm{Laser}} \qquad (10)$$

The hypotheses in Equations 9 and 10 can only be tested for inspection systems in which all other factors affecting measurement accuracy are similar or insignificant.

Table 1 shows the organization of the FHWA data according to the factors mentioned. From this table we can test for the effect of the data reduction format and inspection strategy by comparing Manual to Logging and Photo1 to Video. We can test the data collection process by comparing Photo1 and Photo2 and Mapping to Logging.

A paired Tukey test was selected to perform multiple comparisons of the estimated bias parameters. The Tukey test is constructed as follows: assume the multiplicative biases $\hat{\beta}_j$ for the inspection systems $j = 1, \ldots, J$ are distributed with a mean $\overline{\beta}$ and variance σ^2_{β}. The range of the $\hat{\beta}_j$s is

$$R = \max_j \hat{\beta}_j - \min_j \hat{\beta}_j \qquad (11)$$

Let s^2_{β} have an estimator of σ^2_{β} having ν degrees of freedom, and assume s^2_{β} and $\hat{\beta}_j$ are independent. Then $Q_{J,\nu} = R/S$, where J is the number of inspection systems being compared and S, the standard error of the βs, is Student t distributed. The confidence interval for the $(\hat{\beta}_j - \hat{\beta}_l)$, where $j \neq l$, taking into account that all possible comparisons can be made, is given by Larsen and Marx (*12*) and Box et al. (*13*).

TABLE 1 INSPECTION SYSTEM CHARACTERISTICS IN FHWA DATA

	Inspection System Characteristics		
Inspection system (j)	Measurement Principle	Data Reduction & Inspection Strategy	Data Collection Process
Mapping	-	+	O
Manual	-	-	S
Logging	-	+	S
Photo1	+	-	C
Photo2	+	-	O
Video	+	+	C
Laser	+	+	C

NOTE:

Measurement principle	Data reduction & Inspection strategy	Data collection process
- = direct	- = individual/total	O = objective
+ = indirect	+ = average/range/sample	S = subjective
		C = combined

$$(\hat{\beta}_j - \hat{\beta}l) \pm \frac{q_{J,\nu,\alpha/2}}{\sqrt{2}} s \sqrt{\frac{1}{n_j} + \frac{1}{n_l}} \qquad (12)$$

where

$\frac{q_{J,\nu,\alpha/2}}{\sqrt{2}}$ = a tabulated upper significant value of the Studentized range for J variables and ν degrees of freedom.

One would reject the hypothesis that the parameters are equal if zero is not contained within this interval. The Tukey test was performed using the estimated values of β_j in Table 2. The results of the Tukey test are shown in Table 3.

The hypothesis on the equality of the multiplicative biases was rejected for all pairs of inspection systems except Mapping and Manual, Mapping and Logging, Mapping and Video, Mapping and Photo1, Manual and Video, and Logging and Photo1. The effect of the data reduction format and inspection strategy captured by the difference ($\beta_{\mathrm{Man}} - \beta_{\mathrm{Log}}$) and ($\beta_{\mathrm{Photo1}} - \beta_{\mathrm{Video}}$) was found significant, as the hypothesis that these differences are zero was rejected. Similarly, the effect of the data collection process, represented by the difference ($\beta_{\mathrm{Photo1}} - \beta_{\mathrm{Photo2}}$), was found significant. However, it was found insignificant for ($\beta_{\mathrm{Map}} - \beta_{\mathrm{Log}}$). This discrepancy may be because the Photo1 and Photo2 inspection systems employ photographic imaging techniques with the same measurement principle, so the effect of the data collection process is more pronounced than when Mapping and Logging are compared. They are both direct measurement technologies that use human inspectors, but they differ extremely in the manner in which data is actually collected: Mapping uses a sampling strategy and measures each individual distress on a sample unit to get an estimate of distress on the section, whereas Logging observes the entire section but gives a range estimate of distresses on the section.

Pairwise differences were computed for the random bias parameters using the results in Table 2. The Tukey interval was estimated, and the results are shown in Table 4. This test resulted in rejecting 6 out of 15 pairwise differences. The pairs

TABLE 2 ESTIMATED BIASES FOR DIFFERENT INSPECTION SYSTEMS (ALLIGATOR CRACKING ON FLEXIBLE PAVEMENTS)

	Estimated Parameters (standard errors of the estimates)			
Inspection system (j)	$\bar{\alpha}_j$ (Sqft)	$\hat{\beta}_j$	$S.D.(\epsilon_{ij}) = \sqrt{\psi_j}$ (Sqft)	Coeff. Of det. R^2
1. Mapping	-73.0 (474.2)	0.83 (0.17)	396.9	0.94
2. Manual	37.5 (363.7)	0.49 (0.10)	262.9	0.94
3. Logging	570.0 (845.1)	1.29 (0.54)	646.2	0.61
4. Photo1	-154.5 (527.0)	1.09 (0.21)	444.5	0.95
5. Photo2	-501.3 (551.6)	1.85 (0.23)	551.3	0.99
6. Video	134.0 (474.2)	0.44 (0.17)	472.3	0.77
7. Laser[a]	---	---	---	---

[a] The Laser Inspection system did not report measures for alligator cracking as denoted by --- in the table.

TABLE 3 TUKEY TEST FOR EQUALITY OF MULTIPLICATIVE BIASES (ALLIGATOR CRACKING ON FLEXIBLE PAVEMENTS)

Hypothesis Tested	Results of Test
$\beta_{map} = \beta_{man}$	0.34 Accept
$\beta_{map} = \beta_{log}$	-0.46 Accept
$\beta_{map} = \beta_{photo1}$	-0.26 Accept
$\beta_{map} = \beta_{photo2}$	-1.02 Reject
$\beta_{map} = \beta_{video}$	0.39 Accept
$\beta_{man} = \beta_{log}$	-0.80 Reject
$\beta_{man} = \beta_{photo1}$	-0.60 Reject
$\beta_{man} = \beta_{photo2}$	-1.36 Reject
$\beta_{man} = \beta_{video}$	0.05 Accept
$\beta_{log} = \beta_{photo1}$	0.20 Accept
$\beta_{log} = \beta_{photo2}$	-0.56 Reject (not significant)
$\beta_{log} = \beta_{video}$	0.85 Reject
$\beta_{photo1} = \beta_{photo2}$	-0.76 Reject
$\beta_{photo1} = \beta_{video}$	0.65 Reject
$\beta_{photo2} = \beta_{video}$	1.41 Reject
95% Tukey Interval	±0.57
99% Tukey Interval	±0.71

TABLE 4 TUKEY TEST FOR PARAMETER EQUALITY—RANDOM BIAS (ALLIGATOR CRACKING ON FLEXIBLE PAVEMENTS)

Hypothesis Tested	Results of Test
$\sigma^2_{map} = \sigma^2_{man}$	99.5 Accept
$\sigma^2_{map} = \sigma^2_{log}$	-480.1 Reject
$\sigma^2_{map} = \sigma^2_{photo1}$	76.3 Accept
$\sigma^2_{map} = \sigma^2_{photo2}$	423.3 Reject
$\sigma^2_{map} = \sigma^2_{video}$	-200.8 Accept
$\sigma^2_{man} = \sigma^2_{log}$	-380.6 Reject
$\sigma^2_{man} = \sigma^2_{photo1}$	175.8 Accept
$\sigma^2_{man} = \sigma^2_{photo2}$	522.8 Reject
$\sigma^2_{man} = \sigma^2_{video}$	-101.3 Accept
$\sigma^2_{log} = \sigma^2_{photo1}$	556.4 Reject
$\sigma^2_{log} = \sigma^2_{photo2}$	903.4 Reject
$\sigma^2_{log} = \sigma^2_{video}$	279.3 Accept
$\sigma^2_{photo1} = \sigma^2_{photo2}$	347.0 Accept
$\sigma^2_{photo1} = \sigma^2_{video}$	-277.1 Accept
$\sigma^2_{photo2} = \sigma^2_{video}$	624.1 Reject
95% Tukey Interval	±422.25
99% Tukey Interval	±508.11

σ^2_j is the variance of measurement by inspection system j.

of inspection systems whose random biases are statistically different are

- Mapping and Logging,
- Mapping and Photo2,
- Manual and Logging,
- Manual and Photo2,
- Logging and Photo1,
- Logging and Photo2,
- Logging and Video, and
- Photo2 and Video.

This indicates that the Logging and Photo2 inspection systems have random biases that are significantly different in nature from those of the other inspection systems. For the Photo2 system this may be because alligator cracks and other crack types were jointly reported. For Logging, the difference may be due to the averaging of range estimates of distress that are reported instead of the individual values of distress on a section. However, more pairs of random biases were found statistically equal (9 out of 15 hypotheses were accepted) than were pairs of multiplicative biases (3 out of 15 hypotheses on equality of pairs of biases were accepted). A possible explanation for this difference is that the impact of factors affecting measurement results may more seriously affect multiplicative biases than random biases. This is a useful finding, because one can correct the measured results for systematic biases using the results of a calibration and hence only worry about minimizing random error.

The results of these tests indicate that the inspection systems used in the FHWA data had varying capabilities in measuring alligator cracking (represented by the multiplicative bias β_j). The most distinct inspection systems were Logging and Photo2; their parameters were statistically different from those of the other systems. The main differences between these inspection systems and the others is the limitations of the Logging device, which cannot measure individual distresses and reports ranges of distress instead, and of the Photo2 system, which jointly measures alligator and other areal distresses such as block cracking and patched cracks.

Hypotheses About Distress Characteristics

The distress characteristic that could be tested using the FHWA data set is the dimension of distress, mainly whether linear, area, or volumetric. Table 5 compares the estimated multiplicative biases β_j for distresses with these dimensions on flexible pavements. measures of volumetric distresses showed a range in values (0.34 to 2.09) larger than the ranges of linear (0.43 to 1.48) and areal (0.44 to 1.85) distresses. In general there was an increase in the range of parameter estimates as the number of distress dimensions increased, with the lowest range being for the case of linear distresses. This indicates that the inspection systems are measuring areal and volumetric distresses in a dissimilar way as compared to linear distresses. These differences are mainly due to the additional complexity of the measurement scene when volumetric distresses are involved.

TABLE 5 HYPOTHESES ABOUT DISTRESS CHARACTERISTICS (VARIOUS DISTRESSES ON FLEXIBLE PAVEMENTS)

	Estimated Multiplicative Biases (standard errors of the estimates)		
Inspection System (j)	Longitudinal and Transverse Cracking (linear)	Alligator Cracking (areal)	Potholes and Patches (volumetric)
1. Mapping	0.95 (0.26)	0.83 (0.17)	--
2. Manual	--	0.49 (0.10)	0.61 (0.36)
3. Logging	1.48 (0.65)	1.29 (0.54)	0.37 (0.15)
4. Photo1	0.43 (0.40)	1.09 (0.21)	2.09 (0.20)
5. Photo2	--	1.85 (0.23)	1.59 (0.12)
6. Video	1.17 (0.64)	0.44 (0.17)	0.34 (0.16)
7. Laser	0.98 (0.45)	--	--
Range in estimates of β_j(Max − Min)	(0.43 - 1.48) 1.05	(0.44 - 1.85) 1.41	(0.34 - 2.09) 1.65

-- denotes no parameter estimates for the given technology.

A Tukey test on the equality of the estimated multiplicative bias parameters when measuring different distress types was performed. This test involves the comparison of 7 × 7 = 49 pairs of parameters for each distress dimension, which leads to 49 × 3 = 147 pairs. Only the differences between the same inspection systems for each distress dimension are of interest. These are presented in Table 6, in which they are compared to the Tukey intervals at 95 and 99 percent confidence.

The hypothesis on the equality of parameters was accepted for all the differences between multiplicative biases for linear versus areal distresses and rejected for all differences for linear versus volumetric distresses. This indicates that there is an effect of distress dimension on the results of measurement.

The effect of distress dimension is statistically significant especially for the inspection systems employing optical techniques (Photo1, Photo2, and Video). This result is expected, because the complexity of measurement due to distress dimension is supposed to affect optical techniques more than techniques (such as inspection by humans) that do not depend

TABLE 6 TUKEY TEST FOR EFFECTS OF DISTRESS CHARACTERISTICS

	Results of Hypothesis Test		
Hypothesis Tested	Linear Vs. Areal	Linear Vs. Volumetric	Areal Vs. Volumetric
$\beta_{map} - \beta_{map}$	0.12 Accept	--	--
$\beta_{man} - \beta_{man}$	--	--	-0.12 Accept
$\beta_{log} - \beta_{log}$	0.19 Accept	1.11 Reject	0.92 Reject
$\beta_{photo1} - \beta_{photo1}$	-0.66 Accept	-1.66 Reject	1.00 Reject
$\beta_{photo2} - \beta_{photo2}$	--	--	0.26 Accept
$\beta_{video} - \beta_{video}$	0.73 Accept	0.83 Reject	0.10 Accept
$\beta_{laser} - \beta_{laser}$	--	--	--
Tukey 95% confidence interval	±0.81	±0.54	±0.51
Tukey 99% confidence interval	±1.02	±0.66	±0.65
Ratio of number rejected	0/4	3/3	2/5

– denotes no parameter estimates for the given technology pair.

on a signal (such as intensity of light) coming from the measured object. The nonoptical inspection systems (Mapping, Manual, and Logging) showed no significant difference between the estimated multiplicative biases.

Hypotheses About Section Characteristics

Section characteristics are captured by three factors: the pavement type, the contrast between distresses and their background, and the pattern of distress occurrence. In the FHWA data there were three types of pavement: rigid, flexible, and composite. Rigid pavements can be categorized as having high contrast and a systematic pattern of distress occurrence. Flexible and composite pavement can be categorized as having moderate to low contrast and a haphazard pattern of distress. To test for the joint effect of the contrast and pattern of distress occurrence, one can use the pavement type as a proxy.

The following general hypothesis can be stated on the basis of these section characteristics:

> Inspection systems are equally efficient in detecting and measuring distresses (capability) but distresses differ in their "detectability." That is, if a distress is in a section with high contrast and a systematic pattern of distress occurrence, it is more easily detectable by a given inspection system than if it is in a section with low contrast and a haphazard pattern of distress.

Therefore, the contrast and pattern of distress occurrence characterize the detectability, and capability is represented by the estimated measurement biases α_j and β_j.

The hypothesis that can be tested is whether there is a difference in inspection system biases when measuring distresses from backgrounds with different contrast and pattern of distress occurrence. This hypothesis is tested for situations in which the distresses have the same dimension, to exclude the effects of interaction between distress and section characteristics. Linear distresses on flexible, rigid, and composite pavements were used.

The following unconstrained model system was specified:

$$\begin{aligned} d_{ij1} &= \alpha_{j1} + \beta_{j1} d_{i1}^* + \varepsilon_{ij1} \\ d_{ij2} &= \alpha_{j2} + \beta_{j2} d_{i2}^* + \varepsilon_{ij2} \\ d_{ij3} &= \alpha_{j3} + \beta_{j3} d_{i3}^* + \varepsilon_{ij3} \end{aligned} \tag{13}$$

where

$\alpha_{j1},\alpha_{j2},\alpha_{j3},\beta_{j1},\beta_{j2},\beta_{j3}$ = additive and multiplicative errors for inspection system j when measuring distresses on pavement types 1 (flexible), 2 (rigid), and 3 (composite), respectively; and

$d_{i1}^*,d_{i2}^*,d_{i3}^*$ = true value of distress on section i for pavement types 1, 2, and 3, respectively.

The hypothesis that there is no difference in inspection system biases when measuring distresses from backgrounds with different contrast and pattern of distress can be stated as follows:

$$\begin{aligned} &H_0: \alpha_{j1} = \alpha_{j2} = \alpha_{j3} \\ &\beta_{j1} = \beta_{j2} = \beta_{j3} \\ &[\mathrm{var}(\varepsilon_{ij1}) = \psi_1] = [\mathrm{var}(\varepsilon_{ij2}) = \psi_2] = [\mathrm{var}(\varepsilon_{ij3}) = \psi_3] \end{aligned} \tag{14}$$

To test the hypothesis in Equation 14, the unconstrained model in Equation 13 is estimated to get the values of the parameters β^{UC}_j and ψ^{UC}_j. The true values of distress are extracted using latent variable estimation techniques described in the unpublished data by Ben-Akiva and Humplick, which will be denoted by $(d_i^*)^{UC}$ for each pavement type. Then the observed values of distress on the three pavement types are stacked and a constrained model is estimated, as shown.

$$d_{ij}^C = \alpha_j^C + \beta_j^C (d_i^*)^C + \varepsilon_{ij}^C \tag{15}$$

where the superscript C denotes the results of estimation from the stacked data.

The constrained model in Equation 15 represents the hypothesis

$$\alpha_1 = \alpha_2 = \alpha_3 = \alpha^C$$

$$\beta_1 = \beta_2 = \beta_3 = \beta^C$$

$$\psi_1 = \psi_2 = \psi_3 = \psi^C$$

Similarly, the true values of distress $(d_i^*)^C$ can be extracted from estimated values of β_j^C and ψ_j^C. Because the d_i^*s for the constrained and unconstrained models are estimated from the respective β_js and ψ_js, one can compare the extracted d_i^*s to make inferences about the β_js and ψ_js. This is a preferred procedure because it does not require computation of the error sum of squares, which is tedious to calculate and is required for any test on the equality of the β_js and ψ_js.

The following regression is performed for all three pavement types:

$$\underset{(n_z \times 1)}{d_{uc}^*} = \gamma \underset{(n_z \times 1)}{d_c^*} + \underset{(n_z \times 1)}{\varepsilon} \tag{16}$$

where

z = 1, 2, 3 pavement types,
n_z = $n_1 + n_2 + n_3$, and
n_1, n_2, n_3 = number of flexible, rigid, and composite pavement segments, respectively.

The hypothesis that $\gamma = 1$ is then tested. If $\gamma \neq 1$, then there is a difference in inspection system bias when measuring distresses from backgrounds with varying contrast and pattern of distresses.

The results of the regression in Equation 16 are summarized in Figure 1. The hypothesis $\gamma = 1$ was accepted for the case of composite pavements and rejected for flexible and rigid pavements. This indicates that there is an effect of contrast and pattern of distress occurrence on the results of measurement.

Flexible Pavements

$$d_{UC} = \underset{(0.10)}{0.61}\, d_c \quad R-Square = 0.79$$

Rigid Pavements

$$d_{UC} = \underset{(0.11)}{0.91}\, d_c \quad R-Square = 0.89$$

Composite Pavements

$$d_{UC} = \underset{(0.57)}{1.18}\, d_c \quad R-Square = 0.46$$

Hypotheses

$$H_0 : \gamma_f = \gamma_r = \gamma_c = 1$$

f = flexible, r = rigid, c = composite

The statistic used for testing is:

$$t_{N-2} = \frac{\bar{\gamma} - \gamma_0}{S_{\bar{\gamma}}}$$

Results

Flexible Pavements
T-statistic = -3.90
Reject null hypothesis at 95% confidence interval

Rigid Pavements
T-statistic = -0.82
Reject null hypothesis at 95% confidence level

Composite Pavements
T-statistic = 0.32
Accept null hypothesis at 95% confidence interval

FIGURE 1 Results of hypothesis tests on effects of section characteristics.

CONCLUSIONS

A methodology for identifying the factors affecting the results of measurement was developed and tested using highway inspection data. The success of such a methodology depends on scientifically collected data, such as were generated by the experimental design presented by Hudson et al. (*4*). The methodology can be used to identify directions for future development of inspection technologies, to choose among existing inspection systems, and to correct inspection results for measurement errors.

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DISCUSSION

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The author has done an excellent job of presenting and applying a hypothesis-testing methodology to identify sources of error in the subjective manual, semiautomatic, and high-speed noncontact-type inspection systems for monitoring and evaluating pavement condition. The results discussed in this paper have important implications for selecting equipment and collecting data to evaluate pavement condition for infrastructure maintenance, preservation, and development. The following comments and discussion are related to the FHWA distress data base, on which the author relied to formulate the measurement error analysis problem and hypothesis testing.

The writer was one of the principal team members of the comprehensive FHWA study of pavement condition evaluation equipment (*1–3*) for which pavement test section selection and data collection were carried out with strict adherence to statistical experiment designs. The writer was primarily responsible for site selection and all field data collection.

This scientific study of equipment for and methods of monitoring pavement condition consisted of separate experiment designs for the following equipment categories: deflection, void detection, and distress survey.

Pavement nondestructive testing structural condition evaluation equipment included eight deflection devices (*1*):

- Slow-moving wheel load with manual data recording (dial gauge) and manual processing, requiring stops at the test locations (Benkelman beam).
- Continuously moving equipment with automated data recording by seismic geophone and automated data processing (Curviameter).
- Harmonic dynamic load equipment with automated deflection sensing by seismic geophone and automated processing, requiring stops at the tests locations (Dynaflect and Road Rater).
- Impact dynamic load falling weight deflectometer (FWD) equipment with automated data recording by seismometers or geophones and automated data processing, requiring stops at the test locations (three models of FWD and one replicate FWD unit).

The measurement of voids under concrete pavement for evaluation of structural integrity and assessment of concrete pavement restoration needs required very special equipment (*2*). The following devices were investigated for evaluating their capability of void detection and measurement of void size:

- Proof rolling and visual inspection,
- Deflection survey,
- Ground-penetrating radar equipment,
- Infrared thermography, and
- Transient dynamic response method.

Unfortunately, all of these methods required intensive manual data interpretation and special operator skills.

Seven varieties of equipment and methods were investigated for their suitability and reliability in distress survey and condition evaluation (*3*). Table 1 describes and groups the inspection system characteristics of these methods. Because the distress survey equipment is the subject of the paper, the measurement and processing principles of different distress data elements are summarized for the readers. The main differences among these methods are also highlighted.

- Mapping: Detailed direct manual measurements of all distress types including rutting by walking on selected inspection units within the pavement test section; procedure based on the AASHO Road Test distress mapping procedure; manual data processing. This is the method coded as Mapping.
- Manual visual surveys (PAVER/COPES): Detailed direct manual severity rating and extent measurement by walking using specific sampling and visual inspection guidelines for all distress types including rutting on selected inspection units within the pavement sections; manual data processing. This is the method coded as Manual.
- Semiautomated data logger: Measurements similar to detailed manual visual surveys by walking survey and entering data directly on a hand-held data logger (portable PC); automated data processing. This is the method coded as Logging.
- GERPHO: High-speed automatic imaging of pavement surface on continuous 35-mm photo film, at night only; manual distress data interpretation on full section length; no rutting data; automatic data processing. This is the method coded as Photo1.
- PASCO-Roadrecon survey vehicle: Multifunction high-speed automatic data collection; automatic data processing. Imaging of pavement surface on continuous 35-mm photo films at night only; manual distress data interpretation on full section length; automatic rutting data processing from digitized photo records of transverse profiles; longitudinal profile measurement by laser sensors. This is the method coded as Photo2.
- ARAN video condition inventory survey vehicle: Multifunction high-speed automatic data collection; automatic data processing; video imaging of perspective view and pavement surface; no interpretation of distress data from video. Windshield visual manual distress data collection, using integrated data logger on full section length; automatic rutting data processing from transverse profiles measured by ultrasonic sensors; longitudinal roughness measurement by accelerometer. This is the method coded as Video.
- Laser RST survey vehicle: Multifunction high-speed automatic data collection; automatic data processing; laser survey of pavement surface for measuring longitudinal and transverse profiles and texture data processing (only some transverse cracking data were produced from laser survey and no other distress data were interpreted from laser survey). Windshield visual manual survey for alligator, longitudinal, and edge cracking data and other distress data collection, using integrated data logger on full section length; automatic rutting data processing from transverse profiles measured by laser sensors. This is the method coded as Laser.

It is obvious from these comments that the alligator cracking, longitudinal cracking, and edge cracking data from both Video and Laser devices are essentially collected in the windshield-type visual survey mode using on-board integrated data loggers. These and other distress data, excluding rutting data, are visual, manual, subjective measurements reported by these high-speed multifunction devices. Therefore, these data are not expected to be of the same quality as the distress data processed from the Photo1 and Photo2 equipment, Logging, Manual, and Mapping methods.

The author is encouraged to examine rutting data for hypothesis testing. As described, rutting data were collected by objective measurements by Photo2, Video, and Laser. Direct manual objective measurement was used in Mapping, and subjective data collection procedures were used for rutting survey in Manual and Logging methods.

Pavement management system (PMS) development and implementation is a top priority area on federal-aid highway systems throughout the United States. Pavement condition data monitoring and evaluation, particularly distress data and rutting data, are integral components of the PMS process. Multifunction and high-speed equipment providing objective measurements of pavement condition data are attractive and cost-effective alternatives during the PMS equipment selection process (*4*). However, speed and productivity should not be the only selection criteria; quality and accuracy of pavement condition evaluation and prediction are important as well. Note that the maintenance need assessment and the maintenance work program and budgets depend on the quality of pavement condition data. The author is commended for bringing the subject of data quality and sources of error in distress survey methods and equipment to the attention of pavement community.

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AUTHOR'S CLOSURE

The discussant has provided a valuable description of the inspection technologies that is useful for interpreting the significance of the results of this paper. As mentioned in the conclusion, scientifically collected data, gathered by the methods used in the background papers presented by the discussant, are necessary input to the success of the methodology developed in this paper.

As suggested by the discussant, bias parameters were estimated for rutting data. The results are presented in Table 7. As can be seen from these results, all systems perform very well with respect to the standard error of measurement (which is practically zero for all systems). However, the inspection system with the lowest additive bias is Photo2, which underestimates rutting by only 0.01 in.

The impact of lack of objectivity is seen by an underestimation by Manual of 0.11 in., and the impact of system limitations (resolution) is exemplified by a 0.32 in. overestimation by Video. The systems with the least multiplicative bias, however, are Mapping, which underestimates rutting by a factor of only 0.02, and Logging, which overestimates rutting by a factor of 0.02. The seriousness of over- or underestimation depends on the use to which the data are put. A methodology for choosing among inspection technologies on the basis of their accuracy of measurement and whether the data are used to predict performance or make maintenance decisions can be found elsewhere (*1*).

The Photo2 technology has the highest multiplicative bias for rutting measurements, overestimating them by a factor of 1.23. However, this is not a problem, because the inspection results can be corrected for using the results of the calibration in Table 7. The results in Table 7 indicate that the benefits of direct measurement are undermined by the spatial variation of measurements that cannot be captured by sampling strategy employed by Mapping. However, these effects are additive in nature and hence can be factored out using the results of calibration. On the other hand, the advantages of automation (such as when using Photo2) can be achieved only if the results of measurement are corrected for error of inspection. The advantage of the rutting data is that all the technologies at the moment have insignificant random errors and, because one can correct for systematic errors, there should be no advantage other than cost and speed of data collection.

TABLE 7 ESTIMATED BIASES FOR DIFFERENT INSPECTION SYSTEMS (RUTTING ON FLEXIBLE PAVEMENTS)

	Estimated Parameters (standard errors of the estimates)			
Inspection system (j)	$\bar{\alpha}_j$ (Sqft)	$\bar{\beta}_j$	$S.D.(\epsilon_{ij}) - \sqrt{\psi_j}$ (Sqft)	Coeff. Of det. R^2
1. Mapping	-0.20	0.85 (0.36)	0.00	0.82
2. Manual	-0.11	0.81 (0.30)	0.00	0.86
3. Logging	0.07	1.14 (0.18)	0.00	0.85
4. Photo1	--	--	--	--
5. Photo2	-0.01	2.11 (0.21)	0.00	0.88
6. Video	0.32	0.31 (0.09)	0.00	0.76
7. Laser	-0.06	0.78 (0.27)	0.00	0.86

-- The Photo1 inspection system did not report measures for rutting as denoted by -- in the table. Additionally, the standard errors of the additive biases were not calculated as this is a very time consuming activity.

Since an unbiased system has a multiplicative bias of one, the degree of over or underestimation is calculated as $(\beta_j - 1)^2$, which is 0.02 for Mapping, 0.04 for Manual, 0.02 for Logging, -- for Photo1, 1.23 for Photo2, 0.48 for Video, and 0.05 for Laser respectively.

The author suggests the use of spatial models to estimate the impact of spatial effects on measurement errors. Such work is ongoing; preliminary results have been published by Koutsopoulos and Mishalani (*2*).

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Factors Affecting Condition of Pavements Owned by Local Governments

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In recent years pavement management systems have been made available to local governments. One beneficial feature of a pavement management system has been an increased interest in the condition of pavement sections and those factors that affect it. Consequently, factors affecting the condition of flexible pavements owned by local governments are investigated and identified. After a literature review was conducted, several hypotheses were established. Theoretical or functional relationships were formulated, and data files were described. After these files were procured, multivariate analyses were performed on the data and their results were validated. Several factors affecting the condition of pavements owned by local governments were identified. They included old (or previous) pavement condition score, age of pavement structure since last major work, soil classification (or type), classification of roadway drainage (presence or absence of curb and gutter), surface thickness, functional classification, presence or absence of bus service, and individual jurisdiction. Another conclusion was that actual pavement management system data files used for the statistical modeling with only minimal modification may contain limitations for modeling purposes. They include the need for another time domain in the data files so true time-series analyses can be done. Because local governments do not normally conduct traffic classification counts, the data files did not contain truck count data; such an addition would likely increase the accuracy of the models.

Most pavements owned by local governments were initially constructed with portland cement concrete, but most local agencies currently use asphalt concrete pavements. Local governments generally have several types of pavement. Many pavement sections have been constructed according to standard sections selected on the basis of average daily traffic (ADT). Some pavement sections were determined by assumed traffic loadings based on functional classification. There are also "evolved" pavement sections consisting of a series of surface treatments, such as several asphalt cement chip seals. Over many years, especially with little truck traffic, age hardening can produce structurally sound pavement sections; these are more likely found in rural counties. Sometimes asphalt concrete surfaces have been placed directly on the natural subgrade, which often results in the reduction of the pavement quality. Some local governments, especially older ones, have portland cement concrete pavements overlaid with asphalt concrete mixes. There are many miles of unsurfaced, aggregate roads, mostly in rural counties. The unsurfaced roadway is not evaluated in this research effort.

One unique circumstance is the strength of locally available aggregate used for asphalt concrete mix and aggregate base courses. The strength of the aggregate will probably vary from source to source. Without substantial testing it is difficult to predict the performance of pavement sections that are constructed of material from a particular source.

Finally, local agencies (and state departments of transportation) encounter many streets and roads with poor drainage. When drainage systems allow water to persist in the base material and heavy trucks pass over the pavement section, the water is forced to move. This movement causes migration of the fines in the base or subbase. The fines can actually be "pumped" to the surface, leaving voids. This weakens the structural integrity of the pavement section. This is usually more common with portland cement concrete pavements, but it can also be an issue with asphalt cement concrete pavements.

It would be helpful in identifying and programming maintenance-related activities if local governments knew which factors affected the condition of their pavements. These factors have been probed by Gibby in his unpublished dissertation (*1*). Additionally, some findings discovered with research supported by the California Department of Transportation (Caltrans) have been incorporated into this paper (*2*).

RESEARCH OBJECTIVES

The primary objective of this research was, through statistical modeling, to investigate and identify factors affecting the condition of street and road pavement sections owned by local governments. The analyses were designed to accommodate local circumstances such as the quality of the subgrade and drainage. Other objectives were also established, namely, to investigate the adequacy of local data files and to determine whether a model developed with data from one jurisdiction can be transferred to another jurisdiction.

RESEARCH APPROACH

Five steps were taken to accomplish the objectives of this research effort. They are (a) establishing hypotheses, (b) formulating theoretical or functional models, (c) acquiring and editing the data files, (d) applying multivariate analyses to the data files, and (e) interpreting results and evaluating hypotheses.

Relationships between pavement condition and several potential parameters were discovered from the literature review. Thus, the results of the literature review aided in the second part of the research approach—the identification of compre-

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hensive theoretical or functional relationships for estimating the condition of roadway pavements.

The acquisition of pavement management system (PMS) data files was the third step of the research approach. The data files used came from three distinct PMSs, each with a different pavement condition rating system. These files were readily available without cost. The three methods were systems used by San Francisco (*3*), the Metropolitan Transportation Commission (MTC) (*4*), and CHEC Consultants (*5*). From the Bay Area, the city of San Francisco supplied a copy of its data file and MTC provided a copy of the data file for Alameda County. CHEC Consultants provided copies of data files developed for several clients, namely, the cities of Fairfield and Puyallup and Jackson County. Fairfield, California, is approximately 50 mi northeast of San Francisco, and Puyallup, Washington, is approximately 30 mi south of Seattle. Other jurisdictions were the counties of Jackson in southern Oregon and Alameda in the San Francisco Bay Area. Subgrade soil information was added manually to the San Francisco and Puyallup data files.

The next aspect of the research approach was conducting multivariate analysis on each of the data files. This analysis revealed the most favorable model for each data base. The effort also addressed possible multicollinearity among some independent variables such as functional class and pavement surface thickness. This investigation included the weighted-least-square analysis to determine the validity of the basic regression assumptions—such as, the error term has a uniform variance. Finally, the results were interpreted in light of the objectives and evaluated against the stated hypotheses.

LITERATURE REVIEW

The past three decades have seen substantial effort applied to the evolution of pavement condition analyses, especially during the past 15 years. Generally, these efforts were accomplished with the use of multivariate analysis (modeling). Materials include national, state, local, and private sources from U.S. and Canadian literature. Most of the investigations were developed for or by state departments of transportation. Two factors affecting the condition of pavements consistently appeared in the literature; namely, age of roadway and frequency of trucks (*4,6–8*). Other factors that appeared often included pavement deflection readings and environment, that is, rainfall and soil type (*7,8*). Functional classification (*4,8*) was also found to affect pavement condition. In contrast to what has been developed for states, very little pavement condition analysis work has been developed for local governments. The literature did not report on the development of any pavement condition prediction model for a particular local jurisdiction, nor did it reveal any attempt to model jurisdictional uniqueness. In addition, the present models for local governments did not use factors to assess soil type and presence of drainage facilities. Finally, the literature did not model the presence of urban transit bus service.

MAJOR HYPOTHESES

The first step of the research approach was the development of the null hypotheses presented and discussed in the following. They were designed to use multivariate analyses for the attainment of the research objectives. The alternative hypotheses were the antitheses of these statements.

1. *The old or previous condition score does not affect a new condition score.* Old condition score will be collected frequently in years to come; consequently, it will be convenient if indeed it is a statistically significant factor affecting a new condition score.
2. *The type of subgrade soil does not influence the pavement condition score.* This hypothesis will help verify whether the subgrade is a statistically significant factor affecting pavement condition.
3. *The drainage facilities do not affect the pavement condition.* Pavement condition modeling efforts have not attempted to model whether the existence of drainage facilities affects pavement conditions. For this project, curb and gutter is the parameter used as a surrogate for the drainage issue.
4. *Traffic does not contribute to pavement condition scores.* This hypothesis will help evaluate whether ADT is a statistically significant parameter. The particular interest in ADT is that it may be an adequate surrogate for axle loadings and frequency of loads usually expressed in equivalent single (18-kip) axle loads (ESALs). If ADT is a significant predictor of condition, local governments may not need to conduct truck counts to include this aspect of causality in pavement condition analysis. Currently, local jurisdictions do not conduct truck counts regularly, and it is not likely they will until these counts can be done much more cheaply.
5. *The existence of bus service does not influence the pavement condition score.* This hypothesis will help verify whether bus service is a statistically significant factor affecting pavement condition.
6. *Data bases containing pertinent information that will estimate the pavement condition score do not exist.*
7. *A generic model that identifies factors of pavement condition for local governments cannot be derived*; that is, a model developed for one jurisdiction cannot be applied to another.

MODEL FORMS

Considering the results of the literature review and the hypotheses being tested, one may theorize that the present condition of a pavement section is dependent on several factors. These factors include

SCRN—New (or estimated) pavement condition score,
SCRO—Old (or previous) pavement condition score,
AGE—Age of pavement structure since last major work,
ADT—Average daily traffic,
TI—Traffic index (related to ESALs),
DEFL—Deflection reading,
FC—Functional classification (arterial; FCA, collector; FCB, local),
SC—Soil classification (clay or nonclay),
CG—Roadway drainage classification (presence or absence of curb and gutter),
ST—Surface thickness of asphalt concrete layer,
BS—Bus service (presence or absence of bus service), and
JUR—Jurisdiction.

The functional form may be expressed with both linear and nonlinear formulations as follows, with β0 being the intercept and the other βs being the coefficients of the independent variables.

$$\begin{aligned} SCRN = {} & \beta 0 + \beta 1(SCRO) + \beta 2(AGE) + \beta 3(ADT) \\ & + \beta 4(TI) + \beta 5(FC) + \beta 6(SC) + \beta 7(CG) \\ & + \beta 8(BS) + \beta 9(JUR) + \beta 10(DEFL) + \text{error} \qquad (1) \end{aligned}$$

$$\begin{aligned} SCRN = {} & \beta 0(SCRO)^{\beta 1}(AGE)^{\beta 2}(ADT)^{\beta 3}(TI)^{\beta 4} \\ & (FC)^{\beta 5}(SC)^{\beta 6}(CG)^{\beta 7}(BS)^{\beta 8}(JUR)^{\beta 9} \\ & (DEFL)^{\beta 10}[\exp^{(\text{error})}] \qquad (2) \end{aligned}$$

These model formulations were calibrated with each of the data bases available to this research effort.

DATA FILES

An important aspect of any model development process is the data base. Without reliable data, the likelihood of developing a valid model is very small. For local governments, the major issue is a sufficient but not excessive data base, because these entities often have limited financial and technical resources. Likewise, the formulation of an accurate pavement condition prediction model is dependent, in part, on pavement management system data bases containing sufficient information.

A pavement management system contains a data base that is organized into records or observations. Records typically include, as a general data file, the following:

1. Record number of street segment,
2. Street name and limits of segment,
3. Functional classification,
4. Old pavement condition rating score and year, and
5. New pavement condition rating score and year.

A pavement condition rating score system typically incorporates such items as extent and severity of cracking (alligator, longitudinal, and transverse), rutting and raveling, and ride quality (comfort). One important point is that pavement condition ratings are subjective in nature and some inconsistency among survey personnel should not be surprising. The variables available for each data file are given in Table 1.

A common feature in the data bases is the use of a dummy variable for variables such as FC. Dummy variables take the value of either 0 or 1 for linear models. For nonlinear formulations the value will be 1 or 2.72 so that, after logarithmic transformation, the value becomes 0 or 1, respectively. The data analyses were conducted on a personal computer using dBase or Rbase software for data management and editing. In addition, SAS (Statistical Analysis System) was used for statistical modeling (*9*).

Before the data files could be used for analysis, several types of editing were imperative. First, data files were subject to human coding errors. For example, the San Francisco file had a few duplicate records, and a Fairfield record contained a negative pavement surface thickness. The second type of editing was the modification of the data bases so that they could be used in the modeling analyses. Alpha characters were used for functional classification, and they needed to be converted to numerical form. Another type of editing was the elimination of potential bias in the data. For example, the San Francisco data street segments with cable car service were omitted because those segments have rails set in portland cement concrete. Consequently, those pavement surfaces are not typical. Market Street was also omitted, because the surface street is supported by Bay Area Rapid Transit (BART) tunnel structures. The last form of editing was to delete street segments that were improved between the two time domains, the old and new score. The data files used for linear multivariate analysis possess the descriptive statistics in Table 2.

These files were modified to enable nonlinear analyses to be conducted. So that logarithm transformations could be accomplished, those observations with 0 or negative scores were removed from the data files. Although this data censoring may lead to biased coefficient estimates, the use of more elaborate statistical methods is warranted, because the main objective here is to compare multiplicative models against linear models. For the dummy variables, the values of 0 were changed to 1 and 1 was modified to 2.72 so that the transformed values would be 0 and 1, respectively.

To improve the modeling results, an outlier removal procedure was used on the CHEC data. As a result, records were removed when the new condition score was greater than the mean at a 1 percent level of significance. This removed extreme values that were believed to be coding errors.

TABLE 1 SUMMARY OF VARIABLES IN DATA BASES

VARIABLE	SAN FRANCISCO	FAIRFIELD	PUYALLUP	JACKSON COUNTY	ALAMEDA COUNTY
SCRN	1986	1988	1988	1988	1988
SCRO	1983	1986	1986	1986	1986
AGE	YES	NO	NO	NO	YES
ADT	YES	NO	NO	NO	*
FC	YES	YES	YES	YES	YES
CG	NO	YES	YES	YES	NO
ST	NO	YES	YES	YES	NO
SC	YES	NO	YES	YES	NO
BS	YES	NO	NO	NO	NO

* Only a few records contained ADT.

TABLE 2 DESCRIPTIVE STATISTICS OF DATA FILES

Number of Observations	Variable	Minimum Value	Maximum Value	Mean	Standard Deviation
		San Francisco			
1210	SCR86	-31	100	70.7	27.5
	SCR83	-11	100	75.5	22.2
	ADT87	100	56500	4320	4790
	AGE	4.0	86	19.7	11.0
	FC	0.0	1.0	0.57	0.50
	SC	0.0	1.0	0.54	0.50
	BS	0.0	1.0	0.43	0.50
		Fairfield			
1834	SCR86	0.0	280	13.2	33.5
	SCR88	0.0	390	35.5	49.1
	CG	0.0	1.0	0.93	0.23
	ST	0.17	0.79	0.24	0.08
	FCA	0.0	1.0	0.28	0.45
	FCB	0.0	1.0	0.56	0.50
		Jackson County			
965	SCR86	0.0	360	21.7	49.3
	SCR88	0.0	345	39.1	62.2
	CG	0.0	1.0	0.07	0.25
	FCA	0.0	1.0	0.49	0.49
	FCB	0.0	1.0	0.50	0.50
	ST	0.04	0.71	0.20	0.14
		Puyallup			
898	SCR86	0.0	605	63.7	90.1
	SCR88	0.0	605	57.5	89.5
	CG	0.0	1.0	0.56	0.50
	FCA	0.0	1.0	0.20	0.40
	FCB	0.0	1.0	0.68	0.47
	SC	0.0	1.0	0.60	0.58
	ST	0.06	0.50	0.19	0.09
		Alameda County			
246	SCR86	20	100	77.6	16.7
	SCR88	7.0	100	69.7	22.6
	FC	0.0	1.0	0.93	0.25
	AGE	0.0	57.0	27.4	10.5

DATA ANALYSES

Overview of Data Analyses

The models described above are referred to as "time-lag models" because the old pavement condition score was included to help estimate the new condition score. Non-time-lag analyses were also conducted on the San Francisco and Alameda data bases because one of their variables was the year in which major work, overlay or reconstruction, was most recently accomplished. The reader will recall that there is an insufficient number of pavement condition score histories or time domains to perform true time-series analysis. Time-lag analysis is distinguished from time-series analysis by the number of time domains: for time lag there are only two time domains, but time-series analysis requires more than two. It is possible that for time-lag modeling, "serial correlation" exists—that is, the error of one time domain, the old condition score, is correlated with the error of another time domain, the new condition score. If so, the independent variables are correlated with the error and thereby may lead to "inconsistent" estimates. To evaluate whether this occurs, the data are needed for at least three time domains. The available data files did not have three distinct time domains; consequently, the analysis of the time-domain variables was called time-lag analysis.

To select the "best" model, several statistical criteria were considered (*10*). The four criteria were first identified and then discussed. They include (a) the *F*-ratio test value of model, (b) the R^2 value of the model, (c) the *t*-statistic value for the coefficient of each independent variable, and (d) a plot of the residual of the dependent variable versus the estimated value of the dependent variable. It is important to remember when applying these criteria that some models will perform better on some criteria than others. Often, judgment is essential in the selection of the best model. The results of the statistical modeling are given in Table 3. This table contains features and characteristics of a model for each data base. The following discussion will comment on the results of the analyses of the various data files regarding the factors likely to contribute to the estimation of pavement condition scores. The discussion will examine the results of model analyses variable by variable. It will also point out the relative contribution of each factor when the mean values of the independent variables were substituted into the model. For the dummy variables, 1.0 was substituted.

SCRO

In what was identified as time-lag analyses, all of the data files revealed that the old pavement condition score was a useful variable in estimating a new condition score. In fact, it is a highly significant variable statistically. Its contribution to the new score ranged from 75 percent for San Francisco to 8.4 percent for Fairfield. It should also be noted that for the CHEC rating system (Fairfield, Jackson County, and Puyallup), the contribution of the score becomes much greater

TABLE 3 RESULTS OF STATISTICAL MODELING ANALYSIS

INDEPENDENT	JURISDICTION				
VARIABLE	San Francisco	Fairfield	Jackson C	Puyallup	Alameda C
Model Coefficients					
Intercept	1.24	76.5	43.0	-9.10	47.1
ADT (1,000s)	NS				
AGE	-0.19				-0.26
FC	3.55				-13.1
FCA		-10.1	NS	3.18	
FCB		-11.1	-15.4	5.34	
CG		-23.0	-15.8	-1.57	
ST		-83.5	-56.0	15.3	
SC	6.96			1.99	
BS	-2.87				
SCRO	0.89	1.11	0.93	0.96	0.54
Model F-Ratio Test					
Test Significance *	0.0001	0.0001	0.0001	0.0000	0.0001
Degree of freedom	5,1204	5,1725	4,892	6,744	3,242
R^2 Value(Model)	0.57	0.31	0.38	0.99	0.21
t-statistic Value					
Intercept	0.54	12.7	10.3	-6.03	5.08
ADT	NS				
AGE	-1.98				-2.03
FC	3.37				-2.58
FCA		-3.16	NS	3.16	
FCB		-3.53	-4.77	5.14	
CG		-5.52	-2.69	-2.62	
ST		-6.40	-4.44	3.51	
SC	6.58			4.34	
BS	-2.73				
SCRO	36.6	24.2	15.3	251	6.80
Sample Size	1210	1731	897	751	246

NS - Not Significant

* Probability the F-ratio statistic exceeds the F-ratio test threshold valve.

when the pavement conditions are poorer. This is in contrast to the San Francisco and MTC rating systems, in which the contribution diminishes when the conditions decline because the scores become smaller.

AGE

The only data files containing the AGE variable were San Francisco and Alameda County. Those data files contained a field for the year of the most recent major work or reconstruction. The contribution of the mean value of age was 7.1 percent for San Francisco to 9.6 percent for Alameda County.

ADT

The San Francisco data file was the only one containing ADT. It should be noted here that the accuracy of the ADT is limited because it was estimated for many segments, especially for nonarterials. This variable was statistically significant in estimating the newer condition score for the linear model only, not for the time-lag or nonlinear models. Consequently, one can conclude that ADT is a marginal variable that may, but not necessarily, significantly affect pavement condition scores. This observation is reasonable considering an important pavement design parameter: truck traffic. For pavement design, ADT—in particular, truck traffic—is converted to ESALs and then converted to a traffic index (in California) used to determine pavement thickness. Local governments normally do not conduct traffic classification counts. Consequently, one can readily see that a relationship exists between ADT and traffic index, but it depends on the amount of truck traffic. This amount will probably vary with functional classification. For example, the ADT on an arterial will most likely have a higher percentage of truck traffic than on either a collector or a local street. Consequently, functional classification may be a better variable than ADT. The R^2 value for the model would probably increase substantially if either the truck volumes or ESALs were one of the independent variables.

TI

It was not possible to evaluate traffic index with the available data bases because the traffic index values for the data files available for this modeling effort were based on functional classification. A true value of traffic index would be computed from the average daily truck traffic and the ESALs associated with that data.

FC

As implied in previous sections, functional classification can be used as a surrogate, dummy variable for traffic index and

ADT. This is because arterials serve the highest traffic volumes and the heaviest vehicles. Consequently, arterials may deteriorate at a higher rate than nonarterial streets. From the analysis on the San Francisco data, this was so. In all formulations of the San Francisco models, functional classification was highly significant. For the data files from Fairfield, Jackson County, Alameda County, and Puyallup, the functional classification differentiated arterial, collector, and local streets. It was significant in all of the models. The contribution of functional classification varies from a low of 4.4 percent for San Francisco to a high of 21 percent for Jackson County.

ST

Another variable in several data files for Fairfield, Jackson County, and Puyallup is that of the pavement surface (asphalt concrete surface) thickness. From pavement design principles this surface thickness is, of course, a function of the traffic index as well as other factors. Subsequently, the surface thickness is expected to be a surrogate for traffic index. Similarly, because the traffic index is determined by functional classification, the surface thickness should be a surrogate for it as well. In all three data bases that contain surface thickness it is a statistically significant predictor of a new condition score. Its contribution ranged from 3.7 to 18.1 percent.

SC

From basic geotechnical engineering and pavement design publications, the strength of the subgrade is important to the structural quality of a pavement system. Consequently, it was not surprising when the models of data files containing soil classification, San Francisco and Puyallup, contained this variable as a significant predictor of pavement condition. For San Francisco soil classification contributed 8.4 percent, for Puyallup, only 2.5 percent.

CG

The variable in several data bases evaluating drainage facilities was the presence or absence of curb and gutter. For Fairfield, on a sensitive clay, the curb-and-gutter variable was statistically significant in estimating the new pavement condition score and contributed 16.7 percent toward it. Where better soils existed in Jackson County and Puyallup, curb and gutter contributed 1.2 and 2.0 percent, respectively. This is not surprising, because the subgrade in Fairfield is a more sensitive clay than in the other jurisdictions. Puyallup does have some clay pockets, but overall its subgrade is better than Fairfield's.

BS

The San Francisco data base contained a dummy variable for the presence or absence of bus service. It was a significant predictor of pavement condition and contributed approximately 4 percent to the condition score.

JUR

To model characteristics that are unique to individual local governments (such as the level of maintenance effort, individual maintenance practices, and local materials), three data files were paired. Files from Fairfield, Jackson County, and Puyallup were paired with each other and a dummy variable, JUR, was inserted. The results indicated that JUR is statistically significant in estimating the new pavement condition score. One concern is whether differences in climate may override the uniqueness. Normal rainfall is nearly 20 in. for both Fairfield and Jackson County and more than 40 in. for Puyallup, so the best comparison is of the first two.

FINDINGS AND CONCLUSIONS

The discussion about the findings of the models developed from the multivariate analyses of the data bases available to this project has been divided into four areas. The first portion summarizes the factors affecting the estimation of the pavement condition. Comments about the results of the hypotheses testing will be discussed second. Next, the attainment of the research objectives will be presented, followed by the major limitations that were discovered.

Significant Factors

As discovered from the analyses described in previous sections, several factors contribute to the estimation of pavement condition. Table 4 presents the significant factors across the data bases of the six jurisdictions. Most of the factors in the data bases that were expected to be statistically significant were indeed.

According to the *t*-statistic, the most important variable in the time-lag formulations was the old condition score (SCRO). In the absence of SCRO, AGE became the most important factor; however, it was the least important for the time-lag models. For the data bases that had it (San Francisco and Puyallup), the SC dummy variable was the second most important factor in estimating a new condition score.

The order of importance for the rest of the factors was not so clear. As discussed, FC and ST are correlated. Between these two, ST may be the better estimator, probably because it can take on several values rather than merely 0 or 1 for a dummy variable. By removing the FC variable, the coefficient of ST changed and the coefficients of the other variables did not change dramatically.

TABLE 4 SUMMARY OF SIGNIFICANT FACTORS

DATA BASES	SCRO	AGE	ADT	FC	ST	SC	CG	BS
San Francisco								
Available	Yes	Yes	Yes	Yes	No	Yes	No	Yes
Significant	Yes	Yes	No	Yes	-	Yes	-	Yes
Fairfield								
Available	Yes	No	No	Yes	Yes	No	Yes	No
Significant	Yes	-	-	Yes	Yes	-	Yes	-
Jackson County								
Available	Yes	No	No	Yes	Yes	No	Yes	No
Significant	Yes	-	-	Yes	Yes	-	Yes	-
Puyallup								
Available	Yes	No	No	Yes	Yes	Yes	Yes	No
Significant	Yes	-	-	Yes	Yes	Yes	Yes	-
Alameda County								
Available	Yes	Yes	No	Yes	No	No	No	No
Significant	Yes	Yes	-	Yes	-	-	-	-

Note: The level of significance is 5%.

For a community having a sensitive clay for a subgrade, Fairfield, the variable CG was statistically significant. The notion behind this idea is that the presence of curb-and-gutter systems would prevent moisture from affecting sensitive soils in the roadway subgrades. Another significant factor was the existence of urban transit bus service (BS).

An important discovery unique to this effort was the statistical significance of the dummy variable JUR. Consequently, the set of factors describing the pavement condition of one jurisdiction should not be directly transferred to another.

Hypotheses-Testing Results

The first null hypothesis—that SCRO does not affect SCRN—was rejected because every model included the old score whenever it was contained in data bases. Its alternative hypothesis was accepted.

Next to be rejected was the null hypothesis that the type of subgrade soil does not influence the pavement condition rating score. In fact, the alternative hypothesis (that the subgrade type does affect the score) was accepted. This was because the soil classification was included in both of the pavement condition prediction models where it was an issue, that is, San Francisco and Puyallup.

The third null hypothesis was that drainage through the dummy variable CG does not contribute to the pavement condition score. It was rejected because curb and gutter was a statistically significant variable in two out of the three data files that contained it.

The null hypothesis that ADT did not contribute to pavement rating scores was *not* rejected. The San Francisco data base contained ADT, but it was not a significant estimator of pavement condition. Given the associated limitations, this result is not conclusive. The next hypothesis, about bus service not affecting pavement condition, was rejected; the null hypothesis was accepted, because it did affect the condition.

That sufficient data files to model condition scores did not exist was the sixth null hypothesis, and it needed to be rejected. This rejection was justified because all six data files, with some modification, were sufficient to conduct statistical modeling. Consequently, the alternative hypothesis was accepted. This means that some existing PMSs do contain sufficient data to develop prediction models.

The final null hypothesis was that a generic model for estimating pavement condition scores applicable to several local jurisdictions cannot be derived. This null hypothesis was *not* rejected because the dummy variable JUR was found to be significant.

Attainment of Objectives

The factors affecting the condition of pavements maintained by local governments were identified, thus fulfilling the primary objective of this research effort. These factors are

1. Old (or previous) pavement condition score,
2. Age of pavement structure since last major work,
3. Soil classification (clay or nonclay),
4. Classification of roadway drainage (presence or absence of curb and gutter),
5. Surface thickness,
6. Functional classification,
7. Presence or absence of bus service, and
8. Individual jurisdiction.

Another objective was to investigate the adequacy of data bases maintained by local governments. A major effort was to collect the data required to estimate reasonably the pavement condition score. Even at that, some data used for this research effort were simply not available. Typically, local agencies do not include soil classification in PMS data files. For local jurisdictions having subgrade soils with areas of sensitive clay, the efforts to add soil type to the data base were not excessive. The marginal efforts for additional data should not be a burden, provided a PMS is in use. One missing datum item was truck traffic counts needed to determine the traffic index. Including truck count data will require significant resources. If this information had been available, the index of determination for the best models could probably have been increased substantially. Overall, this objective was met.

Next, one multivariate analysis was performed on the combination of data files of Fairfield and Jackson County with a new dummy variable for jurisdiction included. This factor was significant; consequently, a model developed to estimate the pavement condition for one jurisdiction should not be transferred to another. Instead, a general model identifying those factors can be asserted, but an analysis unique to each jurisdiction is needed to assess which factors are significant. Consequently, the last objective was met.

Major Limitations

There are major limitations associated with this effort. The first deals with the technique of conducting field surveys to determine the condition rating scores. Another limitation relates to the limited number of time domains. The other limitations include classification counts of traffic and types of pavement. ADT was not evaluated conclusively.

The first limitation statement deals with the field collection of the pavement condition rating data. Actions are needed to ensure that the field work will be done in a manner so as to generate condition rating scores that are consistent from one time domain to another. The actions are the use of the same survey personnel for each time domain and the provision of refresher training to the personnel before each survey.

Local data bases with more than two rating surveys (i.e., two time domains) were not available for this project. There was no assurance that serial correlation does not exist in the time-lag analyses. Several data files with more than two time domains are now probably available, which will enable true time-series analysis to be conducted. It is likely that adding another time domain (rating survey) will make nonlinear models more attractive.

Another limitation existed in the data bases. The ADT data were not complete. Also, there were no traffic classification count data in any of the data files; therefore, ESALs could not be modeled. If these data were available, a variable for truck traffic would most likely be included. The addition of such a variable would probably increase the R^2 of the models. Finally, there were types of pavement maintained by local jurisdictions that were not analyzed, such as facilities with evolved pavements or with asphalt concrete mixes placed directly on the subgrade.

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Adaptive Filter Forecasting System for Pavement Roughness

JIAN LU, CARL BERTRAND, W. R. HUDSON, AND B. F. MCCULLOUGH

Forecasting pavement roughness conditions can facilitate decision making within a pavement management system at project and network levels. Because pavement roughness change over time is caused by some important conditions and certain stochastic factors, a parameter and dynamic forecasting model is more appropriate for forecasting roughness with respect to linear, static, and nonparameter forecasting models. Thus, an adaptive filter forecasting system is presented that forecasts pavement roughness conditions by means of an adaptive filter using roughness history. The concept of an adaptive filter forecasting system is introduced, along with its mathematical derivation and least-mean-square algorithm. In testing the system's validity, a given mathematical function is used to simulate changing pavement roughness conditions. In addition, a practical application of the adaptive filter forecasting system is presented. The roughness index used is the root-mean-square vertical acceleration of a response-type road-roughness measuring system. Finally, choice of the adaptive filter structure and its stability, based on roughness data collected from Austin Test Sections, are discussed. The structure of system should be decided before each application by experimental results with certain criteria. This is a major limitation of the system.

Measurement of pavement roughness is an important exercise within a pavement management system (PMS) at project and network levels, because it relates to pavement evaluation, maintenance, and rehabilitation (*1,2*). In addition, pavement roughness measurements have been used in predicting vehicle operating cost, predicting road performance, evaluating road safety, and evaluating passenger degree of comfort (*3–8*; Darlington, unpublished data). Since the AASHO Road Test, much roughness research has been conducted, including studies on measuring techniques, index development, evaluation, specification, and prediction.

However, pavement roughness measurements can reflect only existing states. Unless adequate forecasting models are used to predict future roughness conditions, existing roughness cannot provide reliable information on which to base future planning, maintenance, rehabilitation, and other PMS activities.

Two concepts concerning roughness prediction must be distinguished. The first, which has been the subject of much research (*5,9–11*), can be described by the following equation:

$$R_k = F(k, D_k, M_k, T_k, E_k, \text{etc.}) \qquad (1)$$

where

R_k = roughness at time k,
D_k = pavement structure at time k,
M_k = pavement materials at time k,
T_k = traffic condition at time k, and
E_k = environment at time k.

Roughness at time k is estimated using these existing conditions but disregarding past information.

The second concept can be described by another equation:

$$R_k = F(R_{k-1}, R_{k-2}, \ldots, R_{k-N}) \qquad (2)$$

Roughness at time k, R_k, is forecast using historical roughness records at time $k - 1$, $k - 2$, . . . , $k - N$. This is a time-series modeling problem. It appears that Equation 2 does not consider conditions affecting roughness except past roughness. Conditions of pavement structure, materials, traffic, and environment are time-variable; certain changing trends over time are reflected in the past roughness data sequence. Conditions thus are forecast by understanding the changing processes of past conditions of pavement structure, materials, traffic, environment, and such. In this study, this concept is called time-series forecasting of roughness. It provides better information for decision making in planning, maintenance, and rehabilitation because the future roughness state has been forecast.

Traditionally, linear regression and extrapolation models have been used for forecasting (*12–15*). These are nonparameter estimation models and are usually considered static estimators. It is understood that the changing process of pavement roughness consists of certain trend caused by some conditions and unpredictable stochastic factors. These stochastic factors make a linear static estimator inadequate for forecasting. Linear regression and extrapolation models also have limitations in forecasting pavement roughness.

In the past two decades, several important parameter forecasting models and probability-based models have been applied to transportation areas (*15–17*). Mathematically, the parameter forecasting models most often used are Kalman filtering (*18*), time-series prediction (*19*), spectral analysis (*20*), and adaptive forecasting (*21*).

Recent studies have used an adaptive filter model to forecast roughness. This can be considered a dynamic parameter estimation model—that is, the pavement roughness condition forecast at time step k is a function of past conditions at time step $k - 1$, $k - 2$, . . . , $k - M$ where $M < k$, and M and k are positive integers and a set of parameters estimated by the adaptive filter forecasting system.

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Mathematically, the objective function of this system, minimizing the resulting mean-square error, might be similar to that of the Kalman filtering and time-series prediction models. However, it uses a simplified least-mean-square (LMS) algorithm to search for optimal filter weights or states. This difference means that dynamic response of the system could be better, needing less data storage space than the earlier prediction models. Intuitively speaking, an adaptive filter forecasting system is viewed as one whose structure is adjustable in such a way that its performance improves through contact with its environment.

This paper focuses on a time-series forecasting method for pavement roughness using an adaptive filter forecasting system. The basic concept of the system is introduced, and then its mathematical derivation is described. Results of experiments based on simulation and real roughness data, which is root-mean-square vertical acceleration (RMSVA) (*22*) collected by the Automatic Road Analyzer (ARAN) (*23*), are presented and discussed.

BASIC PRINCIPLES OF ADAPTIVE FORECASTING SYSTEM

Figures 1 and 2 show an adaptive forecasting system and its processors, respectively. In these figures, Z^{-1} is an one-step delay factor, and Z^{-S} is an s-step delay factor (where s is a positive integer). Mathematically, $q(k)Z^{-1} = q(k - 1)$, and $q(k)Z^{-S} = q(k - s)$. As can be seen from Figure 1, the system's core is the adaptive processors, in which all of the parameters (weights) at step k are adjustable. The error of forecast $e(k)$ controls adjustment of the system. From Figures 1 and 2, the following equation can be derived:

$$\hat{q}(k) = \sum_{j=0}^{N} W_{jk} q(k - s - j) \qquad (k = s + 1, s + 2, \ldots) \qquad (3)$$

Equation 3 indicates that $\hat{q}(k)$ is the linear weighted combination of $q(k - s)$, $q(k - s - 1)$, . . . , $q(k - s - N)$. The weights are W_{0k}, W_{1k}, . . . , W_{Nk}, and the index k denotes the time step. If $\hat{q}(k)$ is used to forecast $q(k)$, then the error of forecast at step k is

$$e(k) = q(k) - \hat{q}(k) \qquad (k = s + 1, s + 2, \ldots) \qquad (4)$$

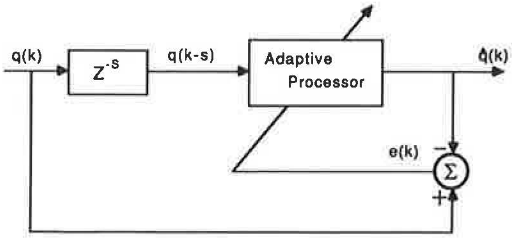

FIGURE 1 Adaptive forecasting system.

The purpose of using an adaptive processors is to adjust the weights at each step k so that the mean square error $E[e^2(k)]$ is minimized. The vectors $\mathbf{W}_k$ and $\mathbf{Q}_{k-s}$ are defined as follows:

$$\mathbf{W}_k = [W_{0k}, W_{1k}, \ldots, W_{Nk}]^T$$

$$\mathbf{Q}_{k-s} = [q(k - s), q(k - s - 1), \ldots, q(k - s - N)]^T$$

With these definitions, Equation 3 can be expressed using vector notation:

$$\hat{q}(k) = \mathbf{Q}_{k-s}^T \mathbf{W}_k = \mathbf{W}_k^T \mathbf{Q}_{k-s} \qquad (5)$$

Now that operation of the adaptive processor has been described, one can consider how the adaptive processor adapts—that is, how the vector $\mathbf{W}_k$ is adjusted as the time-step index k changes.

From Equations 4 and 5, Equation 6 can be derived:

$$e(k) = q(k) - \mathbf{W}_k^T \mathbf{Q}_{k-s} = q(k) - \mathbf{Q}_{k-s}^T \mathbf{W}_k \qquad (6)$$

By squaring Equation 6, the instantaneous squared error can be obtained.

$$\begin{aligned} e^2(k) &= [q(k) - \mathbf{W}_k^T \mathbf{Q}_{k-s}][q(k) - \mathbf{Q}_{k-s}^T \mathbf{W}_k] \\ &= q^2(k) + \mathbf{W}_k^T \mathbf{Q}_{k-s} \mathbf{Q}_{k-s}^T \mathbf{W}_k - 2q(k)\mathbf{Q}_{k-s}^T \mathbf{W}_k \end{aligned} \qquad (7)$$

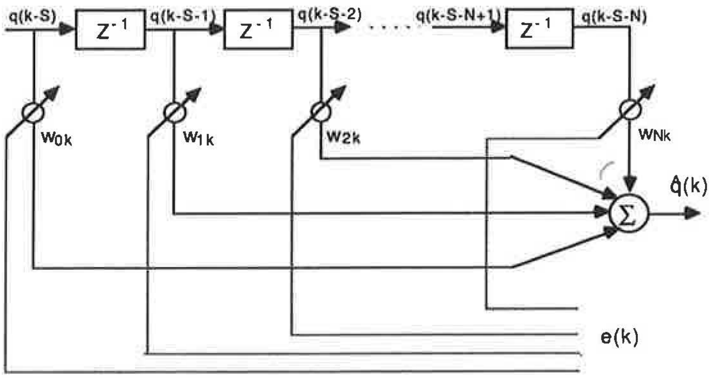

FIGURE 2 Adaptive processors [W_{ik} is adjusted by $e(k)$, $i = 0$ to N].

To find the expected value of Equation 7 over k, it is assumed that $e(k)$ and $q(k)$ are statistically stationary. This assumption can usually be approximately satisfied for the particular pavement roughness conditions. Then the expectation of $e^2(k)$ is

$$E[e^2(k)] = E[q^2(k)] + \mathbf{W}_k^T E[\mathbf{Q}_{k-s}\mathbf{Q}_{k-s}^T]\mathbf{W}_k - 2E[q(k)\mathbf{Q}_{k-s}^T]\mathbf{W}_k \tag{8}$$

Let **R** be defined as the square matrix

$$\mathbf{R} = \mathrm{E}[\mathbf{Q}_{k-s}\mathbf{Q}_{k-s}^T] \tag{9}$$

Thus **R** is the correlation matrix of $q(k - s)$ with dimension $N \times N$. Let **P** be defined as the column vector

$$\begin{aligned}\mathbf{P} &= \mathrm{E}[q(k)\mathbf{Q}_{k-s}^T] \\ &= E[q(k)q(k - s), q(k)q(k - s - 1), \ldots, \\ &\quad q(k)q)k - s - N)]^T\end{aligned} \tag{10}$$

This vector is the set of autocorrelation of $q(k)$. **R** and **P** thus are the second-order statistics of the random variable $q(k - s)$ at step k. By the definitions of **R** and **P**, Equation 6 can be expressed as

$$E[e^2(k)] = E[q^2(k)] + \mathbf{W}_k^T\mathbf{R}\mathbf{W}_k - 2\mathbf{P}^T\mathbf{W}_k \tag{11}$$

According to the assumption that $q(k)$ is statistically stationary, **R** and **P** are a constant matrix and vector, respectively. In this case, $E[e^2(k)]$ is a quadratic function of the weight vector $\mathbf{W}_k$. If the adaptive processor has the ability of "self-study" to seek the minimum $E[e^2(k)]$ by adjusting $\mathbf{W}_k$, and if $E[e^2(k)]$ tends to be minimal when $\mathbf{W}_k$ tends to be optimal solution $\mathbf{W}_k^*$, then the forecast of the processors will be optimal. The question is how to find the optimal solution of $\mathbf{W}_k$ so that $E[e^2(k)]$ is minimized at each step k. This can be solved by the gradient method. The gradient of the mean square error $E[e^2(k)]$ is designated ∇_k and can be expressed by

$$\nabla_k = 2\mathbf{R}\mathbf{W}_k - 2\mathbf{P}$$

To obtain the optimal solution $\mathbf{W}_k^*$ so that $E[e^2(k)]$ is minimized, it is necessary to let

$$\nabla_k = 0 = 2\mathbf{R}\mathbf{W}_k^* - 2\mathbf{P}$$

or

$$\mathbf{W}_k^* = \mathbf{R}^{-1}\mathbf{P} \tag{12}$$

Equation 12 is the optimal solution of $\mathbf{W}_k$. By substituting Equation 12 into Equation 11, and noting that the correlation matrix is symmetric, then

$$\begin{aligned}E[e^2(k)]_{\min} &= E[q^2(k)] + [\mathbf{R}^{-1}\mathbf{P}]^T\mathbf{R}\mathbf{R}^{-1}\mathbf{P} - 2\mathbf{P}^T\mathbf{R}^{-1}\mathbf{P} \\ &= E[q^2(k)] + \mathbf{P}^T\mathbf{R}^{-1}\mathbf{P} - 2\mathbf{P}^T\mathbf{R}^{-1}\mathbf{P} \\ &= E[q^2(k)] - \mathbf{P}^T\mathbf{R}^{-1}\mathbf{P}\end{aligned} \tag{13}$$

Although Equation 12 is the optimal solution of $\mathbf{W}_k$, in a practical sense $\mathbf{W}_k^*$ is not estimated by Equation 12. In the next section, the algorithm to estimate $\mathbf{W}_k^*$ is discussed.

LEAST-MEAN-SQUARE ALGORITHM

Recall in Equation 12 that

$$\nabla_k = 2\mathbf{R}\mathbf{W}_k - 2\mathbf{P}$$

or

$$\mathbf{W}_k^* = \mathbf{R}^{-1}\mathbf{P}$$

By combining these equations, Equation 14 is obtained:

$$\mathbf{W}_k^* = \mathbf{W}_k - 0.5\mathbf{R}^{-1}\nabla_k \tag{14}$$

It can be changed into an adaptive algorithm as follows:

$$\mathbf{W}_{k+1} = \mathbf{W}_k - 0.5\mathbf{R}^{-1}\nabla_k \tag{15}$$

If the vector of weight $\mathbf{W}_k$ is adjusted in the direction of the gradient at each step k and a constant μ $(0 < \mu < 1)$ is defined, then Equation 15 can be simplified as follows:

$$\mathbf{W}_{k+1} = \mathbf{W}_k + \mu(-\nabla_k) \tag{16}$$

where μ regulates step size (from k to $k + 1$) and has dimensions of reciprocal signal power.

To develop the LMS algorithm, $e^2(k)$ itself can be taken as an estimate of $E[e^2(k)]$; then the estimate of the gradient ∇_k can be expressed by

$$\hat{\nabla} = \begin{bmatrix} \dfrac{\partial e^2(k)}{\partial W_{0k}} \\ \vdots \\ \dfrac{\partial e^2(k)}{\partial W_{Nk}} \end{bmatrix} = 2e(k) \begin{bmatrix} \dfrac{\partial e(k)}{\partial W_{0k}} \\ \vdots \\ \dfrac{\partial e(k)}{\partial W_{Nk}} \end{bmatrix} = -2e(k)\mathbf{Q}_{k-s} \tag{17}$$

With this simple estimate of the gradient, the LMS algorithm can be specified by Equations 16 and 17:

$$\mathbf{W}_{k+1} = W_{k-\mu}\nabla_k = \mathbf{W}_k + 2\mu e(k)\mathbf{Q}_{k-s} \tag{18}$$

In this research effort, another parameter—$AL1$—was defined:

$$AL1 = \frac{1}{2\mu}$$

where $AL1$ is called an attenuate factor. Thus Equations 5 and 18 constitute the adaptive forecast model. Equation 18 indicates that the LMS algorithm can be implemented in a practical system without squaring, averaging, or differentiation and is elegant in its simplicity and efficiency.

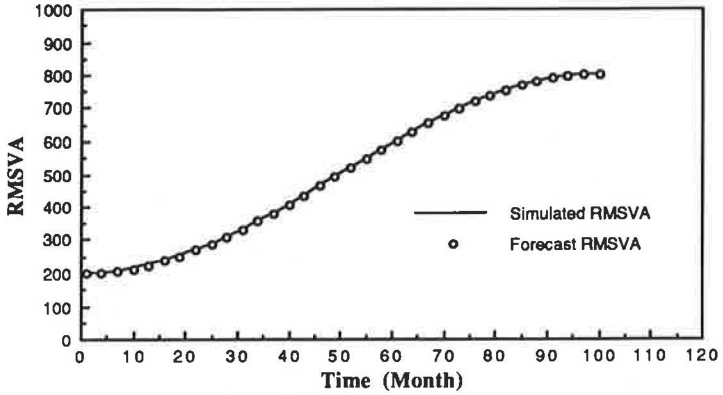

FIGURE 3 Comparison between simulated and forecast RMSVA.

SIMULATION OF ADAPTIVE FILTER FORECASTING SYSTEM

A simulation experiment was conducted to evaluate the performance of the adaptive filter forecasting system. The experiment was conducted by inputing a given mathematical function as a simulation of RMSVA to the system. This was done to prove the applicability of the system for practical purposes. In the experiment, a mathematical function was used to simulate RMSVA as a function of time, or RMSVA(t):

$$\mathrm{RMSVA}(t) = 500 - 300\cos(\pi t/100) \qquad (t\text{: month, } t = 1, 2, \ldots, 100) \qquad (19)$$

The experimental results are shown in Figure 3, with $AL1 = 6.52 \times 10^5$, $N = 3$, and $S = 1$. It is clear from the graph that the forecast RMSVA follows the true (simulated) RMSVA. For this kind of deterministic RMSVA, the system can precisely predict future characteristics of RMSVA by understanding the past process of RMSVA. This ability could be due to the continuously differentiable nature of the sine function input.

APPLICATIONS OF SYSTEM

As stated in the introduction, roughness conditions are forecast by using past roughness data, RMSVA. The amount of past data that must be stored in the forecasting system depends on the order of the adaptive filter. To forecast future roughness, a certain quantity of initial roughness data should be available. Then, after the forecasting system is in use, initial data will be continuously updated by measured data.

Field Data Collection and Preparation

During the study, the adaptive filter forecasting system was applied to forecasts of RMSVA of Austin Test Sections (ATS). Roughness conditions have been monitored by a K. J. Law profilometer at 20 mph since July 1982. The original index is serviceability index (SI) collected every 3 months. However, because the forecasting system is designed for forecasting RMSVA with past RMSVA data measured by the ARAN unit, original data had to be changed to corresponding RMSVA data by a correlation model between the Law profilometer and the ARAN unit. The correlation model has the following form (*23*):

$$\mathrm{SI\ (profilometer)} = 5.297 - 4.742\ 10^{-3}\ \mathrm{RMSVA\ (ARAN)}$$

or

$$\mathrm{RMSVA\,(ARAN)} = 1117 - 210.9\,\mathrm{SI\,(profilometer)} \qquad (20)$$

General experience indicates that the measured data include certain systematic and operational errors. A good data processing technique to reduce the errors is data smoothing. In this study, a three-order smoothing filter was used to smooth the measured data sequence.

Results of Forecasting Roughness Data, RMSVA

Although past roughness had been measured, it was impossible to forecast pavement conditions precisely. This result is different from the simulation experiment. The adaptive filter forecasting system can figure statistical characteristics of pavement roughness conditions using the adaptive processor and past roughness data, RMSVA, for optimal forecasting of roughness conditions; that is, statistically the adaptive filter forecasting system's performance is optimal.

Figures 4 and 5 show results of forecasting RMSVA at Austin Test Sections ATS36 and ATS40, with given adaptive

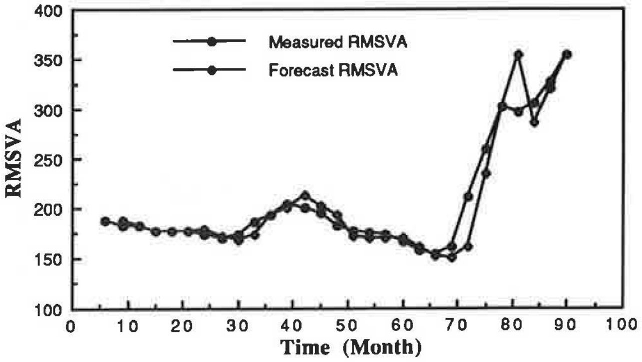

FIGURE 4 Comparison between measured and forecast RMSVA of ATS36.

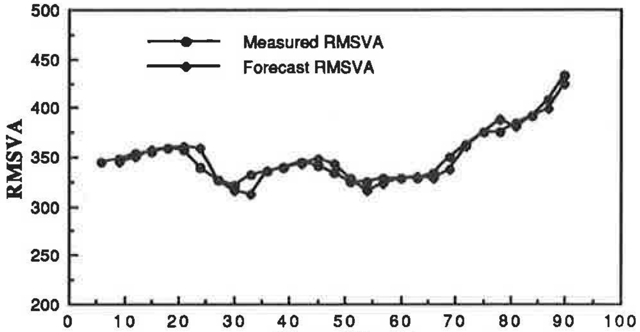

FIGURE 5 Comparison between measured and forecast RMSVA of ATS40.

filter structures ($N = 3$, $AL1 = 5.42 \times 10^5$, $S = 1$) and ($N = 3$, $AL1 = 1.82 \times 10^5$, $S = 1$), respectively. Averaged absolute forecast errors are 9.777 for ATS36 and 5.359 for ATS40.

Figures 6 and 7 show results of forecasting RMSVA of ATS07 and ATS38 by the adaptive filter forecasting system with the structures ($N = 3$, $AL1 = 1.02 \times 10^5$, $S = 1$) and ($N = 3$, $AL1 = 1.118 \times 10^6$, $S = 1$), respectively. These graphs show that some maintenance or rehabilitation activities, such as overlay, took place during the monitoring period, so that the roughness level RMSVA dropped after that work. However, it should be mentioned that the historical roughness data before major maintenance or rehabilitation should not be used to forecast subsequent roughness. Averaged absolute forecast errors for ATS07 and ATS38 are 3.949 and 11.082, respectively.

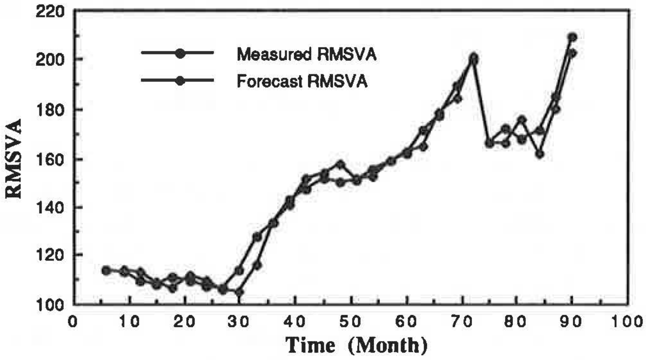

FIGURE 6 Comparison between measured and forecast RMSVA of ATS07.

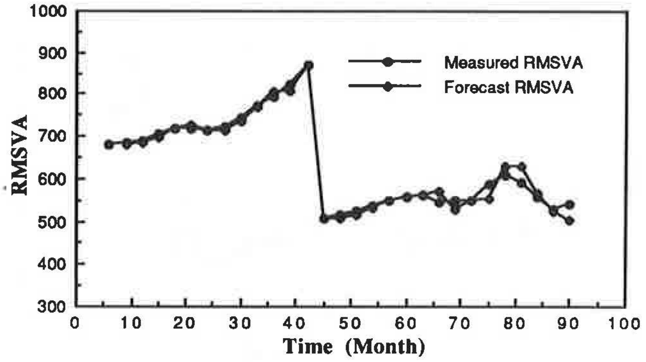

FIGURE 7 Comparison between measured and forecast RMSVA of ATS38.

ADAPTIVE FILTER STRUCTURE CHOICE AND STABILITY

For given pavement roughness conditions, the forecast is affected mainly by the adaptive filter structure (N, $AL1$). In this study, performance was associated with forecast errors, stability, and such. Adequate choice of the order N and attenuate factor $AL1$ of the adaptive filter can result in a relatively accurate forecast and good stability. For a given pavement section, tests thus should be conducted to choose the optimal pair of N and $AL1$ by minimizing the forecast errors, and N and $AL1$ must be updated further when data on the new roughness condition RMSVA are collected. Table 1 gives optimal pairs of N and $AL1$ for ATS40. The resulting optimal N and $AL1$ are based on roughness data RMSVA collected since July 1982. The index E is the averaged absolute forecast error. Any other choice of N and $AL1$ will result in larger E.

It can be understood that for a new pavement without any existing roughness data, the optimal pairs of N and $AL1$ cannot be decided and certain initial readings are needed for forecasting. However, roughness readings from other pavement with closely similar conditions can be used to predict roughness of this new pavement. After several readings have been obtained, the forecasting system will gradually get into optimal state by continuously updating its structure.

Like other dynamic systems, the adaptive filter forecasting system also has the problem of stability. A simple definition of stability adopted in this study is that if the averaged absolute forecast error E is always smaller than a given number or critical value, A, the adaptive filter forecasting system is said to be stable; otherwise it is unstable.

Stability of the system depends mainly on $AL1$ and N. In the plane of ($AL1$, N) a zone should exist where the system should be stable, or it would be unstable. Figure 8 shows the

TABLE 1 OPTIMAL PAIRS OF N AND $AL1$ TO MINIMIZE FORECAST ERROR FOR ATS40

N	2	3	4	5	6
AL1	350000	542000	750000	958000	1154000
E	5.3624	5.3594	5.3664	5.3942	5.4185

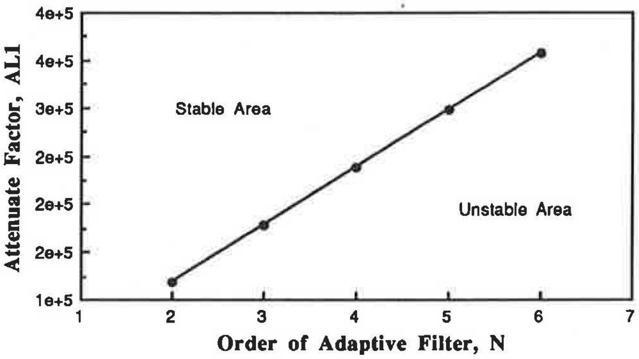

FIGURE 8 Stable and unstable areas in ($AL1$, N) plane.

stable and unstable zones based on RMSVA data collected from ATS40 and $A = 20$. If ($AL1$, N) belongs to the area above the straight line, the system is stable; otherwise it is unstable. In fact, as long as the system is continuously updated, stability will not be a problem because the optimal N and $AL1$ guarantee that the system is in stable zone.

CONCLUSIONS

The adaptive filter forecasting system can be used as a dynamic time-series predictor of pavement roughness conditions. System performance depends both on roughness conditions and structure of the adaptive filter ($AL1$ and N). In choosing $AL1$ and N, consideration should be given to stability of the system. To forecast roughness conditions on a specific pavement section, an adequate number of tests should be run to obtain optimal $AL1$ and N.

The system, like other forecasting models mentioned in this paper, has some limitations for practical application. One of the most critical problems seems to be the convergence that has been discussed in some works (*15,19*). Although in certain situations, the adaptive filter forecasting system could converge to the optimal states with given model structures (i.e., $AL1$ and N), in others the adaptive prediction system might not converge with the same model structures.

In this study the direct application of the adaptive filter forecasting system is to forecast RMSVA. However, in principle this system can be applied to forecasts of other roughness indices, such as SI, international roughness index, and mean absolute slope.

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Procedure to Develop Index Quantifying Transverse Profile and Rutting of Flexible Pavements

JIAN LU, CARL BERTRAND, AND W. R. HUDSON

The amount of rutting on flexible pavements is an important distress parameter to consider when making judgments about rehabilitation of the riding surface. Because severe rutting is dangerous and uncomfortable to the riding public, millions of dollars are spent each year in the United States on rehabilitation of pavements that show such structural deterioration. Thus, network-level decisions about which pavements to rehabilitate should be based on a quantitative rut index that best uses available dollars while protecting the safety of the driving public. The methodology used to develop such a quantifiable rut index for Texas is presented. The Texas Department of Transportation has been collecting rut information by means of survey teams that manually read and record rut-depth information at selected sites throughout the state. The recent purchase of an Automatic Road Analyzer (ARAN) unit (and its associated rut bar) now allows them to collect rut information under traffic conditions and at normal highway speeds. The Center for Transportation Research was contracted to evaluate the ARAN unit and help implement the study findings. The methodology used in developing a rut index based on data collected by the ARAN unit is presented. The conclusions are based on the ARAN's output, but the methodology and index can be applied to any rut-depth instrument that collects and presents rut data in a similar fashion.

Development of a rutting index for use in evaluating Texas highways has been a concern for years. Accordingly, the Texas Department of Transportation (DOT) purchased an instrument for network-level evaluation of the state's highway system. This instrument captures and processes data from its sensors. Data are reported as a transverse profile and summarized into a rut index for the left and right wheelpaths.

This paper presents an approach for developing models to process transverse profile information and thus quantify rut-depth information. These models are then correlated with several summary statistics. Results of the correlations identify a preferred model to quantify transverse profile data captured by the instrument's rut sensors. In this paper, the terms "transverse profile" and "rutting" are used interchangeably. The formulas and resulting calculations are specific to the Texas Automatic Road Analyzer (ARAN), but the methodology can be applied to any rut bar depth-monitoring system as long as the reported information is similar, including number of sensors and the measurement principle.

BACKGROUND

Pavement rutting is defined as the longitudinal depressions left in wheel tracks after repeated load applications (*1*); it results from compaction under load combined with the sideways shoving of pavement material. It has long been considered a measure of performance of flexible pavements, and its characteristics can be used as an indication of structural deterioration and road surface deformation (*2–4*). Excessive rutting directly affects the safety and comfort of the traveling public (*5*). Instrumentation to study pavement rutting has developed significantly in the past few years. Studies have focused on development and evaluation of techniques to measure and predict road roughness and rutting (*6–9*).

The Texas DOT has been using condition survey teams to collect rut-depth data for several years. The process involves placing a straightedge across a travel lane and physically measuring the depth of individual ruts. Because this method is slow and dangerous, the DOT purchased the ARAN unit manufactured by Highway Products International (HPI). This instrument can collect several types of pavement distress information under normal traffic conditions. The ARAN's roughness, rutting, and gyro subsystems have been evaluated over the past 2 years. The Center for Transportation Research (CTR) was contracted to perform this evaluation and help implement the study findings.

One of the ARAN's instrumentation subsystems is used to determine the amount of pavement rutting, as previously stated. Rutting data are acquired from ultrasonic sensors mounted on a rut bar attached to the ARAN's front bumper. This bar can be configured in any of three ways. The bumper itself has seven sensors spaced 1 ft apart from one end to the other. Additionally, one of two sets of extension wings can be attached to the main bumper. Each set has two wings—a left wing and a right wing. For a smaller set, each wing contains two sensors, and for a larger set, each wing has three sensors. The smaller wings allow 11 sensors to be active, providing for the evaluation of the entire width of a 10-ft travel lane. This configuration was chosen for this study.

Sensor data are processed and presented to the user in two formats. Individual sensor readings from a survey section are stored and presented as transverse profile data. They are reported in inches with a resolution to ¹⁄₁₀ in. and can also be viewed in a summary report representing the mean value of each sensor through the length of the survey section. Finall a rut index for each wheelpath is calculated and reported a percentage of rutting for that wheelpath.

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During CTR evaluation of the rut-depth subsystem, two operational characteristics were evaluated for their effects on reported rut data: selectable report interval and vehicle speed of operation. Repeatability of reported rut information was analyzed when these operational characteristics were changed. Findings indicate that operating speed did not significantly affect output from the ARAN's rut subsystem. The user must chose either 0.005 or 0.01 mi as a report interval to obtain the best subsystem repeatability. Additionally, the subsystem has a statistical output, called rut-depth index, that statistically summarizes readings from some, not all, of the sensors. During this study, it was found that the reported rut-depth index was not repeatable, no matter what the operational parameters, because the index is not resulted from the whole profit. Therefore, this index cannot practically be used to report rut depth.

These facts led CTR staff to investigate and develop a useful rut-depth index based on transverse profile data produced by the ARAN unit. It should be pointed out that a test section length of 0.2 mi was used in developing the rut index models. This length was selected because it was also used by the Texas DOT in calibrating its high-speed pavement roughness instrumentation. It was thus convenient, well marked, and readily available.

MEASUREMENT OF RELATIVE TRANSVERSE PROFILES

Pavement transverse profiles can be measured using a rut bar similar to the one shown in Figure 1, having 11 ultrasonic sensors to measure distance between pavement surface and each individual sensor. Horizontal distance between any two adjacent sensors is 1 ft. If a right-angle coordinate is defined as shown in Figure 1, then

$$\{X_i, i = 1, 2, \ldots, 11\} = \{-5, -4, -3, -2, -1, 0, 1, 2, 3, 4, 5\} \quad \text{(ft)}$$

and

$$Y_i + W_i = C \qquad (\text{all } i) \tag{1}$$

where

C = a constant,

$\{X_i, i = 1, 2, \ldots, 11\}$ = transverse distance sequence in x-axis,

$\{Y_i, i = 1, 2, \ldots, 11\}$ = discrete transverse profile sequence, and

$\{W_i, i = 1, 2, \ldots, 11\}$ = measured data sequence by the individual ultrasonic sensors.

Thus,

$$Y_i = C - W_i \qquad (\text{all } i) \tag{2}$$

To obtain Y_i, a transverse profile reference level should be given. If the mean value of the transverse profile sequence $\{Y_i, i = 1, 2, \ldots, 11\}$ is taken as the reference level, the relative discrete transverse profile sequence $\{T_i\}$ can be defined as follows:

$$T_i = Y_i - \overline{Y} \qquad (\text{all } i) \tag{3}$$

where

$$\overline{Y} = \frac{1}{11}\sum_{i=1}^{11} Y_i = \frac{1}{11}\sum_{i=1}^{11} (C - W_i) = C - \overline{W} \quad \text{and}$$

$$\overline{W} = \frac{1}{11}\sum_{i=1}^{11} W_i \tag{4}$$

By combining Equations 2, 3, and 4, the relative transverse profile sequence can be obtained by Equation 5:

$$T_i = C - W_i - C + \overline{W} = -(W_i - \overline{W}) \tag{5}$$

Statistical characteristics of transverse profiles on a given pavement section are of interest when transverse profile smoothness and associated rutting are evaluated. In this study, all sampled transverse profiles at each sampling station were averaged to obtain a mean transverse profile statistically representing transverse profile characteristics of a given pavement section.

Figure 2 shows a mean relative transverse profile, which was measured by the ARAN unit on Austin Test Section

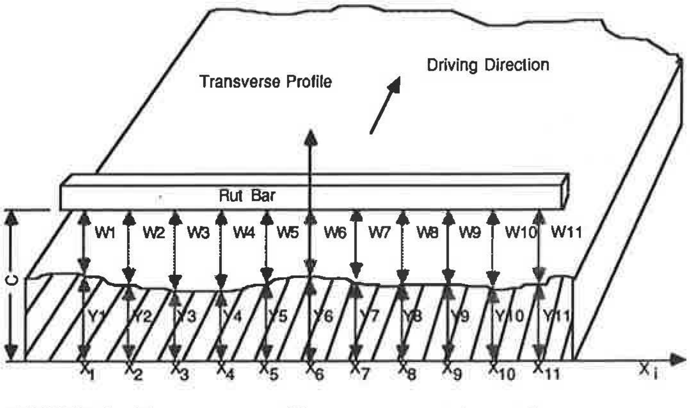

FIGURE 1 Transverse profile measurement by rut bar.

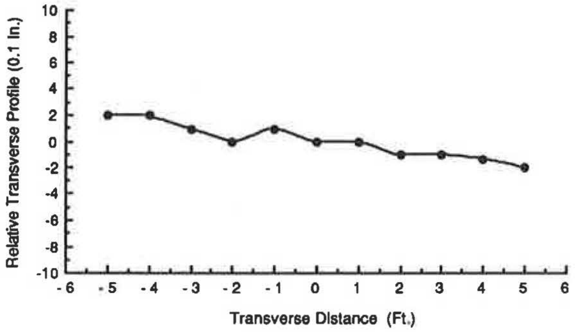

FIGURE 2 Relative transverse profile at ATS28.

ATS28 near Austin, Texas. Relative transverse profile was plotted as seen here.

POLYNOMIAL TRANSFORM OF RELATIVE TRANSVERSE PROFILES

Although relative transverse profiles cannot quantitatively characterize transverse profile smoothness and rutting, they can demonstrate it graphically. In a practical engineering sense, the purpose of measuring transverse profiles is to obtain objective statistics to evaluate transverse profile smoothness and associated rutting.

A transverse profile can be approximately fitted by the mathematical function

$$T_i = F(X_i) \tag{6}$$

where $F(X_i)$ is a continuous function of transverse distance X_i. One of the suitable models of $F(X_i)$ is the polynomial function

$$F(X) = A_0 + A_1X + A_2X^2 + \cdots + A_mX^m \tag{7}$$

where $A_j(j = 0, 1, \ldots, m)$ is the constant coefficient, and m is the order of the polynomial function. In this study, $m = 5$ was chosen. Then, by the notation shown in Figure 1, Equation 7 can be represented as

$$\begin{aligned} T_i = A_0 + A_1(X_i) + A_2(X_i)^2 + A_3(X_i)^3 \\ + A_4(X_i)^4 + A_5(X_i)^5 \end{aligned} \tag{8}$$

The explanation of Equation 8 is that the transverse profile shown in Figure 1 is the weighted summation of polynomials with weights (A_0, A_1, A_2, A_3, A_4, and A_5). The coefficients A_0, A_1, A_2, A_3, A_4, and A_5 thus approximately reflect the geometrical or graphical characteristics of the transverse profile and rut depth. In fact, this approach could be considered a "transformation" of the variables $\{T_i\}$ in the "space domain," to the variables $\{A_j\}$ in the "polynomial domain." Symbolically, this transformation is expressed as

$$\{T_i\} \Rightarrow \{A_j\} \tag{9}$$

Only the magnitudes of the coefficients of the regression model in Equation 7 are of concern because the magnitude of the coefficient A_j indicates weight of the content of the jth-order polynomial function in the associated transverse profile. The transformation shown in Equation 9 can be symbolically represented as follows:

$$\{T_i\} \Rightarrow \{a_j\} \tag{10}$$

where

$$a_j = |A_j| \qquad (j = 0, 1, \ldots, 5) \tag{11}$$

This transformation is defined as the "polynomial transform" in the following discussion, and the symbol "$\Rightarrow$" represents an irreversible polynomial transform.

It might be expected that one or more of the polynomial transform coefficients a_j could be sensitive to transverse profile smoothness. For example, for a given test section if the fourth-order polynomial transform a_4 is relatively larger than that of other test sections, then a_4 indicates that transverse profile of the given section is relatively rougher than that of the others.

In an extreme case all the polynomial transform coefficients would be zero, denoting a corresponding transverse profile that would ideally be constant or perfectly smooth with no rutting. Some or all of the coefficients a_j would be relatively large if the condition of the transverse profile rutting were relatively poor. But it should be mentioned that magnitudes of the coefficients a_j depend on graphic characteristics of the associated relative transverse profile. That is, the larger a_j, the more jth-order polynomial content there is in the transverse profile.

Applying the polynomial transform to evaluate a pavement transverse profile condition would be helpful in understanding the idea just presented. Figure 3 shows two transverse profiles from Austin Test Sections ATS04 and ATS28. Experience tells one that transverse profile smoothness of ATS28 is better, with less rutting, than ATS04, but this evaluation is subjective. Some data should be obtained from these transverse profiles to substantiate this subjective evaluation. If the polynomial transform is applied to these two sections, poly-

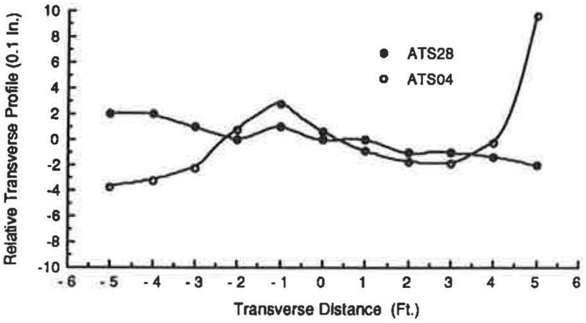

FIGURE 3 Relative transverse profiles at ATS04 and ATS28.

nomial transform coefficients of ATS04 and ATS08 can be listed as follows:

ATS04	*ATS28*
$a_0 = 1.305$	$a_0 = 6.434 \times 10^{-3}$
$a_1 = 1.128$	$a_1 = 0.2623$
$a_2 = 0.610$	$a_2 = 4.429 \times 10^{-3}$
$a_3 = 0.110$	$a_3 = 1.286 \times 10^{-2}$
$a_4 = 2.695 \times 10^{-2}$	$a_4 = 2.331 \times 10^{-4}$
$a_5 = 5.128 \times 10^{-4}$	$a_5 = 2.885 \times 10^{-4}$

From these coefficients it can be seen that all the polynomial transform coefficients of ATS04 are larger than those of ATS28. This example supports the statement that magnitudes of the coefficients a_j, to a certain degree, indicate the conditions of transverse profile smoothness and the associated rutting.

With substitution of the polynomial transform coefficients, the following linear multiple regression model adequately characterizes transverse smoothness and rutting (TSR):

$$\text{TSR} = K_1 + K_2 a_0 + K_3 a_1 + K_4 a_2 + K_5 a_3 + K_6 a_4 + K_7 a_5 \qquad (12)$$

In this model, some of the coefficients (K_n $n = 1, 2, \ldots, 7$) could be zero.

INDEX DEVELOPMENT

A standard reference should be used to develop a new index characterizing transverse profiles. In evaluating pavement transverse profile smoothness and rutting, two statistics are often used: mean value and standard deviation of the measured transverse profile data. But these two statistics do not take into account the sequence of such data. In other words, graphic characteristics of transverse profiles do not affect the two statistics if data sequence values of the associated transverse profile are kept the same. In fact, the graphic characteristic of the transverse profiles is an important factor in evaluating highway safety and passenger comfort. It will affect transverse profile smoothness and rutting.

Graphic characteristics of pavement transverse profile can be obtained from the polynomial transform. The regression model shown in Equation 7 may be a good candidate for evaluating pavement transverse profile smoothness and rutting, although it does not have an obvious physical unit. The procedure of modeling and data analysis for developing indexes characterizing transverse profiles will be presented later. The Texas ARAN served as measuring equipment to collect pavement serviceability index and transverse profile data. Thus, the resulting models are based on the ARAN unit. But the methodology presented here can be applied to other rut-depth measuring equipment.

Choice of Reference Statistics

Transverse profile standard deviation SD was chosen as one of the reference statistics in developing a new index to characterize transverse profiles. Another index, TD, was chosen as a reference statistic defined as

$$\text{TD} = \frac{(\text{TDR} + \text{TDL})}{2} \qquad (13)$$

where

$$\text{TDR} = \frac{(T_1 - 2T_3 + T_5)}{2}, \quad \text{and}$$

$$\text{TDL} = \frac{(T_7 - 2T_9 + T_{11})}{2} \qquad (14)$$

It can be said that TDR is the second-order difference of the outside wheelpath transverse profile and TDL is the second-order difference of the inside wheelpath transverse profile. Although TDR and TDL do not cover the entire transverse profile, they reflect rutting characteristics in the outside and inside wheelpaths, respectively.

Serviceability index (SI) was also considered as a reference statistic. Because the roughness measuring subsystem of the ARAN unit is response-type, measured SI values are the responses of the measuring vehicle to longitudinal and transverse pavement roughness. The SI value thus should be correlated with transverse profile roughness and rutting.

Data Collection and Processing

Field data were collected in the summer of 1989 using the ARAN unit. Table 1 presents measured transverse profile data collected from several flexible pavements. However, all raw data in Table 1 had to be subtracted from associated mean values to obtain the relative transverse profiles.

Table 2 gives the fifth-order polynomial curve-fitting coefficients of the relative transverse profile data, R^2-values of the curve fitting, and values of the reference statistics. Linear correlation between the reference statistics and coefficients can be conducted to evaluate sensitivity of reference statistics to coefficients. Correlation analysis results are as follows:

Statistics	a_0	a_1	a_2	a_3	a_4	a_5
SI	.656	.471	.873	.386	.885	.276
SD	.765	.439	.794	.322	.644	.192
TD	.992	.259	.835	.114	.482	.221

It is seen that the coefficients a_2 and a_4 correlate relatively well with SI. This further proves that measured roughness from a response-type roughness-measuring system has a certain correlation with transverse profile characteristics—that is, response of a vehicle is due not only to longitudinal roughness but also to transverse profile smoothness. However, this cannot be seen if the standard deviations SD of the transverse profiles are considered, because the R^2-value between SI and SD is relatively small.

Transverse Profile Smoothness and Rut-Depth Index Specifications and Development

The multiple regression model in Equation 12 will be considered as the basis for index modeling. In modeling, specifi-

TABLE 1 TRANSVERSE PROFILE DATA AT AUSTIN TEST SECTIONS

	Ultrasonic Sensors (0.1 Inch)										
ATS	1	2	3	4	5	6	7	8	9	10	11
01	145.5	138.3	143.4	139.2	141	137.4	142	141.5	138	143.5	137.5
03	142.8	138.8	142.4	139.2	140.8	142	145.2	145	143.4	145.2	145.2
04	143.8	137.3	143.3	139.3	142.3	139.5	140.3	141.8	141	142	130.5
07	144	141.7	143.7	143	143	142	145	143.3	142	144	145.7
08	141.7	140.7	141.7	141.7	141.7	141.3	145	143	142	144	146
09	140.7	141	141	141	142	141.7	144.7	143.3	142.3	144	146
12	142.3	141.3	142.3	142.7	143	142	147	143.7	142	145	148.7
15	142.5	141	142	141	141	141.5	145	142.5	142	144	146.5
19	138.7	142	140.7	142	142	142	145.3	143.7	142.7	144.7	146.3
20	137.7	141	139.7	141	141	140.7	145.7	142.7	141	144	147
22	142.3	140.3	142.3	142	142.3	140.7	145	142	141	143.3	146
25	146	139	144	140	142	138.3	142.7	142	139.3	144	138
27	141.3	141.3	141	142.3	142	142	144	143	142.3	143.3	144
28	141	142	141	143	142	143	144.3	144	143	144	145
30	152.3	139	149.7	142	146.7	138.7	146	139.3	137	142.7	147.7
31	142.7	138.7	142.7	140	141.7	139.7	142.7	142.3	139.7	143.7	143.3
41	145	141.7	144	142.7	144	141	140.7	140	140	140	139.7
42	140.7	140.3	141	142	142	141	143	142	141	143	143
43	140	140	140	141	141	141	142.3	142	141	143	142
55	142.3	138	142	140	142	138.7	141	141	139	141.7	140

TABLE 2 TRANSVERSE PROFILE POLYNOMIAL TRANSFORM COEFFICIENTS AND REFERENCE STATISTICS

	Coefficients of Polynominal Transform and R^2 Values							Reference Statistics		
ATS	A0	A1	A2	A3	A4	A5	R^2	SI	SD	TD
01	3.009	-0.5435	-0.6773	1.338E-2	2.113E-2	1.619E-3	0.97	2.61	2.775	-0.800
03	1.462	-2.149	-0.2341	0.1866	4.953E-3	-4.423E-3	0.98	3.67	2.370	-1.845
04	1.305	-1.128	-0.6103	0.1102	2.695E-2	-5.128E-4	0.97	1.96	3.703	-1.250
07	1.486	-8.536E-2	-0.2290	-8.559E-3	4.516E-3	2.083E-4	0.97	4.40	1.257	-1.725
08	1.287	-0.4140	-0.2027	4.050E-3	4.167E-3	-1.923E-4	0.98	4.03	1.683	-1.464
09	0.8155	-0.5950	-0.1449	2.674E-2	3.176E-3	-9.776E-4	0.98	3.76	1.758	-0.989
12	1.856	-0.1118	-0.2837	-3.821E-2	5.536E-3	6.731E-4	0.98	4.37	2.334	-2.127
15	1.285	-0.4829	-0.1307	7.503E-3	1.457E-4	-1.603E-4	0.99	4.28	1.804	-1.727
19	0.5372	-0.3694	-0.1628	-7.233E-3	6.148E-3	-3.365E-4	1.00	4.17	2.160	-0.452
20	1.126	-0.1454	-0.2463	-4.861E-2	7.488E-3	6.891E-4	1.00	3.57	2.695	-1.216
22	1.862	1.867E-2	-0.3016	-2.899E-2	6.468E-3	5.289E-4	0.98	4.40	1.734	-2.180
25	2.827	-0.5363	-0.6521	1.773E-2	2.075E-2	1.442E-3	0.99	3.12	2.653	-3.068
27	0.4207	-0.1398	-8.814E-2	-2.316E-2	2.593E-3	7.051E-4	0.91	4.29	1.049	-0.332
28	6.434E-3	-0.2623	-4.429E-3	-1.286E-2	2.331E-4	2.885E-4	0.92	4.43	1.328	0.223
30	6.137	-0.9310	-0.9022	4.458E-2	1.623E-2	-1.026E-3	0.99	2.18	5.054	-7.727
31	2.226	-0.8512	-0.4268	8.402E-2	1.145E-2	-2.083E-3	0.93	3.58	1.721	-2.443
41	0.9126	-0.9509	-0.1633	5.597E-2	4.050E-3	-1.555E-3	0.99	3.93	1.914	-1.107
42	0.8545	-2.294E-2	-0.2128	-2.164E-2	7.168E-3	5.289E-4	0.84	4.24	0.988	-0.720
43	0.4280	-0.3217	-0.1402	-2.375E-3	5.478E-3	2.885E-4	0.86	4.42	1.010	-0.232
55	1.957	-0.4272	-0.4403	5.570E-2	1.375E-2	-1.166E-3	0.90	2.95	1.480	-0.825

cations of the model are necessary because it is improper to use all of the polynomial transform coefficients. Specifications of models can be judged by factors such as R^2-value, sign of coefficient, absolute magnitude of coefficient, and simplicity.

Table 3 lists regression model specifications. The indexes SI, SD, and TD are dependent variables, and the polynomial transform coefficients a_j ($j = 0, \ldots, 5$) are independent variables. Table 4 shows results of the multiple regression models specified in Table 3.

Several important factors must be considered to choose adequate models, as can be seen in Table 4. Factors of R^2-value, sign of coefficient, absolute magnitude of coefficient, and simplicity are concerned, as just stated. Model choices for the references SI, SD, and TD are now discussed individually.

TABLE 3 SPECIFICATIONS FOR MULTIPLE REGRESSION MODEL

	Models	Dependent Variables: SI, SD, TD
Independent Variables a_0 a_1 a_2 a_3 a_4 a_5	1	a_0 a_1 a_2 a_3 a_4 a_5
	2	a_1 a_2 a_3 a_4 a_5
	3	a_1 a_2 a_3 a_4
	4	a_2 a_3 a_4 a_5
	5	a_0 a_2 a_4
	6	a_2 a_3 a_4
	7	a_2 a_4

TABLE 4 COEFFICIENTS AND R^2-VALUES OF ALL REGRESSION MODELS

			Coefficients and R^2 Values of All the Regression Models													
		Independent Variables	Model 1		Model 2		Model 3		Model 4		Model 5		Model 6		Model 7	
Dependent Variables	SI	Constant	4.739	$R^2 = 0.912$	4.704	$R^2 = 0.895$	4.732	$R^2 = 0.875$	4.685	$R^2 = 0.860$	4.646	$R^2 = 0.839$	4.703	$R^2 = 0.855$	4.611	$R^2 = 0.822$
		a_0	6.718								6.626					
		a_1	-0.552		-0.600		-0.423									
		a_2	-68.70		-1.395		-1.139		-1.593		-67.68		-1.411		-1.365	
		a_3	-2.383		-1.153		0.742		-5.192				-3.245			
		a_4	1152.7		-45.09		-54.03		-42.42		1125.4		-48.09		-54.12	
		a_5	234.96		216.42				107.39							
	SD	Constant	0.919	$R^2 = 0.812$	0.933	$R^2 = 0.810$	0.860	$R^2 = 0.706$	0.971	$R^2 = 0.751$	1.006	$R^2 = 0.654$	0.894	$R^2 = 0.688$	1.016	$R^2 = 0.653$
		a_0	-2.966								-1.842					
		a_1	0.981		1.002		0.486									
		a_2	34.60		4.883		4.137		5.215		22.82		4.449		4.388	
		a_3	6.690		6.306		-0.279		12.45				4.300			
		a_4	-592.5		-63.69		-37.59		-68.16		-364.3		-44.42		-36.42	
		a_5	-640.2		-632.1				-450.0							
	TD	Constant	0.172	$R^2 = 0.999$	0.170	$R^2 = 0.999$	0.187	$R^2 = 0.997$	0.160	$R^2 = 0.997$	0.143	$R^2 = 0.995$	0.106	$R^2 = 0.994$	0.078	$R^2 = 0.993$
		a_0	0.285								0.143					
		a_1	-0.274		-0.276		-0.158									
		a_2	-16.20		-13.34		-13.17		-13.44		-14.69		-13.13		-13.11	
		a_3	-1.235		-1.198		0.301		-2.888				-1.011			
		a_4	314.9		264.1		258.2		265.3		283.6		259.5		257.7	
		a_5	144.7		143.9				93.81							

NOTE: SI, SD, and TD are dependent variables; a_i ($i = 0, 1, \ldots, 5$) are independent variables.

SI

Besides longitudinal profile roughness, pavement serviceability index SI measured by a response-type roughness-measuring system such as the ARAN unit is affected by transverse profile smoothness. The smoother the transverse profile, the better the serviceability, or the larger the SI. Mathematically, this logical relationship requires the coefficients of the regression model in Equation 12 to have negative signs according to the meaning of the polynomial transform coefficients. The multiple regression results in Table 4 indicate that only Models 6 and 7 are adequate if the signs are considered. However, the R^2-value of Model 6 is larger than that of Model 7. Model 6 was chosen for this study.

SD

Transverse profile data standard deviation SD does not concern the sequence of transverse profile data, so graphic characteristics of the transverse profile do not significantly affect the SD value. Thus there is no strict requirement for signs of the coefficients of the multiple regression model in Equation 12. Model 1 was chosen for the multiple regression model of Equation 12 because it has the best correlation with SD (higher R^2-value).

TD

According to the definition of TD, it does not consider the whole transverse profile. There is thus no strict requirement for the signs of the multiple regression model in Equation 12. Model 3 was chosen because of simplicity and the R^2-value.

On the basis of the three references SI, SD, and TD and the results of model choice, the three resulting multiple regression models are as follows:

Based on SI:

$$\mathrm{TSR}_{\mathrm{SI}} = 4.703 - 1.411a_2 - 3.245a_3 - 48.09a_4 \tag{15}$$

Based on SD:

$$\begin{aligned}\mathrm{TSR}_{\mathrm{SD}} = {} & 0.919 - 2.966a_0 + 0.981a_1 + 34.60a_2 \\ & + 6.690a_3 - 592.5a_4 - 640.2a_5\end{aligned} \tag{16}$$

Based on TD:

$$\begin{aligned}\mathrm{TSR}_{\mathrm{TD}} = {} & 0.187 - 0.158a_1 - 13.17a_2 \\ & + 0.301a_3 + 258.2a_4\end{aligned} \tag{17}$$

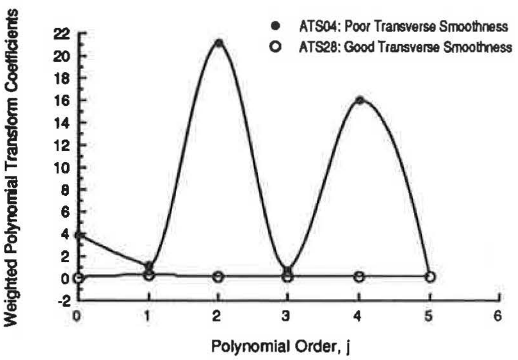

FIGURE 4 Weighted polynomial transforms of transverse profiles at ATS04 and ATS28.

The TSRs shown in Equations 15, 16, and 17 can be considered as the indexes characterizing transverse profile smoothness or rutting.

The following example may be useful in better explaining application of the polynomial transform described earlier. In this example, the model of Equation 16 will be used. The polynomial transform can be expressed by a curve. The horizontal axis (x-axis) is the polynomial order j, and the vertical axis (y-axis) is the weighted polynomial transform coefficient $|K_{j+2}|a_j$ ($j = 0, \ldots, 5$) as expressed in Equation 12, but the weights are the absolute values of the associated coefficients of the multiple regression models. From Equation 16, the weights can be listed as follows:

Polynomial Order j	*Polynomial Transform Coefficient*	*Weights*
0	a_0	2.966
1	a_1	0.981
2	a_2	34.60
3	a_3	6.690
4	a_4	592.5
5	a_5	640.2

Figure 4 shows the weighted polynomial transforms of Austin Test Sections ATS04 and ATS28. Conditions of transverse profile smoothness on ATS04 and ATS28 can be easily distinguished by use of the polynomial transform. It should be mentioned that the rutting judgment from Figure 3 is qualitative and that from Figure 4 is quantitative. The two judgments have essential differences.

Figures 5, 6, and 7 show correlation of the multiple regression models with the references SI (Equation 15), SD (Equation 16), and TD (Equation 17), respectively. The regression model shown in Equation 17 has a very good correlation with TD.

DISCUSSION OF RESULTS

1. In developing indexes characterizing transverse profiles, three reference statistics (SI, SD, and TD) were selected. The purpose was to prove that the developed theoretical model concept and structure correlate with the chosen references. The correlations found also prove the implied use and applicability of the polynomial transform in evaluating pavement transverse profile smoothness. Of course, some better models could be found if the polynomial transform coefficients were directly correlated with subjective judgments on pavement safety and the passenger's degree of comfort. Judgments on rutting by a survey panel for a number of test sections could also be used to calibrate the model coefficients.

2. The presented multiple regression model for Equation 12 can quantitatively reflect the graphical characteristics of transverse profiles. The TSR indexes were developed to evaluate the transverse profile of an asphaltic pavement section. However, the resulting correlation analysis showed no good correlations among the reference statistics SI, SD, and TD. Their correlations are as follows:

Reference Pairs	R^2-*Values*
SI–SD	0.635
SI–TD	0.376
SD–TD	0.595

The indexes from Equations 15, 16, and 17 should have better correlations with SI, SD, and TD, respectively. They can be used to evaluate pavement smoothness and rutting conditions.

3. In new pavement construction, longitudinal roughness is usually used to evaluate whether the constructed pavement satisfies the design requirements. Research has been conducted on longitudinal roughness specifications (*10*). Trans-

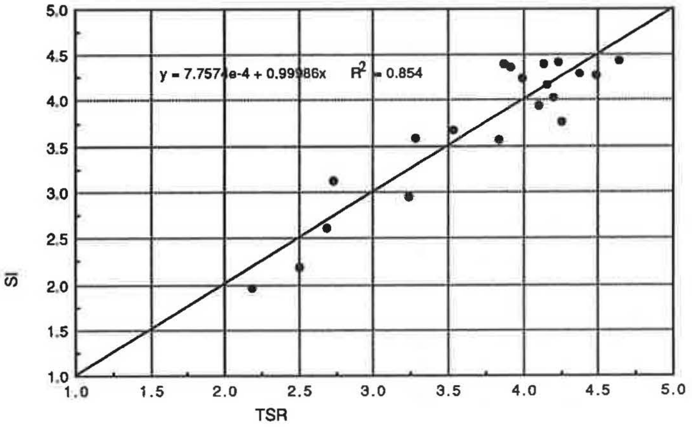

FIGURE 5 Correlation between SI and TSR.

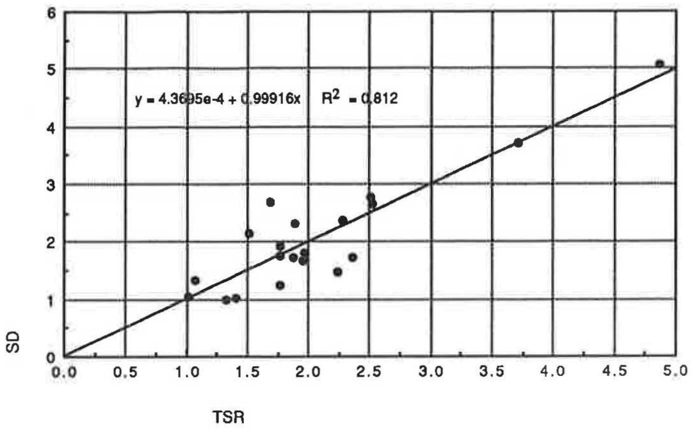

FIGURE 6 Correlation between SD and TSR.

verse smoothness of newly constructed pavement is also an important factor in determining if the constructed pavement satisfies the design requirements. In this case, the index TSR might be a good candidate for a quality-control statistic in evaluating newly constructed pavement. However, further research is needed for more effective application of the developed methodology.

4. Certain differences appear among the multiple regression models of Equations 15, 16, and 17. These models evaluate pavement transverse profile smoothness from different angles according to their associated references. For TSR_{SI} from Equation 15, the larger the TSR_{SI}, the better the transverse profile smoothness because the model was derived from correlation with SI. But for TSR_{SD} and TSR_{TD} from Equations 16 and 17, the smaller the TSR_{SD} and TSR_{TD}, the better the conditions of transverse profile smoothness, because the models were derived from correlations with SD and TD, respectively.

CONCLUSIONS

The study used the rut-depth subsystem of the ARAN unit. The methodology in developing transverse profile smoothness and the rutting indexes can be applied to any system having a rut bar with sensor configuration as shown in Figure 1. All the modeling coefficients must be found if a different system is used.

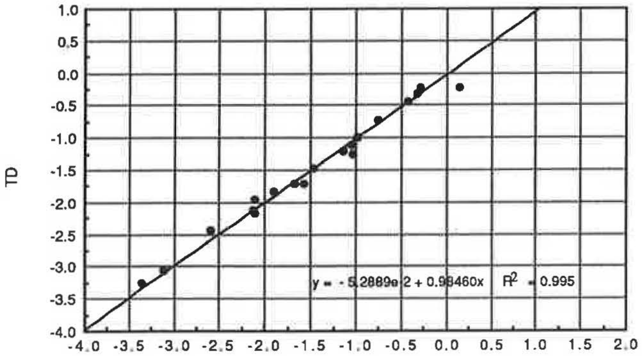

FIGURE 7 Correlation between TD and TSR.

The index models shown in Equations 15, 16, and 17 characterize transverse profile smoothness from three different angles. Further research is needed to decide which index best correlates the safety factor and passenger's degree of comfort.

Relative transverse profile was obtained by averaging all the transverse profiles of a pavement section of a given length, thus smoothing the data. Research is needed to determine the most appropriate pavement length for calculating the TSR. For example, a 0.2-mi section could have the same TSR as a 4-mi section. Resolution of the TSR statistic must be determined by highway agencies for network- or project-level needs.

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Mixed-Integer Programming Model for AASHTO Flexible Pavement Design

XIN CHEN, GERMAN CLAROS, AND W. RONALD HUDSON

A mixed-integer programming model is described that is based on the AASHTO design procedure for flexible pavements and formulated for selecting pavement materials and determining surface, base, and subbase course thicknesses. The objective of the model is to minimize the total cost of pavement structures while meeting the constraints of AASHTO flexible pavement design equations and user-defined criteria. The Flexible Pavement Optimal Design computer program that interfaces with the optimization package LINDO has been developed to obtain quick solutions. Two solutions are given by the program: nonintegers and integers. The program can be used for flexible pavement thickness design in cases in which one or more materials are available for each of three layers if the layer characteristics and material properties are known. The minimum-cost solution can be obtained from the program. Sensitivity analysis has demonstrated that great benefits can be obtained by using this program.

In flexible pavement design, there are usually several material types available for surface, base, and subbase courses. There are many combinations of layer thicknesses for each of the three layers when AASHTO design equations (*1*) are used. In pavement construction, a small reduction in the unit cost of pavement structures can result in considerable savings for the entire project. Therefore, obtaining the best materials at minimum cost is important.

Because the AASHTO DNPS86 program (*2*) has no optimization function, the solutions given by the program may not be the least expensive. Nicholls (*3*) developed a nonlinear optimization program (DNPS86O) using DNPS86 as a subroutine. A minimum-cost solution for the whole design period is obtained by changing design reliability, performance period of initial pavement, and two of the three thicknesses of flexible pavements. Rouphail (*4*) formulated a mixed-integer–linear programming model for minimum-cost design of flexible pavements by changing the number, type, and thickness of paving materials. But the problem of material selection is not addressed in either model.

The flexible pavement design problem is here formulated as a mixed-integer programming model (*5*). The model can select the best combination of different pavement materials for the three layers of pavement structure and give the minimum-cost solution for the selected materials accordingly, while meeting constraints of the AASHTO design equations and user-defined criteria (given a certain level of reliability, performance period of initial pavement, and other input data). A computer program interfacing with the optimization package LINDO (*6*) is developed to get quick solutions. Besides the minimum-cost noninteger solution (layer thicknesses are not rounded to nearest ½ in.), the minimum-cost integer solution (layer thicknesses are integers in inches) can also be obtained from the program.

AASHTO FLEXIBLE PAVEMENT DESIGN EQUATIONS

In the design guide for flexible pavements, the following equations are used to compute the structural number and layer thicknesses:

$$\log(W_{18}) = Z_r S_0 + 9.36 \log(\mathrm{SN} + 1) - 0.20 + \frac{\log\left[\dfrac{p_0 - p_t}{4.2 - 1.5}\right]}{0.40 + \dfrac{1{,}094}{(\mathrm{SN} + 1)^{5.19}}} + 2.32 \log M_r - 8.07 \quad (1)$$

$$\mathrm{SN} = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3 \quad (2)$$

$$D_1^* \geq \frac{\mathrm{SN}_1}{a_1} \quad (3)$$

$$\mathrm{SN}_1^* = a_1 D_1^* \geq \mathrm{SN}_1 \quad (4)$$

$$D_2^* \geq \frac{\mathrm{SN}_2 - \mathrm{SN}_1^*}{a_2 m_2} \quad (5)$$

$$\mathrm{SN}_2^* = a_2 m_2 D_2^* \quad (6)$$

$$\mathrm{SN}_1^* + \mathrm{SN}_2^* \geq \mathrm{SN}_2 \quad (7)$$

$$D_3^* \geq \frac{\mathrm{SN}_3 - (\mathrm{SN}_1^* + \mathrm{SN}_2^*)}{a_3 m_3} \quad (8)$$

where

W_{18} = predicted number of 18-kip equivalent single axle load applications;
Z_r = standard normal deviate;
S_0 = combined standard error of the traffic prediction and performance prediction;
p_0 = initial design serviceability index;
p_t = design terminal serviceability index;
M_r = resilient modulus (psi);
SN = structural number indicative of total pavement thickness required;

Center for Transportation Research, University of Texas, Austin, Tex. 78712.

SN_i = structural number corresponding to modulus of base ($i = 1$), subbase ($i = 2$) and roadbed soil ($i = 3$, $SN_3 = SN$);
a_i = ith layer coefficient;
m_i = ith layer drainage coefficient;
D_i = ith layer thickness in inches; and
D_i^*, SN_i^* = values actually used ($i = 1, 2, 3$).

Equation 1 shows that the structural number required for the total pavement structure can be uniquely determined under the same traffic condition and at a certain level of reliability, but that different materials for base and subbase courses with different resilient moduli will have different structural numbers. Equation 2 shows that the thicknesses of surface, base, and subbase courses depend on layer coefficients, drainage coefficients, and a structural number associated with different layers. Equations 3 through 8 are actually used for the computation of layer thicknesses.

It can be seen from these equations that there are many solutions to layer thicknesses for a particular problem with given traffic, environment, reliability, and materials. An optimal solution with minimum total cost for a pavement structure can be found with trial-and-error methods, but it may take much design time. There is no simple method such as using the ratio of SN to unit cost for quick thickness design.

In cases in which several materials are available for each of the three layers, a simple method using the ratio of the layer coefficient multiplying by the drainage coefficient to unit cost can be used to select the types of materials for the design of noninteger layer thickness, but it may not be true for the design of integer layer thickness. This will be illustrated in a later example.

MIXED INTEGER PROGRAMMING MODEL

Let m, n, r be the number of types of surface, base, and subbase courses available for a project in which the resilient moduli, layer coefficients, drainage coefficients, and unit costs corresponding to each material are known. Then the material selection and thickness design problems can be formulated as follows:

Objective function:

Minimize

$$\sum_{i=1}^{m} C_{1i}D_{1i} + \sum_{j=1}^{n} C_{2j}D_{2j} + \sum_{k=1}^{r} C_{3k}D_{3k} \tag{9}$$

In optimization, the objective value of Equation 9 is divided by 36 to get the unit cost of dollars per square yard.

Subject to

1. Constraints of AASHTO equations:

$$SN_1^* \geq SN_1 \tag{10}$$

$$SN_1^* + SN_2^* \geq SN_2 \tag{11}$$

$$SN_1^* + SN_2^* + SN_3^* \geq SN_3 \tag{12}$$

$$SN_1^* = \sum_{i=1}^{m} a_{1i}D_{1i} \tag{13}$$

$$SN_2^* = \sum_{j=1}^{n} a_{2j}m_{2j}D_{2j} \tag{14}$$

$$SN_3^* = \sum_{k=1}^{r} a_{3k}m_{3k}D_{3k} \tag{15}$$

2. Constraints of structural number

$$SN_1 \geq \sum_{i=1}^{m} SN_{1i}X_{1i} \tag{16}$$

$$SN_2 \geq \sum_{j=1}^{n} SN_{2j}X_{2j} \tag{17}$$

$$SN_3 \geq \sum_{k=1}^{r} SNX_{3k} \tag{18}$$

3. Constraints of maximum thicknesses

$$D_{1i} \leq D_{1\max}X_{1i} \qquad (i = 1, 2, \ldots, m) \tag{19}$$

$$D_{2j} \leq D_{2\max}X_{2j} \qquad (j = 1, 2, \ldots, n) \tag{20}$$

$$D_{3k} \leq D_{3\max}X_{3k} \qquad (k = 1, 2, \ldots, r) \tag{21}$$

4. Constraints of minimum thicknesses

$$\sum_{i=1}^{m} D_{1i} \geq D_{1\min} \tag{22}$$

$$\sum_{j=1}^{n} D_{2j} \geq D_{2\min} \tag{23}$$

$$\sum_{k=1}^{r} D_{3k} \geq D_{3\min} \tag{24}$$

5. Constraints of surface, base and subbase course

$$\sum_{i=1}^{m} X_{1i} = 1 \tag{25}$$

$$\sum_{j=1}^{n} X_{2j} = 1 \tag{26}$$

$$\sum_{k=1}^{r} X_{3k} = 1 \tag{27}$$

where

C_{1i} = unit cost of ith type of surface course in dollars per cubic yard;
C_{2j} = unit cost of jth type of base course in dollars per cubic yard;
C_{3k} = unit cost of kth type of subbase course in dollars per cubic yard;
D_{1i} = layer thickness of ith type of surface course in inches;

D_{2j} = layer thickness of jth type of base course in inches;

D_{3k} = layer thickness of kth type of subbase course in inches;

a_{1i} = layer coefficient of ith type of surface course;

a_{2j} = layer coefficient of jth type of base course;

a_{3k} = layer coefficient of kth type of subbase course;

m_{2j} = drainage coefficient of jth type of base course;

m_{3k} = drainage coefficient of kth type of subbase course;

$D_{s\,\max}$ = maximum thicknesses of surface ($s = 1$), base ($s = 2$), and subbase ($s = 3$) course in inches;

$D_{s\,\min}$ = minimum thicknesses of surface ($s = 1$), base ($s = 2$), and subbase ($s = 3$) course in inches;

SN_s = minimum structural number corresponding to modulus of selected types of base course ($s = 1$), subbase ($s = 2$) course, and effective resilient modulus of roadbed soil ($s = 3$);

SN_s^* = structural number actually used in the models ($s = 1, 2, 3$);

SN_{1i} = structural number corresponding to ith base course calculated using AASHTO Equation 1;

SN_{2j} = structural number corresponding to jth subbase course calculated using AASHTO Equation 1;

SN = structural number corresponding to effective resilient of roadbed soil calculated using AASHTO Equation 1;

X_{1i} = 1 if ith type of surface course is selected, otherwise $X_{1i} = 0$;

X_{2j} = 1 if jth type of base course is selected, otherwise $X_{2j} = 0$;

X_{3k} = 1 if kth type of subbase course is selected, otherwise $X_{3k} = 0$; and

m, n, r = number of surface, base, and subbase courses available, respectively ($i = 1, 2, \ldots, m$; $j = 1, 2, \ldots, n$; $k = 1, 2, \ldots, r$).

In the model above, Equations 10 through 15 correspond to Equations 3 through 8, which are used to compute layer thicknesses required once material types of the three layers are selected. Equations 16 through 18 are used to select the structural numbers computed by Equation 1 for different layers. Equations 19 through 21 ensure that the layer thicknesses of selected materials are no more than the maximum thicknesses specified for the materials, while those of materials not selected are equal to zero. Equations 22 through 24 ensure that the layer thicknesses of selected materials are no less than the minimum thicknesses specified for the materials. Equations 25 through 27 ensure that only one material is selected for surface, base, and subbase courses, respectively.

This model is able to select the best combination of materials for the three layers and determine the optimal layer thicknesses for the selected materials. If there is only one available type of material for each layer, that is, $m = n = k = 1$, the problem is simplified only to the optimization of layer thicknesses; the formulation of the simplified model is then:

Objective function:

Minimize

$$C_1D_1 + C_2D_2 + C_3D_3 \tag{28}$$

Subject to

$$\mathrm{SN}_1^* \geq \mathrm{SN}_1 \tag{29}$$

$$\mathrm{SN}_1^* + \mathrm{SN}_2^* \geq \mathrm{SN}_2 \tag{30}$$

$$\mathrm{SN}_1^* + \mathrm{SN}_2^* + \mathrm{SN}_3^* \geq \mathrm{SN}_3 \tag{31}$$

$$\mathrm{SN}_1^* = a_1D_1 \tag{32}$$

$$\mathrm{SN}_2^* = a_2m_2D_2 \tag{33}$$

$$\mathrm{SN}_3^* = a_3m_3D_3 \tag{34}$$

$$D_1 \geq D_{1\min} \tag{35}$$

$$D_2 \geq D_{2\min} \tag{36}$$

$$D_3 \geq D_{3\min} \tag{37}$$

$$D_1 \leq D_{1\max} \tag{38}$$

$$D_2 \leq D_{2\max} \tag{39}$$

$$D_3 \leq D_{3\max} \tag{40}$$

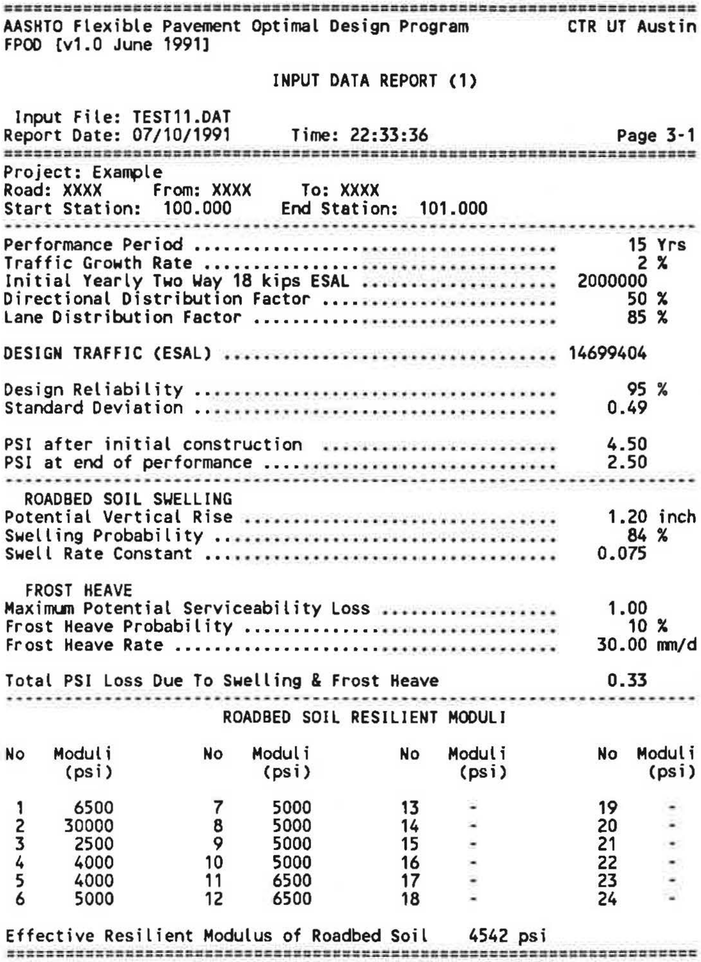
where C_1, C_2, C_3 are the unit costs of surface, base, and subbase course (dollars per cubic yard), and all other variables are as defined before.

A computer program—Flexible Pavement Optimal Design (FPOD)—was developed on the basis of AASHTO equations and the model. The program interfaces with the optimization package LINDO, which can obtain quick solutions. In terms of the *AASHTO Guide for Design of Pavement Structures* and from a practical point of view, all the layer thicknesses should be rounded to the nearest ½ in. (integer solution). For this reason, the program gives two types of solution (nonintegers and integers). It will be demonstrated next that integer solutions always cost more than noninteger solutions.

NUMERICAL EXAMPLE

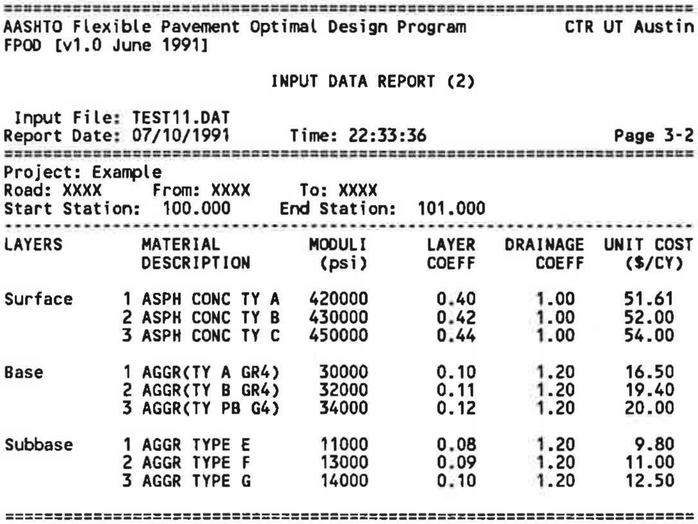
Consider a flexible pavement design for which three types of materials are available for each of the three layers, respectively. The default data of DNPS86 program are used except for those of pavement layer characteristics, material properties, and costs. The minimum base and subbase thicknesses are set to 6 in. FPOD printouts are shown in Figures 1 through 3. Figures 1 and 2 list all the input data, and Figure 3 presents the optimal solutions.

As presented in Figure 3, for noninteger solution, FPOD selects Asphalt Concrete Type C, Aggregate Type A G4, and Aggregate Type F for surface, base, and subbase course material. The thicknesses of the layers are 9.11, 6.00, and 17.31 in., respectively. With regard to integer solution, the model selects Aggregate Type G instead of Aggregate Type F for subbase course material as in the case of the noninteger solution. The total costs of the pavement structure for noninteger solution and integer solution are \$21.72/yd^2 and \$21.80/yd^2, respectively. In this case, the integer solution costs 0.37 percent more than the noninteger solution.



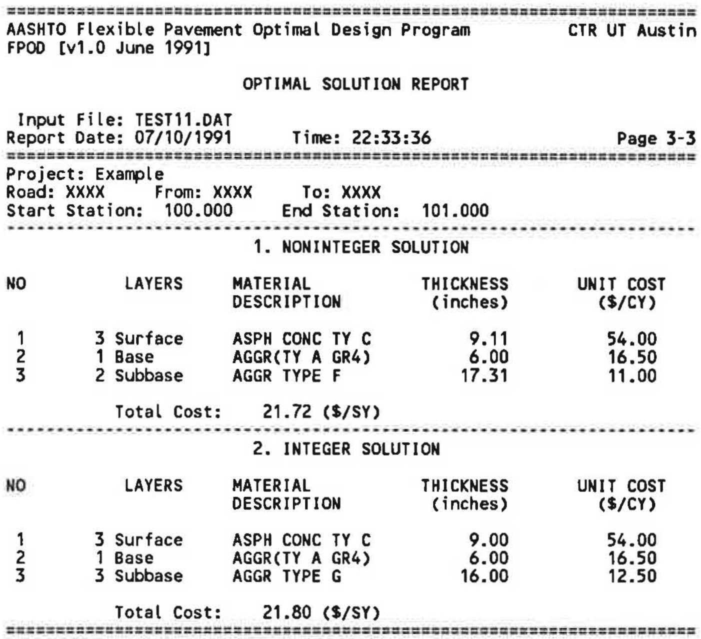
```
========================================================================
AASHTO Flexible Pavement Optimal Design Program           CTR UT Austin
FPOD [v1.0 June 1991]

                              INPUT DATA REPORT (1)

 Input File: TEST11.DAT
Report Date: 07/10/1991      Time: 22:33:36                     Page 3-1
========================================================================
Project: Example
Road: XXXX     From: XXXX      To: XXXX
Start Station:  100.000     End Station:  101.000
------------------------------------------------------------------------
Performance Period ......................................      15 Yrs
Traffic Growth Rate .....................................       2 %
Initial Yearly Two Way 18 kips ESAL .....................  2000000
Directional Distribution Factor .........................      50 %
Lane Distribution Factor ................................      85 %

DESIGN TRAFFIC (ESAL) ................................... 14699404

Design Reliability ......................................      95 %
Standard Deviation ......................................    0.49

PSI after initial construction  .........................    4.50
PSI at end of performance ...............................    2.50
------------------------------------------------------------------------
  ROADBED SOIL SWELLING
Potential Vertical Rise .................................    1.20 inch
Swelling Probability ....................................      84 %
Swell Rate Constant .....................................   0.075

  FROST HEAVE
Maximum Potential Serviceability Loss ...................    1.00
Frost Heave Probability .................................      10 %
Frost Heave Rate ........................................   30.00 mm/d

Total PSI Loss Due To Swelling & Frost Heave                 0.33
------------------------------------------------------------------------
                        ROADBED SOIL RESILIENT MODULI

No    Moduli          No    Moduli         No    Moduli          No   Moduli
      (psi)                 (psi)                (psi)                 (psi)

 1     6500            7     5000          13      -             19      -
 2    30000            8     5000          14      -             20      -
 3     2500            9     5000          15      -             21      -
 4     4000           10     5000          16      -             22      -
 5     4000           11     6500          17      -             23      -
 6     5000           12     6500          18      -             24      -

Effective Resilient Modulus of Roadbed Soil     4542 psi
========================================================================
```

FIGURE 1 FPOD input data report (1).



```
========================================================================
AASHTO Flexible Pavement Optimal Design Program           CTR UT Austin
FPOD [v1.0 June 1991]

                              INPUT DATA REPORT (2)

 Input File: TEST11.DAT
Report Date: 07/10/1991      Time: 22:33:36                     Page 3-2
========================================================================
Project: Example
Road: XXXX     From: XXXX      To: XXXX
Start Station:  100.000     End Station:  101.000
------------------------------------------------------------------------
LAYERS          MATERIAL          MODULI     LAYER   DRAINAGE  UNIT COST
                DESCRIPTION       (psi)      COEFF      COEFF    ($/CY)

Surface       1 ASPH CONC TY A   420000       0.40       1.00     51.61
              2 ASPH CONC TY B   430000       0.42       1.00     52.00
              3 ASPH CONC TY C   450000       0.44       1.00     54.00

Base          1 AGGR(TY A GR4)    30000       0.10       1.20     16.50
              2 AGGR(TY B GR4)    32000       0.11       1.20     19.40
              3 AGGR(TY PB G4)    34000       0.12       1.20     20.00

Subbase       1 AGGR TYPE E       11000       0.08       1.20      9.80
              2 AGGR TYPE F       13000       0.09       1.20     11.00
              3 AGGR TYPE G       14000       0.10       1.20     12.50

========================================================================
```

FIGURE 2 FPOD input data report (2).



```
======================================================================
AASHTO Flexible Pavement Optimal Design Program            CTR UT Austin
FPOD [v1.0 June 1991]

                             OPTIMAL SOLUTION REPORT

 Input File: TEST11.DAT
Report Date: 07/10/1991       Time: 22:33:36                   Page 3-3
======================================================================
Project: Example
Road: XXXX      From: XXXX      To: XXXX
Start Station:  100.000     End Station:  101.000
----------------------------------------------------------------------
                            1. NONINTEGER SOLUTION

NO          LAYERS       MATERIAL               THICKNESS       UNIT COST
                         DESCRIPTION            (inches)         ($/CY)

1         3 Surface      ASPH CONC TY C            9.11           54.00
2         1 Base         AGGR(TY A GR4)            6.00           16.50
3         2 Subbase      AGGR TYPE F              17.31           11.00

            Total Cost:     21.72 ($/SY)
----------------------------------------------------------------------
                            2. INTEGER SOLUTION

NO          LAYERS       MATERIAL               THICKNESS       UNIT COST
                         DESCRIPTION            (inches)         ($/CY)

1         3 Surface      ASPH CONC TY C            9.00           54.00
2         1 Base         AGGR(TY A GR4)            6.00           16.50
3         3 Subbase      AGGR TYPE G              16.00           12.50

            Total Cost:     21.80 ($/SY)
======================================================================
```

FIGURE 3 FPOD optimal solution report.

From this example, it can be seen that the layer materials selected from the model may not be the same in the two solutions. Therefore, the ratio of layer coefficient to unit cost mentioned before cannot be used to select materials for integer solution in some cases.

SENSITIVITY ANALYSIS

The differences between FPOD and other nonoptimization programs such as DNPS86 are (a) FPOD can select the best combination of the materials for a problem if more than one type of material is available for each of the three layers, and (b) FPOD takes unit costs of the three layers into account in the process of thickness design. In other words, the optimal solutions to the thicknesses should be sensitive to unit costs of the three layers. The major consideration is that optimal thicknesses change with the changes of unit costs in the model, so we focus on the sensitivity analysis of change of unit costs versus change of optimal thicknesses of the three layers and compare the FPOD solutions with DNPS86 solutions. In the sensitivity analysis, the default data (given in Figure 1) of AASHTO DNPS86 program is used by changing the moduli of elasticity, layer coefficients, and drainage coefficients of paving materials to average ones. Unit costs of paving materials for surface, base, and subbase courses range from \$40/yd^3 to \$60/yd^3, \$15/yd^3 to \$25/yd^3, and \$8/yd^3 to \$15/yd^3, respectively (as shown in Table 1).

Optimal Solutions Versus Unit Costs

Table 2 presents the sensitivity analysis results for the 8-in. minimum base and subbase thickness. In this case, the layer thicknesses of surface, base, and subbase courses obtained from the DNPS86 program are 9.97, 11.41, and 21.75 in., respectively. In Table 2, the first column lists the unit costs of the three layer materials, Column 2 lists the total costs of DNPS86 solutions, and Column 7 and Column 12 list the total cost changes of FPOD noninteger solutions and integer solutions as compared with DNPS86 solutions. Finally, Column 13 lists the total cost increase of integer solutions as compared with noninteger solutions. In Part 1 of Table 2, the unit costs of base and subbase courses are fixed to \$20/yd^3 and \$10/yd^3, respectively; the unit cost of surface course changes from \$40/yd^3 to \$60/yd^3. In Part 2 of Table 2, the unit costs of surface and subbase courses are fixed to \$50/yd^3 and \$10/yd^3, respectively; the unit cost of base course changes from \$15/yd^3 to \$25/yd^3. Finally, in Part 3 of Table 2, the unit costs of surface and base courses are fixed to \$50/yd^3 and \$20/yd^3, respectively; the unit cost of the subbase course changes from \$8/yd^3 to \$15/yd^3.

Table 2 shows that optimal noninteger and integer solutions change two or more times when the unit costs of the materials for any two layers are fixed and the unit cost of the material for another layer changes within the unit cost range specified. The thickness of the surface course decreases and the thicknesses of the base and subbase courses increase with the increase of unit cost of surface course. As a general rule, the degree of change depends on layer coefficients, drainage coefficients, and resilient moduli. The smaller the layer and drain-

TABLE 1 INPUT DATA FOR SENSITIVITY ANALYSIS

Layers	Elastic Modulus (psi)	Layer Coefficients	Drainage Coefficients	Range of Unit Costs ($/CY)
Surface	450000	0.35	1.00	40 - 60
Base	30000	0.12	1.00	15 - 25
Subbase	15000	0.08	1.00	8 - 15

TABLE 2 SENSITIVITY ANALYSIS (BASE AND SUBBASE ≥ 8 in.)

Unit Cost ($/CY)	DNPS86 Solutions Total Costs ($/SY)	FPOD Noninteger Solutions					FPOD Integer Solutions					
		Layer Thickness (in)			Total Cost ($/SY)	(6)-(2) / (2) %	Layer Thickness (in)			Total Cost ($/SY)	(11)-(2) / (2) %	(11)-(6) / (6) %
		Surface	Base	Subbase			Surface	Base	Subbase			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1. SURFACE COURSE COST CHANGES, BASE COURSE COST 20$/CY, SUBBASE COURSE COST 10$/CY												
40	23.46	14.28	8.00	8.00	22.53	-3.96	14	8	10	22.78	-2.90	+1.11
41	23.74	14.28	8.00	8.00	22.93	-3.41	12	8	18	23.11	-2.70	+0.78
42	24.01	14.28	8.00	8.00	23.33	-2.83	12	8	18	23.44	-2.37	+0.47
43	24.29	14.28	8.00	8.00	23.73	-2.31	12	8	18	23.77	-2.14	+0.16
44	24.57	11.14	8.00	21.75	24.10	-1.91	12	8	18	24.11	-1.87	+0.04
45	24.84	11.14	8.00	21.75	24.41	-1.73	12	8	18	24.44	-1.61	+0.12
46	25.12	11.14	8.00	21.75	24.72	-1.59	12	8	18	24.78	-1.35	+0.24
47	25.40	11.14	8.00	21.75	25.03	-1.46	12	8	18	25.11	-1.14	+0.32
48	25.67	11.14	8.00	21.75	25.34	-1.29	12	8	18	25.44	-0.90	+0.39
49	25.95	11.14	8.00	21.75	25.65	-1.16	12	8	18	25.78	-0.66	+0.47
50	26.23	11.14	8.00	21.75	25.96	-1.03	12	8	18	26.11	-0.46	+0.58
51	26.50	11.14	8.00	21.75	26.27	-0.87	11	9	21	26.42	-0.30	+0.57
52	26.78	11.14	8.00	21.75	26.58	-0.75	11	9	21	26.72	-0.22	+0.53
53	27.06	11.14	8.00	21.75	26.89	-0.63	11	9	21	27.02	-0.15	+0.48
54	27.34	11.14	8.00	21.75	27.19	-0.55	11	9	21	27.33	-0.04	+0.51
55	27.61	11.14	8.00	21.75	27.50	-0.40	11	9	21	27.64	+0.11	+050
56	27.89	11.14	8.00	21.75	27.81	-0.29	11	9	21	27.94	+0.18	+0.47
57	28.17	11.14	8.00	21.75	28.12	-0.18	11	9	21	28.25	+0.28	+0.46
58	28.44	11.14	8.00	21.75	28.43	-0.04	11	9	21	28.56	+0.42	+0.45
59	28.72	9.97	11.41	21.75	28.74	0	11	9	21	28.86	+0.49	+0.42
60	29.00	9.97	11.41	21.75	29.02	0	11	9	21	29.17	+0.59	+0.52
2. BASE COURSE COST CHANGES, SURFACE COURSE COST 50$/CY, SUBBASE COURSE COST 10$/CY												
15	24.64	9.97	20.58	8.00	24.64	0	10	20	9	24.73	+0.37	+0.37
16	24.96	9.97	11.41	21.75	24.96	0	10	12	21	25.06	+0.40	+0.40
17	25.28	9.97	11.41	21.75	25.28	0	11	9	21	25.36	+0.32	+0.32
18	25.59	11.14	8.00	21.75	25.51	-0.27	11	9	21	25.61	+0.12	+0.39
19	25.91	11.14	8.00	21.75	25.74	-0.66	11	9	21	25.86	-0.19	+0.47
20	26.23	11.14	8.00	21.75	25.96	-1.03	12	8	18	26.11	-0.46	+0.58
21	26.54	11.14	8.00	21.75	26.18	-1.36	12	8	18	26.33	-0.79	+0.57
22	26.86	11.14	8.00	21.75	26.40	-1.71	12	8	18	26.56	-1.12	+0.61
23	27.18	11.14	8.00	21.75	26.63	-2.02	12	8	18	26.78	-1.47	+0.56
24	27.50	11.14	8.00	21.75	26.84	-2.40	12	8	18	27.00	-1.82	+0.60
25	27.81	11.14	8.00	21.75	27.07	-2.67	12	8	18	27.22	-2.12	+0.55
3. SUBBASE COURSE COST CHANGES, SURFACE COURSE COST 50$/CY, BASE COURSE COST 20$/CY												
8	25.02	11.14	8.00	21.75	24.75	-1.08	11	9	21	24.95	-0.28	+0.81
9	25.62	11.14	8.00	21.75	25.35	-1.05	11	9	21	25.53	-0.35	+0.70
10	26.23	11.14	8.00	21.75	25.96	-1.03	12	8	18	26.11	-0.46	+0.58
11	26.83	11.14	8.00	21.75	26.58	-0.93	12	8	18	26.62	-0.78	+0.15
12	27.44	14.28	8.00	8.00	26.94	-1.82	12	8	18	27.11	-1.20	+0.63
13	28.04	14.28	8.00	8.00	27.17	-3.10	14	9	8	27.33	-2.53	+0.59
14	28.64	14.28	8.00	8.00	27.39	-4.36	14	9	8	27.56	-3.77	+0.62
15	29.25	14.28	8.00	8.00	27.61	-5.61	14	9	8	27.78	-5.03	+0.61

DNPS86 SOLUTION: Surface Course: 9.97 Inches
Base Course: 11.41 Inches
Subbase Course: 21.75 Inches

age coefficients and the larger the resilient moduli, the more the magnitude of change.

Similarly, the thickness of the base course decreases and the thicknesses of the surface and subbase courses increase with the increase of unit cost of base course.

With the increase of the unit cost of the subbase course, for noninteger solutions, the thickness of the base course remains the same; thicknesses of surface and subbase courses increase and decrease, respectively: for integer solutions, base course thickness decreases from \$9/yd^3, and then increases from \$13/yd^3; surface and subbase course thicknesses change in the same way they do for noninteger solutions.

Optimal Integer Versus Optimal Noninteger Solutions

Table 2 and Figures 4 through 6 show that integer solutions always cost more than noninteger solutions. In this example, the total costs of an integer solution increase up to 1.3 percent compared with a noninteger solution (Figure 6, minimum base and subbase thickness of 10 in.). As a rule, the thicker the minimum thicknesses of base and subbase courses specified, the larger the difference between the two types of solution.

Optimal Versus Nonoptimal Solutions

Figures 7 through 9 show the percentage reduction of total costs of optimal noninteger solutions given by FPOD compared with nonoptimal solutions given by DNPS86. Within a certain range of unit costs, DNPS86 and FPOD get the same solutions; in other words, DNPS86 can also give optimal solutions sometimes. For example, if the unit cost of the surface course is equal to or above \$58/yd^3 (Table 2 and Figure 7), or the unit cost of the base course is between \$15/yd^3 and \$17/yd^3 (Table 2 and Figure 8), both DNPS86 and FPOD give the same solutions, regardless of the minimum thicknesses of base and subbase courses specified. That means DNPS86 so-

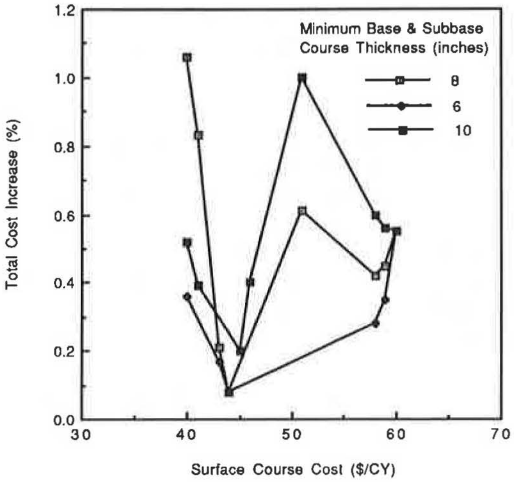


FIGURE 4 Surface course cost versus total cost increase, comparison of integer solutions with noninteger solutions (base course cost, \$20/yd^3; subbase course cost, \$10/yd^3).

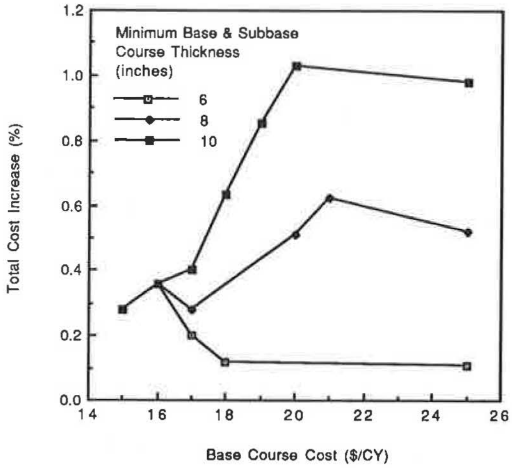


FIGURE 5 Base course cost versus total cost increase, comparison of integer solutions with noninteger solutions (surface course cost, \$50/yd^3; subbase course cost, \$10/yd^3).

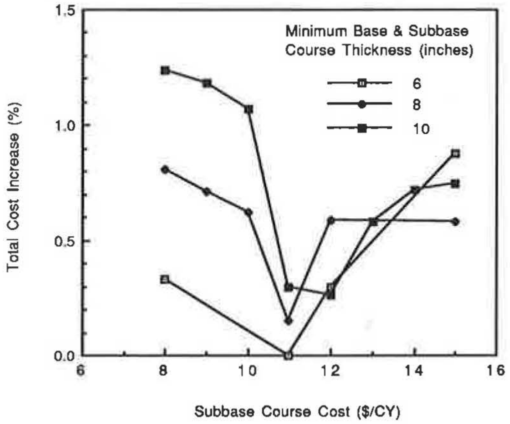


FIGURE 6 Subbase course cost versus total cost increase, comparison of integer solutions with noninteger solutions (surface course cost, \$50/yd^3; base course cost, \$20/yd^3).

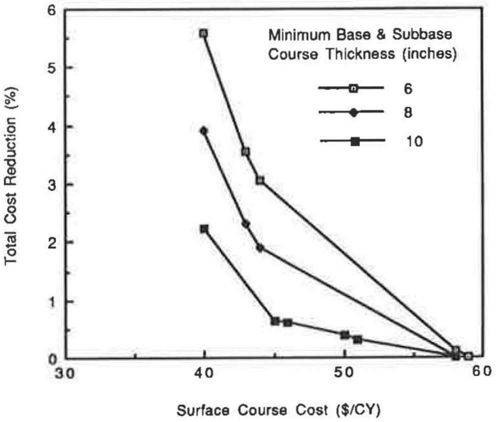


FIGURE 7 Surface course cost versus total cost reduction, comparison of FPOD noninteger solutions with DNPS86 solutions (base course cost, \$20/yd^3; subbase course cost, \$10/yd^3).

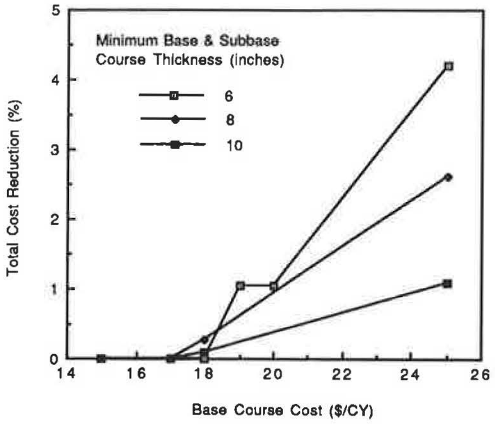


FIGURE 8 Base course cost versus total cost reduction, comparison of FPOD noninteger solutions with DNPS86 solutions (surface course cost, \$50/yd³; subbase course cost, \$10/yd³).

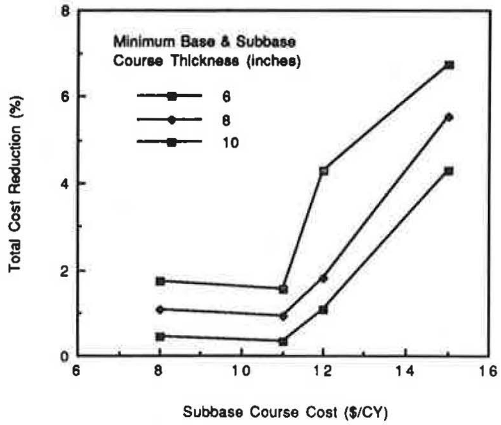


FIGURE 9 Subbase course cost versus total cost reduction, comparison of FPOD noninteger solutions with DNPS86 solutions (surface course cost, \$50/yd³; base course cost, \$20/yd³).

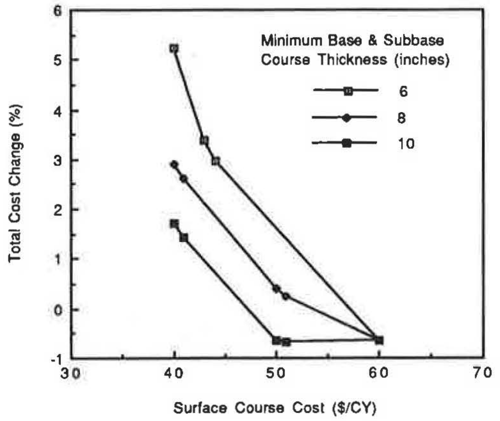


FIGURE 10 Surface course cost versus total cost change, comparison of FPOD integer solutions with DNPS86 solutions (base course cost, \$20/yd³; subbase course cost, \$10/yd³).

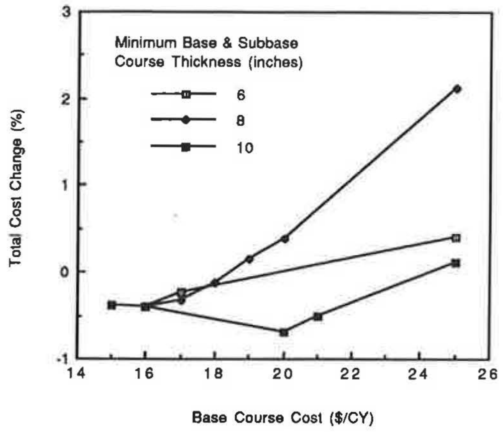


FIGURE 11 Base course cost versus total cost change, comparison of FPOD integer solutions with DNPS86 solutions (surface course cost, \$50/yd³; subbase course cost, \$10/yd³).

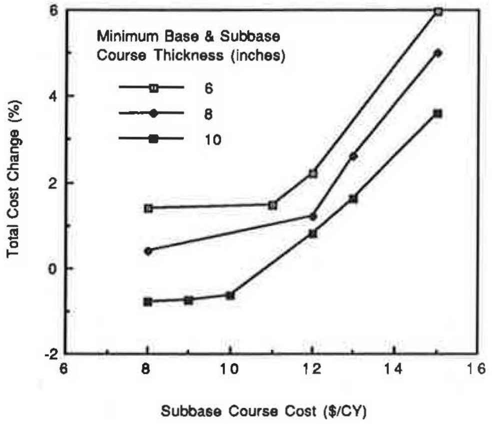


FIGURE 12 Subbase course cost versus total cost change, comparison of FPOD integer solutions with DNPS86 solutions (surface course cost, \$50/yd³; base course cost, \$20/yd³).

lutions are also optimal for those cases. The savings realized by using FPOD is determined by the minimum thicknesses of the base and subbase courses. Generally speaking, the thinner the base course, the larger the savings will be. Figure 7 shows the reduction of total costs can be as large as 5.8 percent for cases in which the minimum thickness is 6 in. If no base course is allowed, the largest saving can be obtained in some cases.

For noninteger solutions, at least one of the optimal thicknesses of base and subbase is the minimum value specified in most cases, but this may not be true for integer solutions.

Figures 10 through 12 show the percentage change of total costs of optimal integer solutions given by FPOD as compared with nonoptimal solutions given by DNPS86. In some cases, the total costs are more than those of the nonoptimal noninteger solutions.

SUMMARY AND CONCLUSIONS

A mixed-integer programming model was formulated for flexible pavement design problems, and an FPOD computer program was developed accordingly. The FPOD program can give the best combination of various paving materials for all three layers and at the minimum-cost thicknesses. It searches for the minimum-cost solution when the costs of the paving materials change. This capability is desirable when the minimum-cost solution is required (the DNPS86 program gives only one solution, regardless of the costs of the paving materials). FPOD gives two types of solution: noninteger and integer. In terms of the *AASHTO Guide for Design of Pavement Structures* and from a practical point of view, an integer solution should be used, but it costs more than a noninteger solution. The additional cost of using integer solutions depends on the minimum thickness of the base and subbase courses and differs from problem to problem. It is recommended that the decision to select one of the solutions be made in terms of the cost ratio of the two solutions and construction experience.

The minimum-cost solutions for flexible pavements very much depend on the minimum thicknesses of the base and subbase courses set by users. As mentioned in the AASHTO guide, the minimum thickness of all the three layers depends somewhat on local practice and conditions.

The present version of FPOD is used only in the design of new flexible pavement; life-cycle analysis has not been taken into account.

ACKNOWLEDGMENT

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Automated Pavement Subsurface Profiling Using Radar: Case Studies of Four Experimental Field Sites

KENNETH R. MASER AND TOM SCULLION

Accurate knowledge of pavement layer thicknesses and material properties is important to pavement management. Often this information is unknown or records are inaccurate, inaccessible, or out of date. The traditional method for obtaining pavement layer data is core sampling, which is time-consuming, labor-intensive, and intrusive to traffic; it also provides information only at the core location. The capability of ground-penetrating radar to provide accurate and continuous pavement layer thickness and property information has been investigated. Four Texas Strategic Highway Research Program asphalt pavement test sites were tested with radar. The accuracy of the radar predictions for asphalt thickness was within ±0.32 in. using the radar data alone, and within ±0.11 in. when one calibration core was used per site. The accuracy of the radar predictions for base thickness was within ±0.99 in. The nominal layer thickness ranged from 1 to 8 in. of asphalt and 6 to 10 in. of base. The actual asphalt layer thickness was shown to vary by more than 20 percent from values assumed from prior records and earlier cores. These variations have been shown to lead to errors of up to 95 percent in base moduli back-calculated from falling weight deflectometer data. The radar results were shown to be repeatable over time and independent of survey speed at up to 40 mph. The radar data were analyzed automatically using software that operated directly on the raw radar waveforms and produced numerical layer thickness profiles. The resulting predictions were correlated with direct in situ measurements and core and material samples. The results of this project have shown that ground-penetrating radar data, when properly analyzed, can provide highly accurate measurements of pavement layer properties for project- and network-level applications.

Pavement layer thickness data are important in many aspects of pavement engineering and management. Mechanistic models for pavement performance, and structural tests that use these models for back calculation, require pavement layer thicknesses as input. Pavement thickness measurements are required for quality control of new construction or overlays and for designing mill and recycle projects. The layer thicknesses represent an important element of a pavement management system (PMS) data base; they are needed for load rating, overlay design, and setting maintenance and rehabilitation priorities. Many state highway agencies have layer thickness records that are inaccurate or difficult to access and use.

Traditionally, core samples have provided the only means for accurately evaluating pavement layer thickness. However, sampling is time-consuming and intrusive to traffic. Depending on the spacing of cores, there is always uncertainty about thickness variations between them. For network-level pavement inventories, cores are an impractical and inadequate means for characterizing pavement thickness.

The objective of study reported in this paper was to demonstrate the accuracy, reliability, and practicality of using ground-penetrating radar (GPR) for continuous measurement of pavement layer properties. GPR's capability in this application has been suggested in several research and experimental studies (*1–3*). In fact, ASTM D4748-87 specifies for the measurement of pavement thickness with radar. In these applications, however, the radar data analysis is qualitative and manual. There has not been a systematic investigation comparing predicted to actual thickness for a range of conditions.

Recent studies (*4,5*) have demonstrated the feasibility of accurately predicting the thickness of asphalt overlays on concrete bridge decks. Investigators have used automated signal processing techniques to obtain quantitative results for asphalt thickness. The specific objective of the work presented herein has been to use these automated techniques in the context of a systematic study to determine the accuracy of radar thickness predictions.

Four sites were chosen for investigation, each representing different layer dimensions and material properties. Quantitative methods for determining thickness and moisture content were applied automatically to the radar data, and continuous output of thickness and moisture content was obtained. This output was compared with the results from direct measurements using cores, material samples, and a penetrometer. The repeatability of the measurement and the effects of radar vehicle speed were also studied.

PRINCIPLES OF GROUND-PENETRATING RADAR

Ground-penetrating radar operates by transmitting short pulses of electromagnetic energy into the pavement using an antenna attached to a survey vehicle (see Figures 1 and 2). These pulses are reflected back to the antenna; the arrival time and amplitude are related to the location and nature of dielectric discontinuities in the material (air-asphalt or asphalt-base, etc.). The reflected energy is captured and may be displayed on an oscilloscope to form a series of pulses that are referred to as the "radar waveform." The waveform contains a record of the properties and thicknesses of the layers within the pavement. Figure 3 shows a typical set of pavement waveforms collected during this project.

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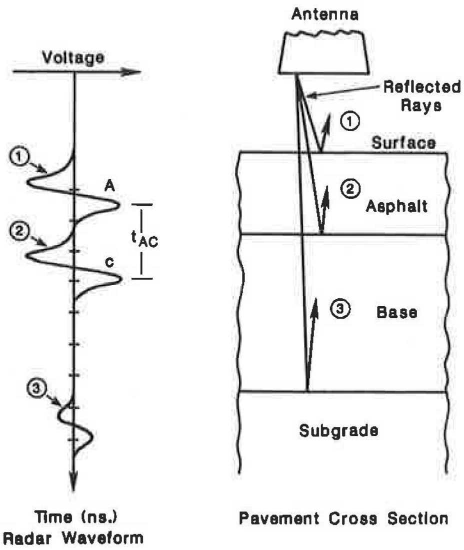

FIGURE 1 Radar pavement model.

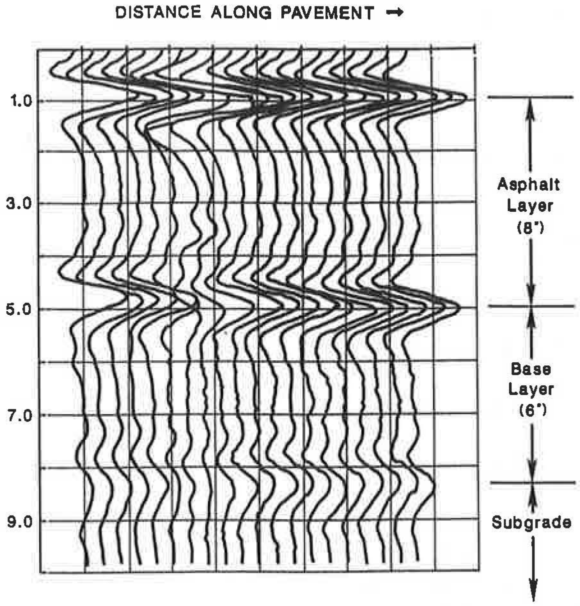

FIGURE 3 Radar pavement data (SH-30, Huntsville, Texas).

The pavement layer thicknesses and properties may be calculated using the amplitude and arrival times of the waveform peaks corresponding to reflections from the interfaces between the layers (see Figure 3). One may calculate the dielectric constant of a pavement layer relative to the previous layer by measuring the amplitude of the waveform peaks corresponding to reflections from the interfaces between the layers. The travel time of the transmit pulse within a layer in conjunction with its dielectric constant determines the layer thickness, as follows:

$$\text{thickness} = \text{velocity} \times \left(\frac{\text{time}}{2}\right) \tag{1}$$

Because the measured time between peaks represents the round-trip travel of the radar pulse, the thickness computation is based on time divided by 2. The radar velocity can be computed from the dielectric constant of the medium, ε, as

$$\text{velocity} = \frac{11.8}{\sqrt{\varepsilon}} \left(\frac{\text{inches}}{\text{nanosecond}}\right) \tag{2}$$

where 11.8 is the radar velocity in free space in inches per nanosecond. Combining Equations 1 and 2, one obtains

$$\text{thickness} = \frac{5.9 \times \text{time}}{\sqrt{\varepsilon}} \tag{3}$$

where time is measured in nanoseconds and thickness, in inches.

FIGURE 2 Radar van.

The radar pulse has a finite width, so the layers must be thick enough for the reflections from each layer to appear without overlap from the surrounding layer. This minimum thickness can be calculated from the radar pulse width (in nanoseconds) and the radar velocity in the medium. For the 1-GHz horn antennas commonly used for this application, this thickness is approximately 2.5 in. in asphalt. Ground-coupled dipole antennas such as those used for geotechnical applications have transmit pulses two to three times longer, and their resolution is limited to much thicker layers.

For thicknesses less than this minimum resolution, a numerical procedure called deconvolution is required. This procedure decomposes overlapping reflections into their individual components and thus allows for thickness determination. Deconvolution analysis carried as part of this project on preliminary field data collected at the Texas Transportation Institute (TTI) annex showed that layer thicknesses as low as 1 in. could be predicted accurately.

The computation of thickness using Equation 1 presumes that the layer in consideration is homogeneous and that its dielectric constant is known. Computation of the surface layer dielectric constant can be made by measuring the ratio of the radar reflection from the asphalt to the radar amplitude incident on the pavement. This ratio, called the reflection coefficient, can be expressed as follows:

$$\text{reflection coefficient } (1-2) = \frac{\sqrt{\varepsilon_1} - \sqrt{\varepsilon_2}}{\sqrt{\varepsilon_1} + \sqrt{\varepsilon_2}} \qquad (4)$$

where the subscripts 1 and 2 refer to the successive layers. The incident amplitude on the pavement can be determined by measuring the reflection from a metal plate on the pavement surface, because the metal plate reflects 100 percent. Using these data, rearranging Equation 4, and noting that the dielectric constant of air is 1, one obtains the asphalt dielectric constant, ε_a, as follows:

$$\varepsilon_a = \left[\frac{A_{pl} + A}{A_{pl} - A}\right]^2 \qquad (5)$$

where A is the amplitude of reflection from asphalt and A_{pl} is the amplitude of reflection from metal plate (negative of incident amplitude). A similar analysis can be used to compute the dielectric constant, ε_b, of the base material. The resulting relationship is

$$\varepsilon_b = \varepsilon_a \left[\frac{(F - R2)}{(F + R2)}\right]^2 \qquad (6)$$

where

$F = \dfrac{4\sqrt{\varepsilon_a}}{1 - \varepsilon_a}$ and

$R2$ = ratio of reflected amplitude from the top of the base layer to the reflected amplitude from the top of the asphalt (*5*).

Note that these analyses make two important assumptions: (a) the layers are homogeneous, and (b) the layers are nonconductive. The first assumption is violated when the layers within the asphalt are not uniform, such as may occur because of overlays or differences in properties of successive lifts of the initial pavement. When these layers are not uniform, intermediate reflections will occur within the asphalt and the use of Equation 3 for the entire asphalt layer will be incorrect. This error can be corrected by recognizing the layering within the asphalt and incorporating this layering into the pavement model.

The second assumption is generally true for asphalt but less so for the base materials. The presence of moisture, salts, and clays produces losses that make Equation 4 less valid. Therefore, one can conclude that asphalt thickness can be accurately measured directly from the radar data if layering is taken into account. On the other hand, the absolute measurement of base properties might be subject to error unless conductivity is taken into account.

The moisture content of the base is determined from its dielectric constant using a common mixture law called the complex refractive index model (*6*), which is expressed as

$$\sqrt{\varepsilon_m} = \sum V_i \sqrt{\varepsilon_i} \qquad (7)$$

where

ε_m = relative dielectric constant of the mixture,
v_i = volume fraction of Component *i*, and
ε_i = relative dielectric constant of Component *i*.

The components of the base material are solid particles, water, and air. The dielectric constants of water and air can be taken as 81 and 1, respectively.

To determine moisture content from this model, one must assume the bulk density of the material and the dielectric constant of the solids. Once these assumptions are made, the moisture content (percent by total weight) can be computed from Equations 5 and 7, making various substitutions for porosity and percent saturation in terms of bulk density, to obtain the following:

$$\text{moisture content} = \frac{\sqrt{\varepsilon_b} - 1 - \frac{\gamma_d}{\gamma_s}(\sqrt{\varepsilon_s} - 1)}{\sqrt{\varepsilon_b} - 1 - \frac{\gamma_d}{\gamma_s}(\sqrt{\varepsilon_s} - 22.2)} \qquad (8)$$

where

ε_b = base dielectric constant (determined from Equation 6),
ε_s = solids dielectric constant (varies from 4 to 8 depending on source material),
γ_d = dry density (pounds per cubic foot), and
γ_s = density of solids (~165 pcf).

These equations serve as the basis for analysis of the data collected during this study.

DESIGN AND CONDUCT OF TEST PROGRAM

A program was designed to collect radar data on in-service pavements and to correlate the predictions from the radar data with direct measurement. Four Strategic Highway Research Program (SHRP) General Pavement Studies (GPS) sites were selected for evaluation, as described in Table 1. The sites were asphalt pavement, because this is the type of pavement for which thickness is the greatest unknown.

TABLE 1 PAVEMENT PROPERTIES FROM INVENTORY DATA

Site	Asphalt Thickness (in.)		Type	Base Thickness (inches)	Dry Density (pcf)
	Top Course	Bottom Course			
SH 30	1.0	7.0	Bituminous treated soil	6.0	115
SH 19	1.0	6.0	Lime-treated fine-grained soil	6.0	---
SH 105	1.0	none	crushed stone	10.0	133
SH 21	2.0	6.0	crushed stone	10.0	131

Each test section was 1,500 ft long: 500 ft preceding the GPS site, 500 ft of the site itself, and 500 ft beyond the site. It was understood that verification sampling could take place only in the first and last 500-ft sections, because the GPS site could not be disturbed.

Radar data was collected by Infrasense, Inc. (Cambridge, Massachusetts) using a van-mounted horn antenna system provided and operated by Pulse Radar, Inc., of Houston, Texas. Data were collected on June 26 and 27, 1990, and taken back to Infrasense for analysis. On the basis of the analysis, areas within each site were identified for direct sampling. The sites were revisited on July 26 and 27, 1990, for repeat radar measurements in the identified areas and for extraction of direct samples at the selected sampling sites. Extraction of direct samples was carried out jointly by TTI and the Texas Department of Transportation (TexDOT).

Radar equipment setup included a number of calibration tests, including an antenna end reflection test, a metal plate reflection test, and a time calibration test. Traffic control was set up by TexDOT to allow for test speeds ranging from 5 to 40 mph. A 4-ft-wide strip of aluminum foil was taped transversely across the test lane at the beginning of the 1,500-ft test section to provide a start marker within the radar data.

Initial data collection (June 26 and 27) at each site involved four radar passes—one at low speed (5 mph) on the left wheelpath, and one each at 5, 15, and 40 mph in the right wheelpath. Data were collected continuously over the 1,500-ft test.

All radar data were continuously digitized and stored to hard disk using a Compaq 386 computer housed in the van. The radar data were subsequently analyzed using the PAVLAYER software developed by Infrasense. This software automates the application of Equations 1 through 8 to the raw radar data as shown in Figure 3. The results in this paper are based on this analysis.

Locations for ground truth were determined after a preliminary analysis of the radar data. This analysis revealed locations and areas where significant variations in thickness and dielectric constant occurred. The sample sites were located so that a reasonable range of values could be obtained at each. Ground truth data were also available from field data collected previously as part of the SHRP.

Three types of tests were carried out: (a) 4-in.-diameter wet-core samples to determine asphalt layer thickness, (b) 6-in.-diameter dry cores to obtain samples for base moisture content, and (c) penetrometer tests to determine base thickness. TTI conducted the wet-core and penetrometer testing and collected the samples and conducted the moisture content tests on samples obtained using the TexDOT dry-core rig. Under certain conditions when the penetrometer progress was slow [e.g., State Highway (SH) 21 and SH-105], attempts were made to determine base thickness visually in the dry-core holes, with occasional success.

DESCRIPTION OF DATA AND RESULTS

The data analysis was carried out using Equations 1 through 6. Asphalt pavement thickness is calculated by (a) determining the radar velocity in the asphalt using the asphalt dielectric constant determined from the surface reflection using Equation 5, and (b) computing the thickness from the velocity and the arrival time of the reflection from the bottom of the asphalt using Equation 4. The base layer thickness was calculated in a similar fashion, except the radar velocity in the base material was determined from the base material dielectric constant computed from the magnitude of the reflection at the asphalt-base interface using Equation 6. The base moisture constant was computed from the base dielectric constant using Equation 8. All of these calculations are completely automated in PAVLAYER so that continuous thickness and moisture profiles with hundreds of waveforms can be computed in a few minutes on a 386 machine.

Typical asphalt thickness, base thickness, and moisture content profiles obtained from the radar data collected during this study are shown in Figure 4. The following sections present and discuss comparisons of these predictions with traditional direct measurements.

Asphalt Layer Thickness

Table 2 shows the thickness data predicted from the radar analysis versus the thicknesses measured from core samples for three of the four sites. Two types of radar predictions are presented in the two columns of the table. The column labeled "radar alone" represents predictions using Equations 3 and 5 without benefit of any core data. The column labeled "core calibration" represents an adjustment of the "radar alone" values on the basis of a calibration of the asphalt dielectric constant using the first core at each site.

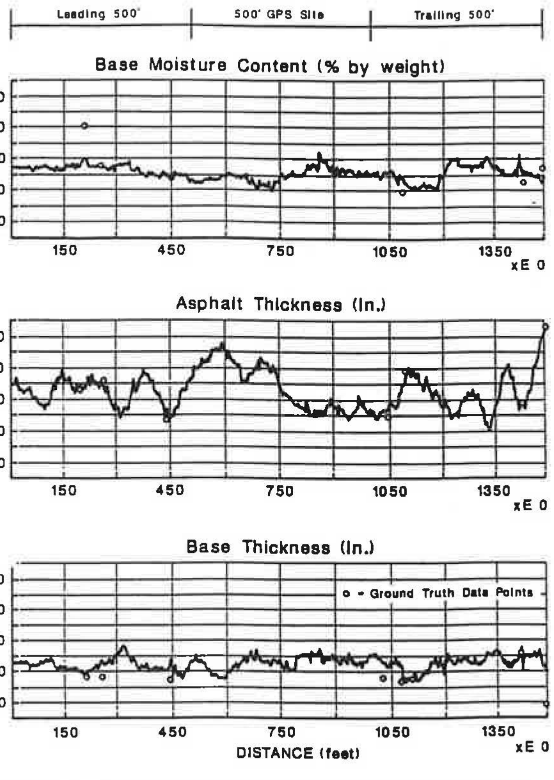

FIGURE 4 Typical data, 5-ft intervals (from GPR data, SH-30).

TABLE 2 PREDICTED VERSUS MEASURED ASPHALT THICKNESS

Site/ Location (ft)	Predicted Asphalt Thickness (in)		Measured Asphalt Thickness (in)
	(radar alone)	(core calibration)	
SH 30-			
210	7.8	7.8	7.8
250	8.0	8.0	8.1
445	6.8	6.8	6.7*
450	6.9	6.9	6.7*
455	6.9	6.9	6.8*
460	6.9	6.9	6.8*
1040	7.2	7.2	7.0*
1062	7.3	7.3	7.0*
1067	7.2	7.2	7.2*
1072	7.3	7.3	7.1*
1105	8.4	7.0	8.5
1441	7.0	7.0	7.4
1495	9.5	9.5	9.8
SH 19-			
25	6.5	6.1	6.1
61	6.6	6.2	6.3
445	6.6	6.2	6.2*
450	6.6	6.2	6.2*
455	6.4	6.0	6.4*
460	6.4	6.0	6.2*
1011	6.9	6.5	6.5
1040	6.4	6.0	6.1*
1062	6.6	6.2	6.2*
1067	6.8	6.4	6.5*
1072	6.9	6.4	6.4*
1078	7.3	6.8	6.8
1150	7.1	6.6	6.8
1193	6.6	6.2	6.3
SH 105-			
5	2.5	2.3	2.3
165	2.5	2.3	1.9
203	2.1	1.9	1.5
255	2.8	2.6	2.0
445	1.9	1.7	1.9*
450	1.9	1.7	1.9*
455	1.9	1.7	1.8*
460	2.0	1.8	1.9*
1040	2.1	1.9	1.8*
1060	2.1	1.9	1.6*
1185	1.6	1.5	1.6

*These values were taken from SHRP field reports.

The thickness data for the fourth site, SH-21, are presented in Table 3. The data from this site revealed two distinct layers of asphalt, the second layer having a higher dielectric constant than the first. Table 3 presents three types of radar prediction: (a) a prediction that ignores this layer information (no calibration), (b) a prediction that considers this layering in the radar analysis (internal calibration), and (c) a prediction that calibrates the asphalt dielectric constant using one core (core calibration).

Tables 2 and 3 present predicted versus measured asphalt thickness for 50 locations on the four pavement sections. To assess the accuracy of the prediction, a linear regression was carried out between predicted and measured values. Two analyses were conducted: one in which the predicted values were based on the best radar data without benefit of core calibration (i.e., middle column of Table 3 for SH-21), and one in which the predicted values incorporated the use of one calibration core per site. The results are as follows:

TABLE 3 PREDICTED VERSUS MEASURED ASPHALT THICKNESS (SH-21)

Site/ Location (feet)	Thickness Predictions (in.)			Measured Thickness from core (in)
	no calib.	internal calib.	core calib.	
SH 21-				
27	8.8	8.2	8.0	8.0
105	9.3	8.7	8.5	8.5
293	9.9	9.3	9.0	9.0
445	9.9	9.3	9.0	8.2*
450	9.8	9.2	9.0	8.5*
455	9.4	8.8	8.6	8.8*
460	9.1	8.5	8.2	9.0*
1035	10.0	9.1	9.1	8.5
1040	9.4	8.8	8.5	8.1*
1084	9.3	8.6	8.4	8.4
1114	9.6	8.9	8.7	8.0
1146	9.2	8.5	8.3	8.1

*These values were taken from SHRP field reports.

$$(T_a)_{\text{measured}} = K1 + K2(T_a)_{\text{predicted}} + \text{random error} \qquad (9)$$

where

$(T_a)_{\text{measured}}$ = asphalt thickness measured directly,
$(T_a)_{\text{predicted}}$ = asphalt thickness computed from radar, and
$K1$ and $K2$ = regression constants.

The regression fit yields the following result ($N = 50$ observations):

Parameter	*Radar Alone*	*Core Calibration*
$K1$	−0.25 in.	−0.012 in.
$K2$	0.998	0.994
R^2	0.98	0.99
Standard error	0.32 in	0.11 in.

The results of this regression indicates that there is an excellent one-to-one relationship between radar prediction and actual thickness ($R^2 = 0.98$ and 0.99) for both cases. These results also indicate that there is a small (0.25 in.) tendency to overpredict the asphalt thickness with radar measurements alone, a tendency that is corrected when the calibrating core is used. This error is probably due to the increasing asphalt dielectric constant with depth, which is not considered in the radar analysis. In terms of accuracy, the results show a potential predictive accuracy of ±0.32 in. with radar alone and of ±0.11 in. with the use of calibrating cores.

The radar-based asphalt thickness data as validated with coring demonstrate that significant variation in layer thickness can occur in short distances such as shown on SH-30. The surfacing thickness reported as 8 in. was in fact measured to vary from 7.0 to 9.5 in. (−12.5 to +15 percent). In fact, SHRP researchers will use a 7.0-in. thickness value, as determined from their cores, to interpret falling weight deflectometer (FWD) tests and to model the performance of the sections. As can be seen in Figure 4, this assumption is substantially in error (up to 2.5 in.) for most of the GPS section. Sample back calculations show that a +2.5-in. error on a pavement assumed to be 7 in. thick produces a 95 percent error in the back-calculated base modulus (*7*).

Base Thickness Predictions

Predicted versus measured base thickness values were correlated for 42 locations on the four pavement sections. The base thickness predictions for the SH-21 site were made using the two-layer asphalt model used for asphalt thickness predictions. To assess the accuracy of the predictions, a linear regression was carried out between predicted and measured values.

$$(T_b)_{\text{measured}} = K1 + K2(T_b)_{\text{predicted}} + \text{random error} \qquad (10)$$

where $(T_b)_{\text{measured}}$ is the base thickness measured directly and $(T_b)_{\text{predicted}}$ is the base thickness computed from radar.

The regression fit yields the following results:

$K1 = 2.47$ in.

$K2 = 0.63$

$R^2 = 0.72$

Standard error = 0.99 inches

Number of observations = 42

These results indicate more scatter (lower R^2) than that observed in the asphalt thickness predictions. The accuracy, as measured by the standard error, is not as good as the asphalt thickness measurements.

Factors that explain the lower accuracy and greater scatter of the base thickness predictions are

- Small errors that occur in the determination of the asphalt dielectric constant have a much greater effect on the computation of the base dielectric constant (see Equation 6) and on the resulting base thickness prediction.
- Geometric attenuation (loss of energy due to spreading of the radar beam) and depth variations in base material properties have not been considered in the analytic model.
- Base thickness ground truth methods are themselves imprecise. For example, thickness determination from cone penetrometer data is based on the interpretation of a 1- to 2-in. transition zone that appears between the base and the subgrade.

Base Moisture Content Predictions

Equation 8 was used for calculating moisture content by using one moisture content sample at each site to estimate a dry density and solids dielectric constant. These estimates were treated as constants for the site in the computation of moisture content at other locations. Using this method, the root-mean-square deviation between predicted and measured moisture content at 21 locations was 1.9 percent by weight.

An alternative application of radar to the measurement of base moisture variations is in looking at moisture content changes over time. The repeat survey carried out as part of this program was used to experiment with this concept. For most of the sites, the moisture content computations were identical for each of the two surveys. For one site, however, a significant change in moisture content occurred over a 100-ft length of the site. This result is shown in Figure 5. This result clearly shows that there is a localized pavement section whose base properties have changed during the month between the two surveys.

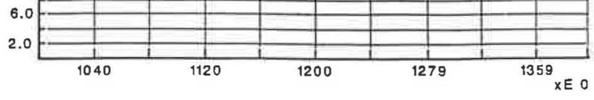

FIGURE 5 Detection of change in base moisture content, percent by weight (from GPR data, SH-30).

Effect of Survey Speed

Surveys at each site were conducted at three speeds: 5 mph, 15 mph, and 40 mph. The objective was to evaluate the sensitivity of the radar prediction to vehicle travel speed. In principal, vehicle speed should affect only the density of the collected data. On the basis of the data rate of the radar system, the three speeds would generate data at distance intervals ranging from 1 to 3 ft. To test for the presence of any other speed effects, the data collected at each of the driving speeds were analyzed at 5-ft intervals and compared. The comparison showed identical results (*8*).

CONCLUSIONS

The results of this effort have provided quantitative confirmation of the accuracy and repeatability of ground-penetrating radar for predicting asphalt and base layer thicknesses in pavement. The accuracy, as represented by regression fits of 50 and 42 data points, respectively, shows standard errors of 0.32 in. for asphalt layer thickness, 0.11 in. for asphalt thickness when one calibration core per site is used, and 0.99 in. for base layer thickness. Asphalt thicknesses ranging from 1 to 10 in. were measured with radar.

These results can be achieved using short-pulse horn antenna equipment in conjunction with a radar analysis model that incorporates the properties of the asphalt and base layers. The radar model must also account for the overlap of reflected pulses that occurs with asphalt fewer than 2.5 in. thick.

The results show that the radar predictions using these methods are repeatable and that the radar survey speed can be up to 40 mph without any effect on the results.

The radar and direct measurement results, as described herein, clearly illustrate the presence of otherwise unpredictable variations in pavement layer thickness. These variations were shown to be as high as 2.5 in. over a 40-ft distance. Such variability can produce large errors in prediction of layer moduli using FWD and similar tests and can lead to incorrect pavement assessment and overlay design. This variability and its consequences will also have a significant effect on the validity of the pavement performance prediction models to be produced by SHRP.

The results also suggest that changes in base moisture content over time can be clearly revealed by repeated radar surveys. Measurement of spatial variation of moisture content is also possible, if the composition and dry density of the base material is relatively uniform.

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