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Contents

Foreword

The 16 peer-reviewed papers in this volume were presented at the 1993 Annual Meeting of the Transportation Research Board in two sessions entitled "Pavement Management at the State and Local Levels" and sponsored by the TRB Committee on Pavement Management Systems. Irrgang and Maze report on a survey to assess the state of pavement management as practiced by state highway agencies. Lee et al. summarize the development of simplified predictive performance models for pavement management purposes. Using a dynamic programming formulation and a mechanistic pavement performance model, Chua et al. present a dynamic decision model for pavement management.

Grivas et al. characterize pavement conditions, specify treatment options for those conditions, and apply a linear programming technique to formalize an optimization methodology for program planning and budget allocation in network-level pavement management. In a comparison of optimization techniques and ranking techniques, Sharaf shows that the former are superior when applied to maintenance management systems. Sekiguchi et al. discuss the combined use of ground penetrating radar and a borehole camera to obtain more accurate thickness determinations for pavement management systems. On the basis of two methods of thinking (event and systemic), Novak and Kuo categorize analysis methods for pavement management systems as reactive or generative. Smith et al. describe the development of a pavement management system for the Delaware Department of Transportation, designated the Pavement Management and Planning program.

Hallenbeck presents preliminary results of an analysis of the seasonal volume patterns for different vehicle classes in the state of Washington. Wang et al. review important aspects of the network optimization system used by the Arizona Department of Transportation and discuss new implemented improvements. Grivas, Schultz, and Tanner document a case study for the New York State Thruway Authority employing a methodology to develop and use condition measures for pavement network characterization. Harper and Majidzadeh describe an integrated pavement and bridge management system that optimizes the allocation of scarce resources to minimize costs or maximize benefits. Collura et al. present the results of a survey to estimate the service life and costs of typical maintenance techniques and develop performance curves. Zhang et al. use fuzzy sets concepts to develop an index model called the overall acceptability index for flexible pavements. Chen et al. describe a graphical urban roadway management system to assist in scheduling maintenance and rehabilitation projects at the network level. Grivas and Schultz present a condition-based methodology for developing preliminary treatment recommendations for pavement projects, efficiently using available pavement management data.

Status of Pavement Management Systems and Data Analysis Models at State Highway Agencies

FEDERICO C. IRRGANG AND T. H. MAZE

The state of pavement management as practiced by state highway agencies (SHAs) is explored. A survey was conducted of each SHA to determine the stated and implicit objectives of its statewide pavement management system; if the SHA uses a ranking system for priority ranking and selecting projects, what are the mechanics and variables of the ranking system? If the SHA uses an optimization methodology for selecting projects, what is the methodology, what constraints are used, and what is the objective function? At the time of the survey (fall 1991), about a third of the SHAs had developed and were operating a pavement management system that includes network optimization. Sophisticated pavement management systems apply a mixture of the sciences of pavement design, highway maintenance/rehabilitation, and systems analysis. Clearly, SHAs understand the conventional sciences of pavement design and highway maintenance and rehabilitation. However, SHAs are less familiar with system analysis and the science of pavement management systems. As a result, the promotion of the science of pavement management is recommended, as is the development of standard terminology, standard data collection procedures, and structured analysis methodologies. In general, the same is true for the promotion of the science of maintenance management of all types of public infrastructure.

In March 1989 FHWA set a policy requiring that each state highway agency (SHA) have a pavement management system (PMS) (*1*). Each PMS must be based on concepts described in the AASHTO publication *Guidelines on Pavement Management*: "A PMS is a systematic approach to providing highway administrators and engineers with the types of information needed to effectively and efficiently manage their highway pavements" (*2*).

The FHWA policy states that SHAs were to have a PMS operational by January 13, 1993. This paper reports the findings from a survey of SHAs and evaluates the current status (the survey was completed during the fall 1991) of PMS implementation as SHAs work to meet the 1993 deadline.

Besides evaluating the state of the practice, the paper also provides a benchmark for the maturing science of pavement management as practiced by state agencies. As more agencies practice and learn about pavement management, the science of pavement management will be applied more extensively and improved. Structuring a science for pavement management implies developing standard terminology, standard data collection procedures, and structured analysis methodologies.

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Most simply, PMSs can be structured into three components:

- A data base containing information on the pavement inventory; pavement condition data; construction, maintenance, and reconstruction history; traffic data; maintenance, rehabilitation, and reconstruction (MR&R) cost data; and possibly other data (e.g., accident data).
- A data analysis package that uses information in the data base to allocate resources to potential MR&R projects. The data analysis systems used by SHAs vary in sophistication from structured engineering judgment to mathematical programming coupled with statistically based pavement condition forecasts.
- A feedback process to verify and improve the reliability of the PMSs.

Drawing from questionnaires returned by SHAs, the paper explores the state of the practice of the pavement management. Besides determining the progress SHAs are making toward implementing PMSs, the paper identifies the objectives used by SHAs for maintenance resource allocation and identifies the processes used within the data analysis components to allocate resources (usually a ranking system or an optimization model).

METHODOLOGY

Fifty-two questionnaires, each containing four open-ended questions, were mailed to pavement management engineers at the 50 SHAs and the highway agencies in Washington, D.C., and Puerto Rico. Thirty-nine agencies returned the questionnaire, and eight SHAs were interviewed by telephone. The remaining five SHAs did not respond either to the initial letter or to the follow-up telephone contacts. It was thought that the five nonresponsive SHAs were likely to still be in the initial developmental stages of implementing a PMS. The nonresponsive SHAs tend to bias the results, but a 90 percent response rate provides adequate information for assessing the state of the practice.

The questionnaire contained the following questions:

1. Is there a precise objective for your state's pavement management process? If so, what is it?
2. Is there an implicit objective for your state's pavement management process? If so, in your judgment, what is it? (Please do not identify abstract objectives like obtaining

the best pavements for the taxpayers' investment. Please be specific.)

3. How does your state's pavement management process prioritize the allocation of resources to alternative projects? If you have priority-ranking criteria or a ranking matrix, please send a copy to the Iowa Transportation Center.

4. Does your PMS contain a network optimization model (i.e., a mathematical model, such as a linear program)? If it does, what is its objective function? What mathematical programming technique does it use? What are the constraints?

Some SHAs submitted reports documenting their PMSs instead of answering the questions. As a result, the data collected are based on the researchers' interpretation of those reports instead of direct answers to the questionnaire.

For each SHA, a summary sheet was completed with all the answers to the questions. Once a summary sheet was completed for each response, a data base was developed using a spreadsheet program. The data base included 50 columns and 47 rows. Each row corresponded to a SHA, and each column represented specific information. The columns were divided into the following sections:

1. The first section identified whether the SHA has a PMS and, if so, what type of algorithm is used to allocate resources (i.e., prioritization scheme or a network optimization).
2. The second section identified factors used to priority rank projects, if a prioritization system is used.
3. The third section identified the methodology used to predict pavement performance. Generally, pavement performance predictions models are used to generate inputs for multiyear programs developed by network optimization models.
4. The fourth section specified the mathematical programming techniques used in the PMS's optimization models, if an optimization is used.
5. For SHAs using optimization models, the fifth section identified the constraints used in each optimization model.
6. For SHAs using optimization models, the sixth section identified the objective function of the model.
7. In most cases, a state will have general purpose for developing a PMS. For example, one state developed a PMS to help defend itself from accusations of prejudicial resource allocation decisions. These purposes were coded in the seventh section.

Each section was divided into all the possible outcomes for the question dealing with each section issue. For example, in Section 3, the columns were titled linear programming, integer programming, dynamic programming, nonlinear programming, incremental benefit-cost analysis, and marginal cost-effectiveness analysis. These are six optimization techniques employed by PMSs. When an SHA indicated which optimization technique it used, a 1 was placed in the corresponding column; otherwise the cell was left blank.

PRIORITIZATION AND OPTIMIZATION

Several methodologies are used to allocate MR&R resources, and agencies have generated their own unique terminology for these methods. They include pavement ranking criteria, pavement condition analysis, priority assessment models, network-level optimization models, prioritization models, and identification of MR&R strategies (*1–5*). The terminology is somewhat confusing and some agencies have developed unique names to identify similar techniques. However, for the purpose of this paper, methodologies are divided into two categories: project prioritization methods and network optimization.

Project prioritization is a method of data analysis that combines pavement condition data into a score or index that represents overall pavement condition. The pavement score is generally expressed on a scale of 10 to 100. All pavement sections are ranked and categorized by type of pavement, traffic volume, road classification, and other factors related to the pavement section. Some SHAs have more complex ranking criteria for which various factors such as friction, structural capacity, and geometric deficiencies are used to establish pavement section ranking (factors most commonly used for prioritization are identified later in the paper). MR&R resources are allocated on the basis of the pavement section's ranking and the priority assigned to it.

A network-level optimization model identifies the network MR&R strategies that maximize the total network benefits (or performance) or minimize the total network cost subject to network-level constraints such as budget limits and desired performance standards (*2*). The pavement section condition values are used as model parameters, decision variables represent the application of selected MR&R strategies to sections, and resource limits and minimum pavement condition or overall minimum pavement network performance are constraints. The model's decision variables determine which treatments are to be applied to which pavement sections.

Most optimization models consider future pavement condition and allocate resources over a span of several years. Therefore, pavement condition prediction models provide technical input to pavement management network optimization models. The performance prediction methods (used by SHAs) will be discussed later.

Table 1 gives the percentage of the SHAs that are or will be capable of performing each level of data analysis. The percentages in the third column of Table 1 total to more than 100 percent because SHAs that use an optimization model often also have a prioritization methodology.

Of the surveyed SHAs, 77 percent priority rank projects and 2 percent plan to implement a prioritization model. Twenty-eight percent of the responding SHAs have network-level optimization models, and an additional 19 percent will have optimization models in the future. Four of the 47 SHAs did

TABLE 1 Agencies with Data Analysis Capabilities

Data Analysis Capability of Agency	Number	Percentage
No PMS	4	9
Prioritization model	36	77
Plans for prioritization model	1	2
Optimization model	13	28
Plans for optimization model	9	19

not have a PMS implemented (as of fall 1991) but were working with a consultant or in-house to develop one.

FACTORS USED TO PRIORITY RANK PROJECTS

Several models exist for developing priority indexes. Usually they are composites of several pavement section condition measures. SHAs were found to use one or more of the condition measures listed in the following; the frequency of their use is identified in Table 2.

- Pavement distress: The evidence of defects in the pavement (e.g., ruts, cracks, potholes, faulting, and blow-ups) is considered pavement distress.
- Ride or pavement roughness: Roughness is a measurement of a vehicle's response to roughness of the pavement profile.
- Traffic: Traffic is generally taken into account through using the average daily traffic volume or estimating equivalent single-axle loadings that a pavement has received. Pavement sections with higher traffic volumes usually receive higher priorities.
- Economic factors: When a treatment is assigned to a project on the basis of life-cycle cost analysis, several economic factors may be used in prioritization, including benefit-cost ratios and cost-effectiveness ratios.
- Functional class: Although several functional classification schemes are used by SHAs, functional classification is sometimes used in prioritization and results in higher-classification roadways' receiving a higher priority.
- Accidents: Accident rates are often taken into consideration when ranking projects, especially with regard to safety-related maintenance activities.
- Friction or skid resistance: Skid resistance is a major component when safety-related maintenance is evaluated.
- Geometric deficiencies: Some SHAs consider the number of specific geometric deficiencies that could create safety problems when selecting MR&R projects. This assumes that the geometric deficiencies could be corrected through MR&R activities. Typical geometric deficiencies used are the number of narrow structures per mile, shoulder width, number of substandard stopping sight distances, lane width, and substandard horizontal curves per mile (*6*).
- Structural capacity: Most SHAs measure the structural capacity of a pavement through measuring the deflection or curve of the pavement that results from a static or repeated load.
- Engineering judgment: Some agencies structure their priority-ranking criteria to include engineering judgment or to be primarily based on engineering judgment.
- Age: When age is taken into account, it generally enters the priority analysis through measuring the number of years the pavement's performance will remain acceptable (remaining service life concept).
- Location: Some SHAs will provide a higher priority to a pavement on the basis of its strategic location. For example, highways that serve production centers, schools, and military facilities must be maintained in good condition without risking possible road closing.

TABLE 2 Factors Used To Priority Rank Projects

Factor	Number	Percentage
Distress	27	57
Ride	21	45
Traffic	19	40
Economics	8	17
Functional classification	7	15
Accident rate	6	13
Friction	5	11
Geometric deficiencies	5	11
Structural capacity	4	9
Engineering judgment	3	6
Age	3	6
Location	1	2

As presented in Table 2, distress, ride, and traffic are the most common factors used in pavement section priority indexes. However, it is clear that there is a great diversity in the conditions included in the priority indexes. Only distress is used by more than half of the SHAs surveyed.

MATHEMATICAL PROGRAMS USED IN OPTIMIZATION MODELS

Thirteen SHAs use network optimization models. Only four strategies are in use, however, and other mathematical program techniques, including dynamic programming, have been proposed (*7*). The distribution of the four approaches are as follows:

Technique	*Number*	*Percentage*
Linear programming	7	55
Integer programming	2	15
Incremental benefit-cost	2	15
Marginal cost-effectiveness	2	15

Linear and integer programs are two widely used mathematical programming techniques and are commonly applied to solve a range of problems in all sectors of government and business. They are naturally suited to issues dealing with resource allocation. Incremental benefit-cost is a recursive algorithm and seeks to allocate each increment of resources to projects that provide the largest possible increment of benefit. A close cousin to incremental benefit-cost is marginal cost-effectiveness. The primary difference in the two methods is the terminology used (benefits versus effectiveness). However, both should result in the same solution.

PAVEMENT PERFORMANCE PREDICTION METHODOLOGIES

Performance is the "ability of a pavement to fulfill its purpose over time" (*2*). A prediction method is "a mathematical description of the expected values that a pavement attribute will take during a specified analysis period" (*2*). Prediction models provide parameters to pavement management optimization so that they can base the selection of future MR&R programs on the forecasted conditions.

Although most prediction models are deterministic, probabilistic models are being implemented in SHAs. The prediction models identified in the survey are as follows:

- Performance curves: A performance curve defines variations of pavement attributes over time. SHAs create performance curves for their particular conditions. An SHA will have as many performance curves as different pavement types exist in its jurisdiction. In other words, a bituminous pavement with high traffic and low subgrade strength may have a different performance curve than a concrete pavement with low traffic and medium subgrade strength. Performance curves normally calculate expected serviceability and age relationship over the entire design period (*3*). Other attributes or indexes can also be used to establish new relationships. These include structural capacity versus age, skid resistance versus age, and a measure of distress versus age. The relationship between the variables is commonly estimated using regression.
- Markov chain: The Markov chain is a probabilistic model that accounts for the uncertainties present with respect to both the existing pavement condition and future pavement deterioration. The underlying concept of this method is that a pavement section may be in one of several states or conditions and that unless maintenance or rehabilitation is undertaken, the condition of the pavement will worsen over time. The amount of pavement deterioration in a given period, such as a year, is a random variable depending only on the most recent state of the pavement and the amount and type of traffic loading that the pavement accrues during that period of time.

One of the difficulties of using a Markov chain model for predicting performance is that it predicts the proportion of the entire pavement network falling into each pavement condition category during each future period. Because it forecasts the distribution of future pavement conditions, it does not predict the specific condition of a specific section and does not allow any later project-level analysis. A Markov chain model is only useful for network-level analysis.
- Survival rate: When an MR&R treatment is applied, a pavement section increases its condition rating. The potential gain of rating is defined as the net expected increase of pavement rating of the section. To predict the effects of MR&R treatments over a chosen planning period, the potential rating gain is affected by a pavement survival rate. For each section, a pavement survival matrix, which that contains the survival probability for each distress type and MR&R treatment, is developed. The term "survival" indicates that the pavement condition is still expected to rate high enough that it will not require additional MR&R work at a future specified point (*8*). For each specific highway section, each particular MR&R strategy, and each distress, the survival probability (or rate) decreases with time. For example, for Year 0, when the treatment is applied, the survival probability is 1; at Year 2, it could be 0.8; at Year 4, 0.5, and so forth. If, for example, the rating change of a particular pavement section is desired 3 years after a certain treatment is applied, the potential gain of the pavement section due to the treatment application at Year 0 is multiplied by the survival probability for that section and treatment for Year 3.

This prediction method is quite data-intensive. Data sets must be collected to develop each pavement survival matrix for each pavement category. This information is managed in the form of vectors and matrices when all the sections, distresses, and MR&R strategies are analyzed together. This method of prediction looks at the effect of each MR&R treatment on each type of distress of a pavement section. Therefore, it is used in optimization models that look for maximum maintenance effectiveness.

The following table shows the number of SHAs that use each of the three prediction models:

Performance Prediction Method	*Number*	*Percentage*
Performance curve	6	46
Markov chain	5	38
Survival rate	1	8
Did not identify	1	8

The choice of pavement prediction modeling methodology is linked to the optimization method selected and the objective of the pavement management optimization. As mentioned, the Markov chain is a prediction tool only for network-level optimization and thus linked to network-level analysis, network objectives, and specific optimization techniques.

CONSTRAINTS USED IN OPTIMIZATION MODELS

When selecting projects, SHAs are constrained by different factors. All proposed projects cannot be funded in a single year or even through a multiyear funding plan. Some SHAs did not identify a specific constraint and others specified more than one. The following constraints were identified in SHAs with optimization models:

- Budget: The budget is the maximum level of funding available in 1 year or in several years over a multiyear plan. The constraint would ensure that the solution does not exceed the available budget.
- Minimum pavement condition requirement: This constraint could be either a minimum average network performance or a maximum percentage of sections allowed below the minimum acceptable value.
- Resources: Resources for pavement MR&R can be categorized as materials and supplies, equipment, or personpower (*8*). Similarly to a budget constraint, a resource constraint does not allow the solution to exceed the amount of available resources.
- Others: Some SHAs included constraints such as the number of days available to perform construction activities (this is particularly important in Snow Belt states where cold weather restricts the number of days in which maintenance can be performed), local legislative requirements, and political issues (e.g., equal distribution of funds to geographic districts of a state).

The following table summarizes the frequency with which each type of constraint is used by the 13 SHAs that operate a network optimization model:

Constraint	*Number*	*Percentage*
Budget	13	100
Minimum pavement condition requirement	5	39
Resources	2	15
Other	5	39

OBJECTIVE FUNCTIONS USED IN OPTIMIZATION MODELS

Each of the mathematical programming techniques requires a specific objective function. Mathematical programs select solutions to decision variables that optimally satisfy the objective function subject to the identified constraints. Four categories of objective functions were employed by SHAs using network optimization:

- Minimize cost: When various MR&R strategies are available for each project in the analysis period, the alternatives that satisfy the constraints and generate the lowest overall cost are selected.
- Maximize area under performance curve: When an MR&R strategy is assigned to a pavement section, a performance curve for this section is predicted. The area under this curve is a measure of effectiveness and is a surrogate for the benefit of applying the treatment. The optimal combination of strategies will be those that maximize the combined increase in the area under all the performance curves while satisfying the optimization's constraints.
- Minimize disutility: Instead of predicting pavement performance, some agencies predict the severity of distress, the level of maintenance costs, or the user cost. The area under these curves is called disutility. The objective function defines the selection of treatments that minimize disutility.
- Maximize maintenance effectiveness: This is the ability of a treatment strategy to eliminate a particular distress type for as long as possible (*8*). For example, in a pavement section for which rutting is a problem, the treatment that eliminates the ruts from the pavement for the longest amount of time will be the one selected. An optimization model can use this objective function when survival rates are used to predict the performance of each pavement section for each distress type.

The following table summarizes the objective functions used by the 13 SHAs that operate network-level PMSs (because some optimization models can use more than one objective function, the percentages do not total 100):

Objective Function	*Number*	*Percentage*
Minimize cost	8	62
Maximize area under performance curve	5	39
Minimize disutility	1	8
Maximize maintenance effectiveness	1	8

CONCLUSIONS

The state of the practice in pavement management is still in the developmental stages. All SHAs either have a PMS or are working to achieve an acceptable PMS. Roughly a quarter of the SHAs have advanced their PMSs to the network optimization level. The results presented here, however, indicate that more states use a network optimization model than was previously estimated (*4*).

Given the widespread use of pavement management, it seems clear that there is a need to promote the development of the science of pavement management. More specifically, a mature science of pavement management should have a common terminology, standard data collection procedures, and comparable data analysis methods. To the contrary, through the survey it was found that different agencies sometimes use incomparable terminology, that some agencies did not appear to understand the objectives of the analysis models imbedded within their own PMS computer software, and that little technical information on the science of PMSs appears to be shared between states. At the national level, the researchers believe that it is time to promote the science of pavement management. It is probably also true that the science of maintenance management of all types of public infrastructure needs to be developed into a more formal and structured field of knowledge. In prior research, we have noted that maintenance management training is painfully lacking in conventional engineering education and continues to be an area desperately needing improvement (*9*).

One of the more specific conclusions is that there appears to be no unanimity in the inputs to the pavement management process or to the analysis methods and objectives used. SHAs have selected diverse analysis tools and methods for use within their PMSs. For example, in the development of a composite measure for priority ranking pavement MR&R work, there is little consensus on which factors are important. Distress is the most frequently used factor in pavement priority indexes, but only about half of the SHAs used distress in their prioritization methodology. Inputs to the prioritization process include one or more of 11 categories of pavement condition measures and other economic, traffic, or safety factors. Even though only 13 SHAs have reached the level of performing network optimization, four fundamentally different methodologies are being applied and other optimization methodologies are being proposed or developed (*7*).

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Simplified Pavement Performance Models

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There is a great need for simplified pavement performance models that can be used for forecasting pavement condition on the basis of a minimal amount of available data. The development of predictive models is summarized for five conventional pavement types: asphalt concrete (flexible), composite, jointed plain concrete, jointed reinforced concrete, and continuously reinforced concrete. These models predict the present serviceability rating (PSR) using only knowledge of the pavement's age, cumulative equivalent single-axle loads, and a pavement structural parameter (structural number for flexible, overlay thickness for composite, and slab thickness for concrete pavements). The models were developed from data from several reliable and readily available data bases in Illinois. A unique calibration technique was introduced and incorporated into the proposed models so that they can be used to predict the performance of existing and new pavements. The models were then extended through the development of adjustment factors to various functional groups and climatic zones using data from the actual multiyear nationwide Highway Performance Monitoring System (HPMS) data bases. The accuracy of PSR prediction was tested for several thousand HPMS sections throughout the United States using a user-friendly computer program (SIMPERF). The results appeared to be very reasonable in a large proportion of cases analyzed. However, the models are empirical and definitely not suitable for use in pavement design or for comparison of the performance of different pavement types.

Many pavement management activities require the prediction of pavement performance in a network. One example is the determination of future pavement rehabilitation needs for a state highway network from which a multiyear plan for rehabilitation is formed. In fact, every agency that owns or is responsible for pavements and wishes to manage those facilities in a rational manner needs to be able to predict the performance of their pavements.

However, collecting reliable inventory and monitoring data to develop predictive models for a large pavement network system is a formidable and very costly task. Many agencies do not have comprehensive data bases that can provide a lot of data about each section in the network. Although this inadequacy is improving through the development of pavement management systems, most agencies can provide only the current condition (in various forms), the current average daily traffic (ADT) and percentage trucks, type of pavement, and perhaps some design and rehabilitation history for their pavement sections. Thus, there is a great need for simplified pavement prediction models that require only a minimal amount of data likely to be available in the pavement management data base.

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FHWA must report to the Congress on a regular basis the long-term needs of the nation's highway system, and pavements are the system's largest component. This paper summarizes the development of predictive models and mean adjustment factors to be used in the Highway Performance Monitoring System (HPMS) pavement performance simulation process (*1*).

IDENTIFICATION OF PAVEMENT GROUPS

Item 28 of the HPMS data elements (pavement attributes) includes 15 pavement types, which cover nearly all combinations of original construction and rehabilitation types. However, they are separated into only two main groups—flexible and rigid pavements—in the HPMS pavement performance simulation process. The AASHTO flexible and rigid pavement equations are then used for pavement performance simulation. This procedure has some obvious deficiencies.

To more adequately represent a wide variety of different pavement attributes in the HPMS, the following five major conventional pavement types were considered:

- Asphalt concrete (flexible, or FLEX),
- Composite (AC/portland cement concrete, or COMP),
- Jointed plain concrete pavement (JPCP),
- Jointed reinforced concrete pavement (JRCP), and
- Continuously reinforced concrete pavement (CRCP).

Some pavement types—including unimproved road, graded and drained, soil, grave, or stone, bricked, blocked, and other combinations—in the HPMS were not considered in this study.

Nine climatic zones (Item 68) based on Thornthwaite potential evapotranspiration and moisture index and their interaction (*1,2*) were also considered in the "group" identification. This provides a fairly adequate consideration of the diverse climates and geographic areas that exist across the United States, including any combination of wet, intermediate, and dry climates in freeze, freeze-thaw, and no-freeze regions.

Item 9 of the data elements contains 12 functional systems. After analysis and discussion with FHWA, they were condensed into two major functional groups (FGROUP). Interstate highways and principal arterials were treated as one group, and minor arterials and all collectors were treated as the other. This grouping was done to reflect expected differences in cross sections, drainage, and pavement performance.

The HPMS data base was then divided into similar performance groups, which were expected to have similar deterioration mechanisms and performance relationships. A given pavement group was defined having the same general pavement type, functional group, and climatic zone as previously described. It is assumed that pavements within the same group more or less follow the same performance pattern. Thus, predictive models need only be developed for a few groups of conditions, as opposed to many different types of pavement design, functional system, climatic region, and rehabilitation type.

DEVELOPMENT OF PERFORMANCE PREDICTION MODELS

After considerable review of different regression techniques, it was decided that nonlinear regression should not be used to develop predictive models for the HPMS because of the high possibility of having many errors in the data base. Several trials using nonlinear regression produced unacceptable models largely due to including some bad data points in the analysis. Therefore, the following steps were adopted to develop predictive models:

1. A feasible general present serviceability rating (PSR) loss model form was assumed including variables based on engineering knowledge and available data bases.
2. Least-median-squares, or "robust," regression was performed to identify the potential outliers by using this assumed model form (*3,4*).
3. After screening out possible outliers, traditional least-squares regression was then used to obtain the regression coefficients and summary statistics.

Because it cannot be guaranteed a priori that the assumed functional form is valid, the analysis must proceed iteratively so that a more meaningful and reliable model can be developed. An alternating conditional expectations algorithm (*5*) was also applied to find other possible transformations of each explanatory variable to maximize the squared multiple correlation coefficient (R^2) for the next trial.

A new statistical package named S-PLUS, which has been widely used by statisticians for data analysis (*6–8*), was selected because of the availability of these techniques. S-PLUS is very strong in its graphics, data exploration tools, and flexibility but weak in data base management as compared with the most well-known and widely used statistical package, SAS (*9*). As a result, SAS was used primarily for data retrieval and data summary whereas S-PLUS was used for most of the modeling processes.

Attempts To Develop Models Directly from HPMS Data Base

Five sets of the HPMS data base in 1982, 1984, 1986, 1988, and 1989 were first retrieved from magnetic tapes (*1*) and downloaded to a personal computer (PC) for further analysis. To obtain the needed history of the HPMS pavement performance, the data were merged by their unique identification number, that is, sample number (Item 24) and sample subdivision (Item 25).

Initially, major research efforts were focused on developing predictive models directly from the HPMS data base using data from 1984 to 1989. Several feasible model forms were used to develop the performance prediction models. Robust regression successfully identified portions of the data base as potential outliers, which after deletion improved the regression dramatically. However, the regression models were still not adequate for implementation. This attempt was unsuccessful because of problems with the HPMS data base, such as missing data, highly variable performance histories, and apparent errors in many important data elements.

Alternative Data Bases for Model Development

Owing to the difficulties in developing prediction models directly from the HPMS data base, other accessible data bases were considered for developing PSR prediction models for each of the five major pavement types. They include the pavement management data base from the Illinois Department of Transportation, the Illinois portions of the NCHRP Project 1-19 data base (*10*), the original AASHO Road Test data (DS 7322) (*11*), and some additional data from the extended road test (1962–1974) (*12,13*).

The Illinois pavement management data base contains detailed information about pavement inventories, materials, distress surveys, condition rating surveys, maintenance and rehabilitation records, and traffic data. The most recent data (March 1991)—which contain six condition rating surveys, in 1981, 1982, 1984, 1986, 1988, and 1990—were obtained to construct data bases for CRCP and composite pavements.

The NCHRP Project 1-19 data base, which contains some existing Illinois Interstate JRCP pavements and sections from the original and the extended AASHO Road Test for JRCP, was used to construct a JRCP data base. The JPCP data base was constructed from the original and the extended AASHO Road Tests. The serviceability records of flexible pavements of the original AASHO Road Test at 22-week (or 11-index-day) intervals were obtained to create the data base for flexible pavement.

Proposed Predictive Model Form

After considerable evaluations of different model forms including linear, logarithm, and other simplified forms, the following functional form was chosen to develop the proposed HPMS predictive models for all five major pavement types:

$$PSR = PSR_I - a * STR^b * AGE^c * CESAL^d \tag{1}$$

where

PSR_I = initial value of PSR at construction (4.5 used in analysis);
STR = existing pavement structure: structural number for flexible pavement, total AC overlay thickness for composite pavements (in.), and slab thickness for concrete pavements (in.) (1 in. = 25.4 mm);
AGE = age of pavement since construction or major rehabilitation (overlay) (years); and
CESAL = cumulative 18-kip equivalent single-axle loads (ESALs) applied to pavement in the heaviest traffic lane (millions).

This nonlinear model form is also an implicit linear model since after transformation it becomes

$$\log_{10}(\text{PSR}_I - \text{PSR}) = \log_{10}a + b * \log_{10}\text{STR} + c * \log_{10}\text{AGE} + d * \log_{10}\text{CESAL} \quad (2)$$

This nonlinear model form permits a realistic consideration of age, traffic, and pavement structure on the prediction of PSR. Subsequent model development has shown that this equation form fits all of the pavement types reasonably well.

Note that the structural number is reported as an indicator of pavement structure for both flexible and composite pavements in the HPMS data base so that the AASHTO FLEX equation could be used to predict performance. However, composite pavements perform dramatically different from flexible pavements due to different failure modes. It is believed that the AC overlay thickness rather than the structural number or the underlying concrete slab thickness is the dominating factor in the performance of composite pavements. Thus, overlay thickness was used in the model development. The questionable determination of structural number for composite pavements is no longer needed in the HPMS data base since no adequate guidelines are available.

Summary of Proposed Predictive Models

The regression coefficients and summary statistics of each predictive model for all five major pavement types are summarized in Table 1. The standard error of estimates (*SEE*) as provided in the table is also a very good indicator of the accuracy of the prediction of the loss of PSR (ΔPSR). The number of potential outliers identified and then excluded from the model are also indicated by parentheses in the table. For example, 31 out of 553 data points were deleted from the FLEX model.

The statistics of the CRCP model are not very good as expected, since both D-cracked and non-D-cracked pavements from the Illinois Interstate highways were all included in the data base to develop this model. This model can be improved after more D-cracking information is collected in the HPMS data base.

To check the adequacy of each proposed model, the predicted ΔPSR values were plotted against the actual values as shown in Figures 1 through 5. Several sensitivity analyses of the variables included in each model were also performed and found to be very reasonable (*14*). In general, the PSR curves of FLEX, COMP, and CRCP are in a concave shape or have more rapid loss of PSR early. The PSR curves of JPCP and JRCP are in a convex shape or have more rapid loss of PSR later.

APPLICATION OF PROPOSED MODELS TO HPMS

Calibration of Models to Existing Pavement Conditions

On the basis of the proposed predictive models, a fixed family of curves could be developed for different pavement structures. Unfortunately, both age and cumulative ESALs are not available in the HPMS data base. Therefore, it is necessary to obtain the best estimates of pavement age and cumulative ESALs through knowledge of only the current annual ESALs and the current year condition of an existing pavement structure in the HPMS data base.

Assume that there is a direct relationship between pavement age and cumulative ESALs:

$$\text{CESAL} = \text{AGE} * \text{ESALPYR} \quad (3)$$

where ESALPYR is current yearly ESALs in millions.

TABLE 1 Summary of Proposed Predictive Models

	Model				
	FLEX	COMP	JPCP	JRCP	CRCP
$\log_{10}a$	1.1550	-0.4185	0.5104	1.7241	0.7900
b	-1.8720	-0.1458	-1.7701	-2.7359	-1.3121
c	0.3499	0.5732	1.0713	0.3800	0.1849
d	0.3385	0.1431	0.2493	0.6212	0.2634
R^2	0.52	0.58	0.79	0.57	0.37
SEE	0.45	0.38	0.26	0.40	0.31
N	522 (31)	509 (0)	117 (3)	254 (21)	1204 (65)

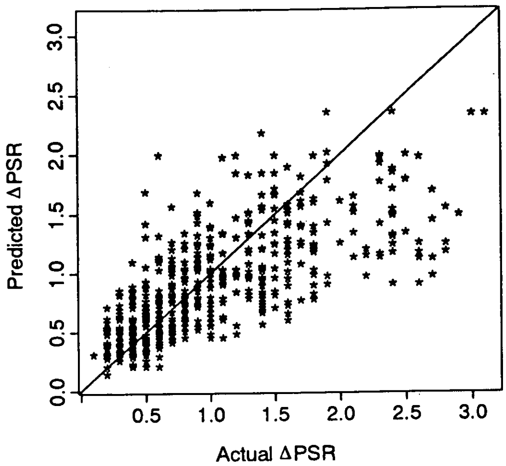

FIGURE 1 FLEX model: predicted versus actual ΔPSR.

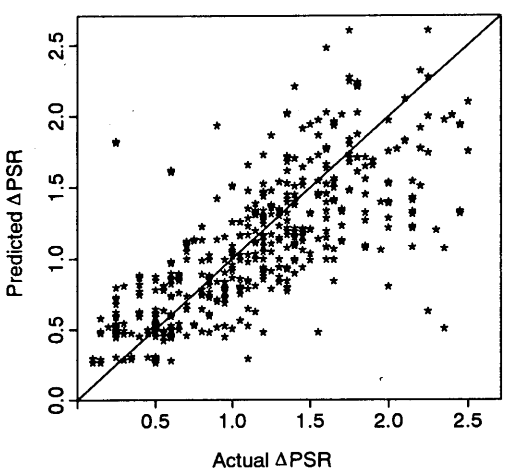

FIGURE 2 COMP model: predicted versus actual ΔPSR.

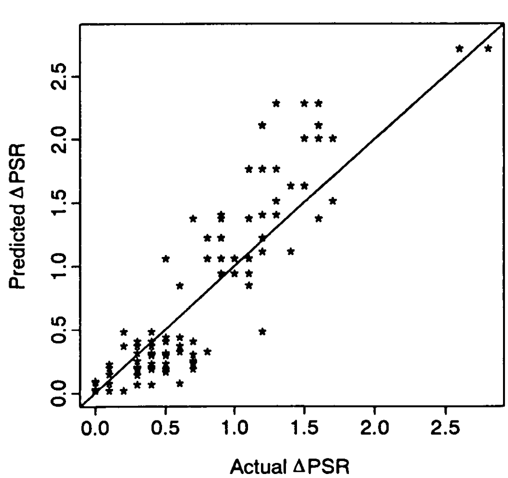

FIGURE 3 JPCP model: predicted versus actual ΔPSR.

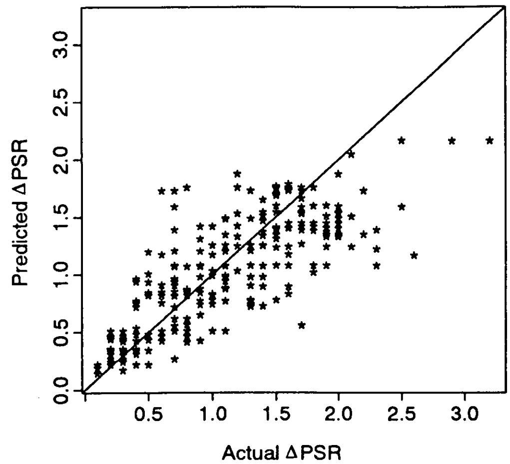

FIGURE 4 JRCP model: predicted versus actual ΔPSR.

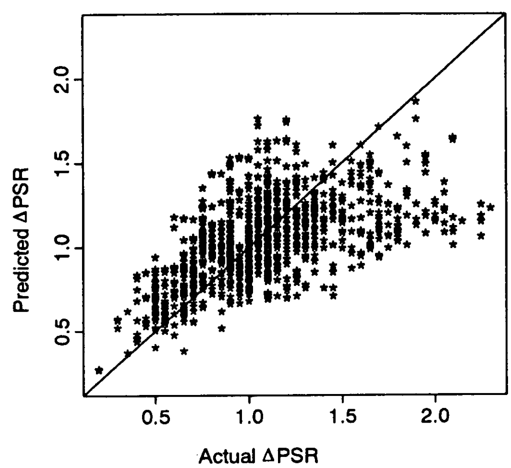

FIGURE 5 CRCP model: predicted versus actual ΔPSR.

To locate the current year condition in a unique performance curve, the following calibration constants (C_1 and C_2) could be treated as the best estimates of pavement age and cumulative ESALs, respectively:

$$C_1 = \text{AGE} = \left[\frac{\text{PSR}_I - \text{PSR}_1}{a * \text{STR}^b * \text{ESALPYR}^d}\right]^{\frac{1}{c+d}} \tag{4}$$

$$C_2 = \text{CESAL} = C_1 * \text{ESALPYR} \tag{5}$$

where PSR_1 is the current year pavement condition. Thus, the proposed models can be reformulated to the following form, which is a function of the current year condition, a pavement structure parameter, and the current annual ESALs of an existing pavement in the HPMS data base:

$$\text{PSR} = \text{PSR}_I - a * \text{STR}^b * (C_1 + \Delta\text{YEAR})^c * (C_2 + \Delta\text{ESAL})^d \tag{6}$$

where ΔYEAR is the change in age of pavement in years, and ΔESAL is the change in cumulative ESALs in millions.

Adjustment Factors for Different Pavement Groups

In addition, adjustment factors similar to the regional factor adopted in the 1972 *AASHO Interim Guide* (*15*) were introduced to adjust the rate of deterioration of PSR of the proposed models for different pavement groups in the HPMS data base. The adjustment factor is defined as the ratio of the average rate of deterioration in a particular climatic zone and functional group to that determined by the proposed models:

$$AF_j = \frac{\Delta PSR_j}{\Delta PSR} = \frac{PSR_I - PSR_j}{PSR_I - PSR} \tag{7}$$

where

AF_j = adjustment factor in pavement group j;
ΔPSR_j, PSR_j = actual ΔPSR and PSR values of existing pavements in group j, respectively; and
ΔPSR, PSR = predicted ΔPSR and PSR values of existing pavements determined by proposed models, respectively.

An adjustment factor greater than 1.0 indicates that the actual rate of PSR loss is greater in that pavement group than the rate predicted by the model based on Illinois conditions, and vice versa. For example, the effects of adjustment factors of a flexible pavement with a structural number of 6 and traffic load of 0.5 million ESALs per year are illustrated in Figure 6.

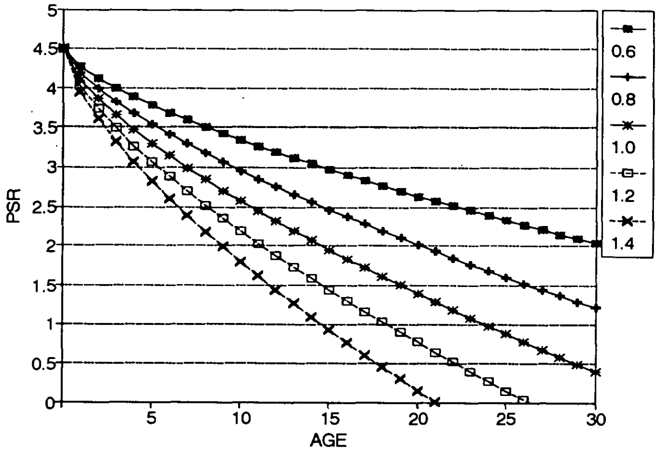

FIGURE 6 Effects of adjustment factors on pavement performance.

Determination of Mean Adjustment Factors

Five sets of the HPMS data base in 1982, 1984, 1986, 1988, and 1989 were received from FHWA. However, it was decided not to use the 1982 data base for determining adjustment factors after discussion with FHWA personnel. Any PSR value that increases more than 0.5 or decreases more than 0.75 a year was deleted to avoid retrieving sections that have been rehabilitated or had apparently deteriorated too fast to be believable. In addition, only the 3-, 4-, and 5-year PSR drops were retrieved because the PSR records may not be updated during a very short reporting cycle (1 or 2 years).

A total of 85,533 data points were obtained from all five major pavement types, nine climatic zones, and two functional groups. Note that a few very large or very small adjustment factor values (1.7 percent of the data), which were outside the range of −10 to 10, were excluded from further consideration. The mean adjustment factors determined on the basis of a different number of sections ranging from several thousand down to only one in a few cases are summarized in Table 2 (*14*). Mean values based on fewer than 25 data points and marked with an asterisk in Table 2 should not be strongly considered.

The mean values vary widely across pavement type, climatic zone, and functional group, especially when they were determined on the basis of only a few data points. In general, pavements in the South (fewer freeze-thaw and cold temperatures) showed a lower deterioration rate than those in northern climates. And pavements in the western United States (drier climate) showed a lower rate of deterioration than those in wetter climates in the East.

In addition, higher variation of the adjustment factors was observed for pavements in minor arterials and collectors. This may also be explained by the fact that the most important indicator of pavement structure (structural number or slab thickness) is not recorded in the HPMS data base. Thus, default values for these pavement sections rated as heavy, medium, and light (Item 31) (*1*) were assigned to determine the adjustment factors.

The adjustment factors as given in Table 2 obviously have very strong effects on the pavement performance prediction. They were also evaluated for several thousand HPMS sections using a user-friendly PC program (SIMPERF). The overall results showed that using either very high or very low adjustment factors produced unreasonable future service lives of HPMS pavement sections. A recommended adjustment factor ranging from approximately 0.4 to 1.5 is believed to provide reasonable PSR predictions. Many of the mean values that fall outside this range are the result of a small sample size and would thus be expected to be highly variable.

Table 3 provides the recommended mean adjustment factors for use as defaults in the HPMS analytical process. The mean values were recommended, unless the value fell outside this range. For those lower than 0.4 or higher than 1.5, the value was assigned to that cell. The only exception to these rules was for CRCP in minor arterials and collectors, where a value of 1.0 was assigned for wet and intermediate zones and a value of 0.5 was for dry zones.

Currently, the recommended adjustment factors along with the proposed predictive models are implemented in the SIMPERF program for pavement performance and subsequent rehabilitation simulations. Users can modify the recommended adjustment factors, which can be easily adjusted to more accurately reflect the performance of any given pavement type.

TABLE 2 Mean Adjustment Factors Directly Generated from SAS Program

	PTYPE									
	FLEX		COMP		JPCP		JRCP		CRCP	
	FGROUP		FGROUP		FGROUP		FGROUP		FGROUP	
	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL
	AF	AF	AF	AF	AF	AF	AF	AF	AF	AF
	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN
ZONE										
1. Wet; Freeze	0.59	0.81	1.04	1.11	0.56	0.99	0.87	2.27	0.57	-0.18*
2. Wet; Freeze-Thaw	0.37	0.85	1.13	1.07	0.33	0.64	1.25	1.46	0.39	2.12*
3. Wet; No Freeze	0.44	0.69	0.78	0.31	0.60	0.55	0.57	0.97	1.08	1.74*
4. Intermediate; Freeze	0.27	0.49	0.55	1.15	0.46	0.52	0.23	0.12*	0.94	0.00*
5. Intermediate; Freeze-Thaw	0.52	0.71	0.26	0.64*	0.66	1.61*	2.09	1.00*	1.10*	1.56*
6. Intermediate; No Freeze	0.43	0.65	0.71	0.87	0.27	1.34	1.71	2.61	2.02	.
7. Dry; Freeze	0.22	0.43	0.76*	2.53*	0.79	0.79	0.22	0.00*	0.17	.
8. Dry; Freeze-Thaw	0.32	0.39	-0.44*	0.00*	1.80	.	0.49*	0.00*	-0.13	.
9. Dry; No Freeze	0.38	0.79	0.26	.	0.22	0.66*	-0.30*	2.10*	0.45*	.

Note:
INT/OPA = Interstate highways and other principal arterials, FGROUP=1
MA/COL = minor arterials and collectors, FGROUP=2
* = mean AFs based on 25 data points or less
. = data unavailable

TABLE 3 Recommended Mean Adjustment Factors for Different Pavement Groups

	PTYPE									
	FLEX		COMP		JPCP		JRCP		CRCP	
	FGROUP		FGROUP		FGROUP		FGROUP		FGROUP	
	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL	INT/-OPA	MA/C-OL
	AF	AF	AF	AF	AF	AF	AF	AF	AF	AF
	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN	MEAN
ZONE										
1. Wet; Freeze	0.59	0.81	1.04	1.11	0.56	0.99	0.87	1.50	0.57	1.00
2. Wet; Freeze-Thaw	0.40	0.85	1.13	1.07	0.40	0.64	1.25	1.46	0.40	1.00
3. Wet; No Freeze	0.44	0.69	0.78	0.40	0.60	0.55	0.57	0.97	1.08	1.00
4. Intermediate; Freeze	0.40	0.49	0.55	1.15	0.46	0.52	0.40	0.40	0.94	1.00
5. Intermediate; Freeze-Thaw	0.52	0.71	0.40	0.64	0.66	1.50	1.50	1.00	1.10	1.00
6. Intermediate; No Freeze	0.43	0.65	0.71	0.87	0.40	1.34	1.50	1.50	1.50	1.00
7. Dry; Freeze	0.40	0.43	0.76	1.50	0.79	0.79	0.40	0.40	0.40	0.50
8. Dry; Freeze-Thaw	0.40	0.40	0.40	0.40	1.50	0.40	0.49	0.40	0.40	0.50
9. Dry; No Freeze	0.40	0.79	0.40	0.40	0.40	0.66	0.40	1.50	0.45	0.50

Further Discussion

An adjustment factor represents the ratio of the PSR loss of a section of highway to the PSR loss predicted by the model for that section. Many reasons for differences in performance are not climate-related, including different subgrades, materials, construction quality, design (such as joint design), and maintenance. The adjustment factors should be compared only within a pavement type since each predictive PSR model was based on a different data base. Comparisons between different pavement types are not meaningful.

The predictive models were developed on the basis of field data from regular in-service pavement sections, which included maintenance (except the AASHO Road Test pavements). Heavy maintenance could prevent pavements from deteriorating to very low serviceability levels. All of the proposed models show that the rate of deterioration decreases as PSR decreases, which reflects the impact of maintenance when the pavement conditions get worse.

The FLEX model was based on the original AASHO Road Test data only. The mean adjustment factor for wet-freeze zone is 0.59 based on 5,685 data points for Interstate highways and principal arterials. This indicates that the pavements in the HPMS data base in the same climatic zone have performed better than the pavements of the AASHO Road Test. In general, the mean values decrease from wet to dry climatic zones, which means that flexible pavements in drier climates show a lower rate of PSR loss. The Interstate highways and principal arterials have lower adjustment factors and thus perform better than the minor arterials and collectors.

In addition, the adjustment factors for the FLEX model were computed for two major functional groups and for levels of ADT greater and less than 6,000 vehicles. Even when similar traffic levels were considered, the Interstate highways and principal arterials still exhibited lower adjustment factors than the other group. Also, within the same functional group, the ADT level did not appear to cause a consistent difference in the adjustment factors. These results may indicate that some physical difference, such as improved drainage or construction quality for the Interstate highways and primary arterials, results in a lower rate of deterioration for the same traffic level.

The COMP model was based on the in-service Illinois Interstate highway pavements. In the wet-freeze climate, the mean adjustment factor is close to 1.0, indicating that on average other pavements in this zone are performing similarly. As with flexible pavements, the adjustment factor decreases with a drier climate and increases with the lower functional group.

The JPCP model was based on the AASHO Road Test data plus a few sections that were left in-service on I-80 for 14 years. The mean adjustment factor for wet-freeze zone is 0.56 based on 946 data points for Interstate highways and principal arterials. This indicates that the pavements in the HPMS data base in this climate zone have performed better than the JPCP at the AASHO Road Test. The mean values generally show a decrease going from wet to dry climatic zones, which means that JPCP in drier climates shows a lower rate of loss of PSR. A previous study showed that JPCP in a dry-nonfreeze climate performed much better than that in a wet-freeze climate (*10*).

The JRCP model was based on the AASHO Road Test data plus many sections from regular Illinois Interstate highways. The mean adjustment factor for the wet-freeze zone is 0.87 based on 2,149 data points for Interstate highways and principal arterials. This indicates that the pavements in the HPMS data base in this climate zone have performed about the same as the combined AASHO Road Test and Illinois Interstate highways. The mean values show a wide range of results over different climatic zones. However, the number of data points from many of the JRCP sections is very limited, which has caused some wide-ranging results.

The CRCP model was based on many sections from regular Illinois Interstate highways. The mean adjustment factor for the wet-freeze zone is 0.57 based on 462 data points for Interstate highways and principal arterials. This indicates that the pavements in the HPMS data base throughout this climate zone have performed better than the Illinois Interstate highways. This may be due to the large amount of D-cracking in the Illinois CRCP pavements. The values show a wide range of results over different climatic zones, but, as in the JRCP sections, the number of data points from many of the CRCP sections is very limited and results in wide-ranging results.

Summary of Proposed HPMS Performance Prediction

The proposed HPMS performance prediction equations for both existing and new pavements in pavement group j based on only knowledge of a given pavement structure, current year condition, and current yearly ESALs are summarized as follows:

$$\mathrm{PSR}_j = \mathrm{PSR}_I - \mathrm{AF}_j * [a * \mathrm{STR}^b * (C_{1j} + \Delta\mathrm{YEAR})^c * (C_{2j} + \Delta\mathrm{ESAL})^d] \tag{8}$$

$$C_{1j} = \mathrm{AGE}_j = \left[\frac{\mathrm{PSR}_I - \mathrm{PSR}_1}{\mathrm{AF}_j * (a * \mathrm{STR}^b * \mathrm{ESALPYR}^d)}\right] \tag{9}$$

$$C_{2j} = \mathrm{CESAL}_j = C_{1j} * \mathrm{ESALPYR} \tag{10}$$

The calibration constants (C_{1j} and C_{2j}) can be treated as the best estimates of current pavement age (AGE_j) and current cumulative ESALs (CESAL_j) for any existing pavement in Group j.

To predict the performance of an existing pavement, proper coefficients based on major pavement type, climatic zone, and functional group are first selected, that is, AF_j, $\log_{10}a$, b, c, and d (Tables 1 and 3). C_{1j} and C_{2j} based on known STR, PSR_1, ESALPYR, and these coefficients are then determined. Thus, the future performance can be estimated for different future ΔESAL and ΔYEAR using Equation 8.

Numerical Example

Consider a high type-flexible pavement classified as a rural major collector and located in Climatic Zone 1. This pavement has a structural number of 5.0 and its current condition is 3.5 in 1991. The current yearly ESAL is 0.2 million with an average compounded future yearly ESAL growth rate of 6 percent.

The coefficients of the proposed model for flexible pavements (as given in Table 1) are $\log_{10}a = 1.1550$, $\log_{10}b =$

-1.8720, $\log_{10}c = 0.3449$, and $\log_{10}d = 0.3385$. The adjustment factor for pavements from rural major collectors in climatic zone 1 (as given in Table 3) is $AF_j = 0.81$. Thus, the best estimates of current pavement age (C_{1j}) and current cumulative ESALs (C_{2j}) using Equations 9 and 10 are $C_{1j} = 5.007$ years and $C_{2j} = 5.007 * 0.2 = 1.001$ million ESALs.

Therefore, the following equation can be used to predict the future performance of this pavement:

$$PSR_j = 4.5 - 0.81 * 10^{1.1550} * 5.0^{-1.8720} * (5.007 + \Delta YEAR)^{0.3499} * (1.001 + \Delta ESAL)^{0.3385} \quad (11)$$

ΔESAL based on a compound yearly ESAL growth rate (ESALGRW) can be calculated by

$$\Delta ESAL = \frac{ESALPYR * (1 + ESALGRW) * [(1 + ESALGRW)^{\Delta YEAR} - 1]}{ESALGRW} \quad (12)$$

or, in this case,

$$\Delta ESAL = \frac{0.2 * (1 + 0.06) * [(1 + 0.06)^{\Delta YEAR} - 1]}{0.06} \quad (13)$$

CONCLUSIONS

The proposed models predict the PSR using only knowledge of the pavement's age since construction, cumulative ESALs, and a pavement structural parameter. The models were developed for five major pavement types based on data from the original and the extended AASHO Road Tests, the NCHRP Project 1-19 (COPES), and the Illinois pavement feedback system data bases.

A unique calibration technique was introduced and incorporated into the proposed models so that they can be used for performance prediction of both existing and new pavements. For an existing pavement, the predictive model is calibrated to its current condition and then projected into the future, which also greatly reduces the prediction error. Mean adjustment factors were also determined using the actual multiyear nationwide HPMS pavement performance data and engineering judgment to extend these models to other climatic zones and functional groups.

The reasonableness of PSR predictions was tested for several thousand HPMS sections throughout the United States according to researchers' past experience in pavement performance using the SIMPERF program. The results appeared to be reasonable in a large proportion of cases analyzed using the recommended adjustment factors. The mean adjustment factors can be easily adjusted by individual states or FHWA.

Many other factors also affect the performance of these pavements, although these factors are not reflected in the simplified models. Thus, the models should be used only to predict the performance of existing pavements. They are definitely not appropriate for use in pavement design or for comparison of the performance of different pavement types.

Within this context, it is believed that the predictive models and adjustment factors for other geographic and climatic areas and functional groups are approximate but reasonable for the purposes intended. There is no doubt that these models represent far more realistic predictions than what exists in the HPMS analytical process. The predictive models and adjustment factors can also be improved over time if additional data are added to the HPMS.

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Dynamic Decision Model for Pavement Management Using Mechanistic Pavement Performance Submodel

K. H. CHUA, C. L. MONISMITH, AND K. C. CRANDALL

A dynamic decision model for pavement management has been developed on the basis of a dynamic programming formulation. The transition probabilities are determined by a mechanistic pavement performance model formulated within a stochastic framework. In this way, the individual distress modes may be modeled in the pavement condition states, which can be helpful in identifying the proper rehabilitation treatment. Furthermore, the Markovian assumption that the transition probabilities are time-invariant is no longer necessary with the proposed methodology. A numerical illustration demonstrates that the impact of variations in excess user and highway agency costs and other management decisions on the optimal rehabilitation policy can be evaluated explicitly.

To meet the challenges posed by an aging pavement network and the problem of funding, a pavement management system (PMS) is necessary to determine the most cost-effective strategy for rehabilitating the network while sustaining a level of pavement performance for the users. A rehabilitation alternative may be more costly in the initial capital outlay, but it may perform better in terms of its life, needing fewer remedial actions and lower associated costs to the user and agency. The PMS should be able to provide the economic trade-offs between alternatives in terms of life-cycle costs.

In recent years, highway agencies have developed and implemented several PMSs, including the PAVER, PARS, WSPMS, RAMS, OPAC, CALTRANS PMS, and HDM III (*1–7*). Some use the present condition approach, wherein the structural and serviceability condition of the network are first evaluated by means of a condition survey of various distress indicators. The rehabilitation that best restores the deficiency in each pavement segment is identified. No life-cycle cost comparisons of the alternatives, however, are considered, with the result that the selected strategy may not be the most cost-effective. In this case, funds are usually allocated using a priority list based on highway use and condition of pavement. Those projects outside the available budget will be deferred to the next period for consideration.

Where life-cycle cost comparisons are available, the predictive models for pavement performance either are deterministic or consider only the mean value performance. Such systems use a static or open-loop decision process, since the analysis is based on projected performance derived from the current situation. The analysis yields a sequence of rehabilitation activities that minimizes an objective function without considering that the pavement may perform better or worse than predicted. The best strategy derived in this way is not the optimal.

Instead, a dynamic programming approach for PMS is presented herein. It takes into account the actual pavement performance at each stage and yields a rehabilitation policy that is most cost-effective for each pavement segment for the period of the planning horizon subjected to management policies and operation constraints. Other dynamic decision models have been formulated on the basis of the Markovian process, in which the transition probabilities are dependent only on the state of the pavement and independent of the history, thus time-invariant (*8,9*). However, in the present approach, such an assumption has not been made. Furthermore, a stochastic mechanistic pavement performance model provides the framework for determining the transition probabilities, unlike the time-variant transition probabilities based on the pavement condition index (PCI) that are employed in PAVER (*10,11*). A numerical illustration is also included to demonstrate its potential and implementation.

DYNAMIC PROGRAMMING OPTIMIZATION

At the beginning of a discrete time period, formally called a stage, a pavement section is said to be in one of a set of possible pavement conditions Z, also called states. These states are determined from condition surveys conducted at the beginning of each stage. At each stage, after observing the state, a rehabilitative activity λ from a set Λ of all possible actions is chosen and implemented. From the state at that time and the action chosen, an expected cost is incurred. Because of the stochastic nature of the problem, the state of this pavement at the next stage is not known deterministically; instead, there is a probability distribution for the transition. The objective for the dynamic program model is to determine a rehabilitation policy that minimizes the expected value of the sum of the costs incurred over the planning horizon. This is achieved through recursive relations based on the principle of optimality (*12*).

Consider a planning horizon of n stages and an objective value function

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$V_k(i_1)$ = minimum expected value of sum of costs incurred for remaining planning process given that at beginning of stage k, pavement is at state i_1 (1)

Suppose at stage k, the pavement is in state i_1 and the optimal policy is desired. If an action $\lambda' \in \Lambda$ is chosen at stage k, then cost $C(i_1,\lambda')$ is incurred and the next state will be i_2 with transition probability $P(i_1,i_2,\lambda')$. If the next state is i_2, then the problem becomes equivalent to one that starts in state i_2 at stage $k + 1$. Hence, with λ' chosen, the least expected cost at stage k is

$$C(i,\lambda') + \sum_{i_2} P(i_1,i_2,\lambda') \cdot V_{k+1}(i_2)$$

Thus, the least cost at stage k obtained without restricting the decision to λ' is given by

$$V_k(i_1) = \min_{\lambda \in \Lambda} \left[C(i_1,\lambda) + \sum_{i_2} P(i_1,i_2,\lambda) \cdot V_{k+1}(i_2) \right] \quad (2)$$

and the optimal decision is the action λ that yields the minimum in Equation 2, also known as the optimality equation.

The optimality equation provides the mechanism for recursively determining the value of the objective function at the start of the planning horizon beginning with the values at the boundary. Starting with boundary conditions $V_n(i_2)$, the objective value at stage $k = n - 1$, $V_{n-1}(i_1)$, is found according to Equation 2. Then with $k + 1 = n - 1$ in the equation, the objective value at stage $k = n - 2$, $V_{n-2}(i_1)$, is derived from $V_{n-1}(i_2)$ determined from the previous step. In this recursive fashion, the objective value at the start of the planning horizon $V_0(i_2)$ is eventually determined with $k = 0$. The set of optimal decisions for each state at each stage form the optimal rehabilitation policy for the problem.

States Classification

The pavement states are characterized by pavement features that will affect the rehabilitation costs and transition probabilities from stage to stage. Features that are also determinants of the costs or transition probabilities but invariant with respect to the rehabilitation decisions and stages (such as pavement width) are not included in the states classification. Accordingly, two pavement features—namely, pavement distress condition and pavement structure—are identified for the classification of pavement states.

Pavement Distress Condition

The proposed methodology measures pavement performance in terms of individual modes of distress, in contrast with systems that use a composite index that combines the individual distresses, such as the PCI in PAVER and PCR in WSPMS. In this way, the individual defects are not masked so that the rehabilitation alternative that can best correct the deficiency can be prescribed.

The level of distress for each distress mode in the pavement is discretized into a set of collectively exhaustive, mutually exclusive bounds corresponding to varying degrees of damage. Accordingly, the pavement distress condition for m distress modes can be described by a vector $(d_1, d_2, \ldots, d_m)$, where d_1 is the distress level for the first distress mode and d_m is the distress level for the mth distress mode. The greater the number of levels, the finer will be the discretization and the more accurate (but computationally more difficult) will be the optimization model. For demonstration purposes, a three-level discretization of the fatigue distress mode is shown in Table 1, characterized by a damage index according to some pavement performance model that is described later.

Pavement Structure

The pavement structure is adequately characterized when all relevant changes to the structure are known. These changes are recorded by a vector $(n_0, n_1, \ldots, n_q)$ corresponding to a sequence of q modifications to the structure. Considering only routine maintenance and overlay alternatives for the present illustration, one definition for the components $n_1, n_2, \ldots, n_q$ of the pavement structure vector is given in Table 2. In this case only the overlay alternatives will modify the pavement structurally, and only these decisions are recorded in the vector. A number of routine maintenance type activities may be performed on the pavement between these overlays. These do not modify the pavement structurally and hence are not recorded in the vector.

The first element in the pavement structure vector, n_0, denotes the number of underlying cracked asphalt layers. Thus, for a structure with q overlays, n_0 can take values from 0, 1, $\ldots$, q. For example, a pavement structure described by $(1,\lambda_{V1},\lambda_{V3},\lambda_{V2})$ will comprise three overlays—namely, thin

TABLE 1 Damage-Level Discretization for Fatigue Distress Mode

Damage Level	Description	Damage Index
D1	> 45% cracking (severe)	> 1.0
D2	10% - 45% cracking (intermediate)	0.72 - 1.0
D3	< 10% cracking (minimal)	0.0 - 0.72

TABLE 2 Rehabilitation Alternatives

Rehabilitation Type	Activity	Designation
Routine Maintenance	Do Nothing	λ_{O1}
	Patching	λ_{O2}
Overlay	Thin	λ_{V1}
	Medium	λ_{V2}
	Thick	λ_{V3}

Examples:
Pavement structure after sequence of rehabilitation activities
λ_{O1} λ_{V1} λ_{O2} λ_{V2} -- (V1,V2)
λ_{V1} λ_{O1} λ_{O2} λ_{V2} λ_{O1} -- (V1,V2)
λ_{V1} λ_{V2} λ_{O2} λ_{V3} -- (V1,V2,V3)

followed by thick and medium overlays—with one cracked layer, that is, the original asphalt layer.

Objective Function and Optimality Equation

It is usual and appropriate to consider the rehabilitation policy with the lowest expected net present cost for the planning horizon to be the most efficient allocation of scarce rehabilitation funds while satisfying management constraints. Accordingly, the objective function can be defined as

$V_k(\mathbf{i},\mathbf{j})$ = minimum expected net present cost from start of year k to end of planning horizon given that distress condition vector is $\mathbf{i}$ and pavement structure vector is $\mathbf{j}$; $\mathbf{i} = (d_f)$ and $\mathbf{j} = (n_0, n_1, n_2, \ldots, n_q)$, $n_0 \leq q$ modifications to pavement structure (3)

where d_f is the distress level for fatigue mode.

On the basis of this objective function, the optimality equations for "not failed" conditions are defined as follows:

$$V_k(\mathbf{i},\mathbf{j}) = \min \begin{cases} \lambda_{V1}: & r_1 \cdot C_a(\mathbf{i},\lambda_{V1}) + r_2 \cdot C_u(\mathbf{i},\lambda_{V1}) \\ & + \sum_{\mathbf{i}_2} \left\{ P(\mathbf{ij},\mathbf{i}_2\mathbf{j}_2,\lambda_{V1},k,y) \cdot \left[\frac{r_3 \cdot y \cdot U(\mathbf{i},\mathbf{i}_2)}{(1+r)^{y/2}} + \frac{V_{k+y}(\mathbf{i}_2,\mathbf{j}_2)}{(1+r)^y} \right] \right\} \\ \vdots & \\ \lambda_{Vp}: & r_1 \cdot C_a(\mathbf{i},\lambda_{Vp}) + r_2 \cdot C_u(\mathbf{i},\lambda_{Vp}) \\ & + \sum_{\mathbf{i}_2} \left\{ P(\mathbf{ij},\mathbf{i}_2\mathbf{j}_2,\lambda_{Vp},k,y) \cdot \left[\frac{r_3 \cdot y \cdot U(\mathbf{i},\mathbf{i}_2)}{(1+r)^{y/2}} + \frac{V_{k+y}(\mathbf{i}_2,\mathbf{j}_2)}{(1+r)^y} \right] \right\} \\ \lambda_{O1}: & r_1 \cdot C_a(\mathbf{i},\lambda_{O1}) + r_2 \cdot C_u(\mathbf{i},\lambda_{O1}) \\ & + \sum_{\mathbf{i}_2} \left\{ P(\mathbf{ij},\mathbf{i}_2\mathbf{j}_2,\lambda_{O1},k,y) \cdot \left[\frac{r_3 \cdot y \cdot U(\mathbf{i},\mathbf{i}_2)}{(1+r)^{y/2}} + \frac{V_{k+y}(\mathbf{i}_2,\mathbf{j}_2)}{(1+r)^y} \right] \right\} \\ \vdots & \\ \lambda_{Oq}: & r_1 \cdot C_a(\mathbf{i},\lambda_{Oq}) + r_2 \cdot C_u(\mathbf{i},\lambda_{Oq}) \\ & + \sum_{\mathbf{i}_2} \left\{ P(\mathbf{ij},\mathbf{i}_2\mathbf{j}_2,\lambda_{Oq},k,y) \cdot \left[\frac{r_3 \cdot y \cdot U(\mathbf{i},\mathbf{j}_2)}{(1+r)^{y/2}} + \frac{V_{k+y}(\mathbf{i}_2,\mathbf{j}_2)}{(1+r)^y} \right] \right\} \end{cases}$$

$$\text{for } k = 0, 1, \ldots, H-1 \qquad (4)$$

where

y = number of stages for transition, defined by $y = \begin{cases} y_\lambda & \text{if } k + y_\lambda \leq H \\ H - k & \text{if } k + y_\lambda > H \end{cases}$

H = number of stages in planning horizon (years);

y_λ = minimum life of rehabilitation activity λ, during which nothing should be done to pavement section;

$\lambda_{V1},\lambda_{V2}, \ldots, \lambda_{Vp}$ = elements of Λ_V, set of overlay alternatives considered;

$\lambda_{O1},\lambda_{O2}, \ldots, \lambda_{Oq}$ = elements of Λ_O, set of routine maintenance alternatives considered;

$C_a(\mathbf{i},\lambda)$ = agency cost for implementing alternative λ at distress condition $\mathbf{i}$;

$C_u(\mathbf{i},\lambda)$ = cost to users as a result of ongoing rehabilitation activity at distress condition $\mathbf{i}$;

$U(\mathbf{i},\mathbf{i}_2)$ = average annual excess cost to users as a result of pavement condition going from $\mathbf{i}$ to $\mathbf{i}_2$;

r = discount rate;

r_1, r_2, r_3 = parametric weightings of costs to agency and users;

$P(\mathbf{ij},\mathbf{i}_2\mathbf{j}_2,\lambda,k,y)$ = transition probability for rehabilitation activity λ, from pavement state $\mathbf{ij}$ at stage k to pavement state $\mathbf{i}_2\mathbf{j}_2$ at y stages ahead; and

$\mathbf{i}_2,\mathbf{j}_2$ = new condition and pavement structure states as a result of rehabilitation.

Accordingly, with λ_V, $\mathbf{j}_2$ is defined as $\mathbf{j}_2 = (n_0, n_1, \ldots, n_m, \lambda_V)$; with λ_O, $\mathbf{j}_2$ remains unchanged.

In the case of "failed" pavement sections, only overlay alternatives are considered in Equation 4. In most situations it is reasonable to assume that the maximum tensile strain occurs at the bottom of the uncracked asphalt-bound layer. The fatigue cracks generated then propagate to the surface. Thus, when the pavement becomes severely cracked, $\mathbf{j}_2$ is defined by $\mathbf{j}_2 = (m + 1, n_1, \ldots, n_m)$.

The optimal objective function is computed as a parametric sum of both agency and excess user costs so that the respective cost components can be weighted differently as judged by the management using cost parameters r_1, r_2, and r_3. In Equation 4, $C_a(\mathbf{i},\lambda)$ and $C_u(\mathbf{i},\lambda)$ are assumed to be incurred at the beginning of the stage, which can be easily modified to account for any time delay. In the case of excess user costs $U(\mathbf{i},\mathbf{i}_2)$ associated with pavement condition, the average of the excess user costs associated with the pavement conditions at the beginning of stage k and those at the beginning of stage $(k + y)$ is taken to be representative. Thus, these costs have been factored by the discount factor for $y/2$ years.

Boundary Conditions

The boundary conditions at the end of the planning horizon should reflect the long-term expected costs obtained by following an optimal rehabilitation policy from that time on. These costs are not easily established, requiring extensive analyses for a wide range of initial pavement conditions and structures. Instead, it is usual to characterize the value of the pavement section at the end of the cycle by a salvage value (*13*). These are assessed as negative costs according to

$$V_H(\mathbf{i},\mathbf{j}) = -S(\mathbf{i},\mathbf{j}) = -\left(\frac{L_R}{L_O}\right)E_k \qquad (5)$$

where $S(\mathbf{i},\mathbf{j})$ is the salvage value at the end of the planning horizon, determined as a simple proportion of the cost of the last overlay, E_k, according to the ratio of the remaining life, L_R, to the expected life of the overlay, L_O.

With this model, a pavement section with a lower amount of distress and longer mean life to failure at the end of the planning horizon will have a higher salvage value and may have a lower long-term expected cost.

TRANSITION PROBABILITIES

Mechanistic Pavement Performance Model

The transition probabilities are determined within a framework of a mechanistic pavement performance model of Chua et al. (*14*). The pavement section is first analyzed to determine the controlling structural response that governs the extent of damage for each distress mode. For the present purpose, a multilayered elastic analysis is adequate. The remaining life of the pavement before the distress exceeds prescribed levels can then be determined from the distress submodels.

For fatigue cracking, the maximum tensile strain in the asphalt bound layer is the controlling structural response, and the criteria adopted for fatigue cracking in the distress submodel are similar to those obtained from the AASHO Road Test (*15*). Accordingly, the allowable number of load repetitions at maximum tensile strain ε_t can be expressed as

$$N_{f45} = 18.4C\left[4.325 \times 10^{-3}\varepsilon_t^{-3.291}\left|\frac{E^*}{6.89}\right|^{-0.854}\right] \quad (6)$$

$$N_{f10} = 13.3C\left[4.325 \times 10^{-3}\varepsilon_t^{-3.291}\left|\frac{E^*}{6.89}\right|^{-0.854}\right] \quad (7)$$

where

N_{45}, N_{10} = allowable number of repetitions for 45 and 10 percent cracking, respectively;
$|E^*|$ = dynamic asphalt mixture modulus (kPa); and
C = factor accounting for asphalt content and degree of compaction suggested by Pell and Cooper (*16*).

The accumulated damage caused by a range of strain levels due to varying applied load and temperature changes (which affect the stiffness of the asphalt bound layer) is determined by the linear summation of cycle ratios (*17*) as

$$\sum_i \left(\frac{n_i}{N_{wi}}\right) \leq 1 \quad (8)$$

where n_i is the number of load applications at tensile strain ε_t and N_{wi}, the corresponding allowable number of load applications for w percent cracking. When the sum reaches unity, w percent cracking in the pavement—in particular, 45 or 10 percent cracking corresponding to Equations 6 and 7, respectively—is deemed to have taken place.

Stochastic Framework

Generally, a distress state such as depicted in Table 1 is characterized by two damage indexes: L_{w_j} and L_{w_k}, representing the lower and upper bounds, respectively. For a failed section (severely cracked), only a lower bound applies. The damage index is defined so that a value of unity denotes 45 percent fatigue cracking, whereas a value of 0.72 denotes 10 percent cracking derived from the ratio of N_{10}/N_{45}. For w percent cracking, the damage index L_w is determined by the ratio N_w/N_{45}.

Consider a limit-state function $g_{wT}(\mathbf{x})$ such that

$$g_{wT}(\mathbf{x}) = L_w - \sum_{t=1}^{T}\sum_i \left(\frac{n_{it}}{N_{45i}}\right) \quad (9)$$

where

$\mathbf{x}$ = vector of input variables in mechanistic model;
n_{it} = actual number of load applications at tensile strain ε_i in tth year;
N_{45i} = allowable number of load applications at tensile strain ε_i, corresponding to 45 percent cracking; and
L_w = damage index corresponding to w percent extent of fatigue cracking.

If $g_{wT}(\mathbf{x}) < 0$, the linear sum of cycle ratios after T years of load applications would have exceeded the damage index, and fatigue cracking would have exceeded w percent extent of pavement surface. Thus, the probability of transition into a distress state bounded by L_{w_j} and L_{w_k} would be given by

$$P_F = P[g_{w_jT}(\mathbf{x}) < 0] - P[g_{w_kT}(\mathbf{x}) < 0] \quad (10)$$

For a pavement with initial condition L_{w_i}, the limit-state function should be modified to

$$g_{wT}(\mathbf{x}) = L_w - \sum_{t=1}^{T}\sum_i \left(\frac{n_{it}}{N_{45i}}\right) - L_{w_i} \quad (11)$$

Each term in the right-hand side of Equation 10 is evaluated as

$$P[g_{wT}(\mathbf{x}) \leq 0] = \int_{g_{wT}(\mathbf{x}) \leq 0} f_{\mathbf{x}}(\mathbf{x})d\mathbf{x} \quad (12)$$

where $f_{\mathbf{x}}(\mathbf{x})$ is the joint probability density of variables $\mathbf{x}_1$, $\mathbf{x}_2$, . . . , $\mathbf{x}_n$. The exact solution to Equation 12 involves an n-fold integration of the joint probability density of the n basic variables. In general, numerical integration is necessary since an analytical evaluation is possible only in a few special cases. However, it becomes computationally difficult when the number of variables is more than just a few. Instead, the solution is determined by full-distribution reliability methods (*14,18*).

MANAGEMENT DECISION PROCESS

The decision process for pavement management is shown schematically in Figure 1. The inputs to the optimization model include the transition probabilities derived from the mechanistic distress submodels, a matrix of agency and user excess costs, and salvage values derived via a cost submodel in StoMe, a stochastic mechanistic PMS (*19*), and a set of management policies. The management policies comprise a set of rehabilitation alternatives and their minimum lives, y_λ of Equation 4; a rehabilitation-condition policy that determines the range of alternatives feasible for each pavement condition (e.g., overlay when failed and routine maintenance/overlay otherwise); a performance criterion that defines the minimum reliability of the rehabilitation activity or, conversely, the maximum probability of transition to failure; the cost weightings of Equation 4; and the discount rate.

The result from the optimization is the optimal strategy for each pavement section along with the associated expected annual performance and capital outlay. The expected performance gives the probability that the pavement section can be found in the various states at the beginning of each year in the planning horizon, and the expected capital outlay gives the expected cost of rehabilitation to be expended by the agency at the start of each year, expressed in present money value. Depending on the actual performance of the pavement section, the true capital expenditure can be higher or lower than the expected. Since all costs are expressed in dollars per lane mile, the sum of the expected annual capital outlay weighted by the lane miles of the section will yield the expected yearly budget requirements to implement the optimal policy for the network. The policy is optimal in the sense of least total expected cost discounted to present value terms.

The sum of the expected performance of each pavement section weighted by the lane miles will yield the expected fraction of the network that will be found in the various pavement condition states at the beginning of each year through the planning horizon. In this way, management can predict the expected fraction of the network that will manifest a specified physical distress indicative of poor ride quality to the users.

NUMERICAL EXAMPLE

Data

The pavement section used in the present illustration is a three-layered system comprising a 7-in. asphalt concrete layer over a 12-in. granular base layer underlain by subgrade. The properties of the pavement layers, environment, and traffic variables are given in Table 3. The table also shows the distribution type and variation in the variables. Most of the distributions are similar to those obtained from available data (*20,21*).

For simplicity, a single category of truck traffic and a single seasonal variation in air temperature, base, and subgrade stiffnesses have been assumed, although the stochastic model is capable of handling multiple categories of truck traffic and seasonal variations. The effect of increasing the number of random variables in the data set to account for more general applications will only mean more computer processing time for each analysis.

Management Inputs

The rehabilitation alternatives considered in the present study include routine maintenance, 2-in. asphalt concrete overlay (thin, λ_{V1}), and 4-in. asphalt concrete overlay (medium, λ_{V2}). The rehabilitation-condition policy adopted will require an overlay when the pavement is failed (i.e., $D1$), and either an overlay or routine maintenance otherwise. Furthermore, a minimum life of 7 years is imposed on overlays so that no further overlay is permitted within this time after an overlay

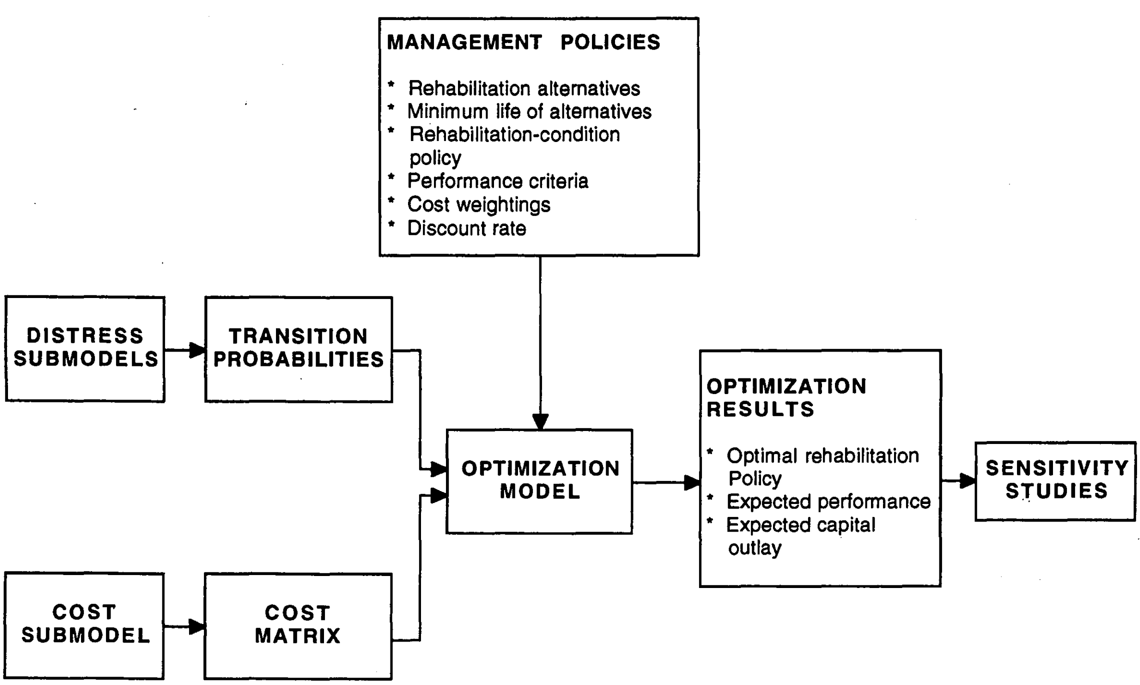

FIGURE 1 Management decision process.

TABLE 3 Distribution of Variables

Variable Description	Distribution Type	Mean	Standard Deviation
Mean monthly temperature	Weibull	14.56°C	4.21°C
Annual traffic volume	Lognormal	12,000,000	504,000
Annual growth rate	Lognormal	4.5%	1.0%
Axle load[c]	Shifted Exponential	75.65 kN	13.35 kN[a]
Tire pressure[c]	Normal	620 kPa	103.4 kPa
Truck percentage	Lognormal	10.0%	0.1%
Axle factor	Normal	3.5	0.175
Wheel distance	Deterministic	450 mm	-
Lane factor	Deterministic	1.0	-
Loading time	Deterministic	0.02 sec	-
Asphalt content	Uniform[b]	10.0%	11.0%
Volume of air voids	Uniform[b]	4.0%	5.0%
Penetration index	Deterministic	0.0	-
Ring & Ball Softening Point	Deterministic	59°F	-
Poisson's ratio (asphalt)	Deterministic	0.3	-
Base stiffness	Lognormal	206.7 MPa	30.94 MPa
Subgrade stiffness	Lognormal	68.9 MPa	10.27 MPa
Poisson's ratio (base)	Deterministic	0.35	-
Poisson's ratio (subgrade)	Deterministic	0.40	-
Base thickness[d]	Lognormal	300 mm	20 mm
Asphalt concrete thickness[d]	Lognormal	175 mm	10 mm
Overlay thickness	Lognormal	as specified	10 mm

Notes:
a) Shift in Exponential distribution
b) Lower and upper limits for uniform distribution indicated in the mean and standard deviation columns, respectively
c) Axle load and tire pressure are positively correlated with coefficient 0.85
d) Base and asphalt concrete thicknesses are negatively correlated with coefficient -0.85

is laid. The analysis is performed over a planning horizon of 14 years at a 4 percent discount rate.

The costs to the agency for the various alternatives are presented in Table 4. It is expected that preparation costs will increase with deteriorating condition of the pavement. Furthermore, with a thicker overlay less preparation will be necessary since the minor defects can be ignored. The placement costs are a function of the volume of asphalt needed for the overlay. The corresponding excess user costs associated with the rehabilitation works are also included in the table for day- and night-time lane closures. In essence, these costs are contributed by the increased time and fuel expended in the queue at the lane closure (*19*). For the present study, r_1 and r_2 are unity in value. The excess user cost due to deteriorating pavement condition has not been included in the study for lack of quantification.

Analysis

Transition Probabilities

The 1-year transition probabilities to failed states for routine maintenance from not-failed states are shown in Figure 2 for Years 0 to 13. It can be seen that the transition probabilities increase by up to 34 percent in that period. An increase by up to 46 percent is evident in the case of 8-year transition probabilities for thin overlays also shown in the figure. The increases can be attributed to the annual growth in the traffic volume. Thus, the assumption of a constant transition matrix in the Markov decision process is invalid when traffic growth is considered.

Optimal Rehabilitation Policy

Without any rehabilitation treatment, it is expected that the condition of the pavement section will deteriorate with time as demonstrated in Figure 3, which depicts the probability that the pavement is in the not-failed state. By the end of the planning period, the probability of not-failed has fallen below 50 percent. With a rehabilitation strategy, the rapid deterioration of the pavement section is impeded when the pavement structure is restored and strengthened by the asphalt concrete overlays. The probability of the not-failed condition is maintained above 78 percent throughout the planning period, averaging about 81 percent.

The optimal policy is to provide a thin overlay whenever the pavement fails and routine maintenance otherwise. The expected capital outlays required at the end of each year to implement the optimal policy are shown in Figure 4. At the end of Year 0, the capital outlay is only $95/lane-mi for routine maintenance. Subsequently, the actual expenditure will depend on the actual condition of the pavement at the end of that year.

Effect of Time of Lane Closures

The excess user costs are higher during daytime lane closures because of increased delays and more fuel consumed as a

TABLE 4 Agency and User Excess Costs

		Condition States		
		D1	D2	D3
Agency Costs				
Preparation	Routine Maintenance	$0	$1240	$95
	Thin Overlay	$3280	$1540	$955
	Medium Overlay	$1910	$1195	$955
Placement	Routine Maintenance	$0	$0	$0
	Thin Overlay	$29850	$29850	$29850
	Medium Overlay	$59700	$59700	$59700
Excess User Costs				
0% Day-time Closure	Routine Maintenance	$0	$72	$48
	Thin Overlay	$95	$95	$95
	Medium Overlay	$190	$190	$190
100% Day-time Closure	Routine Maintenance	$0	$15990	$10660
	Thin Overlay	$21320	$21320	$21320
	Medium Overlay	$42640	$42640	$42640

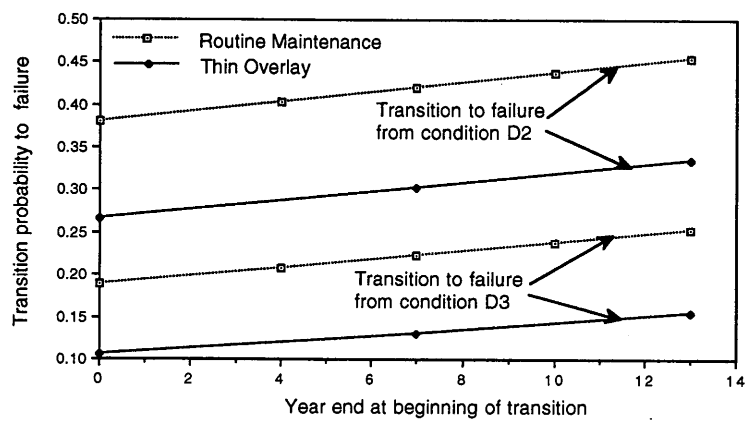

FIGURE 2 Variation of transition probabilities with time.

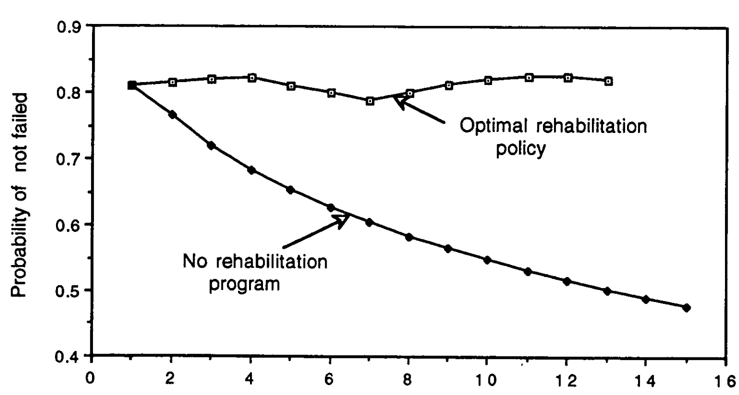

FIGURE 3 Effect of rehabilitation policy on pavement condition.

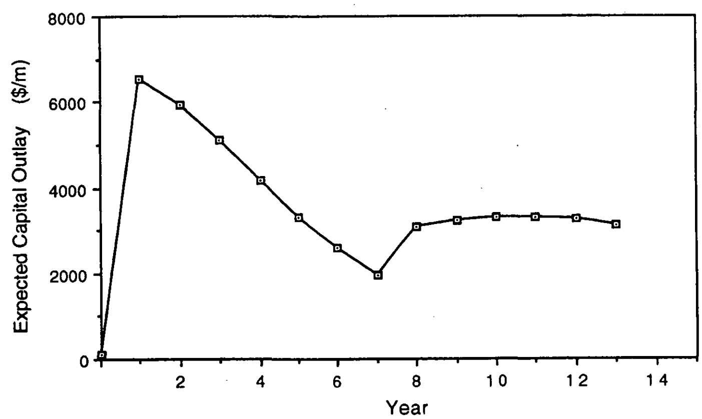

FIGURE 4 Expected capital outlay for optimal rehabilitation policy.

result of longer queues due to heavier daytime traffic. For demonstration purposes, the excess user costs for various proportions of day- and nighttime construction are obtained simply by direct proportioning from the values indicated in Table 4.

The optimal policy with the increasing proportion of daytime closures is no longer overlay only when failed. For example, the optimal policy for 40 percent daytime closure is shown in Table 5 as a function of pavement condition and structure. The effect of increasing user costs is summarized in Table 6, which gives the number of not-failed condition states that require thin overlays. Essentially, overlays are preferable to reduce the number of rehabilitation activities to keep down excess user costs as they become more and more substantial compared with total rehabilitation costs, especially with routine maintenance.

Effect of Discount Rates

There have been controversies over the rate of discounting that should be applied for the economic evaluation of public projects. A real rate of 4 percent, reflecting the yield for long-term government bonds, has been used in this study (*22*). As the discount rate is increased, the more-expensive rehabilitation alternative is deferred in favor of cheaper alternatives. Thus, routine maintenance tends to be favored to postpone the cost of overlays, as demonstrated in Table 7, which shows the number of not-failed condition states requiring overlays decreasing with increasing discount rate for the case of 40 percent daytime lane closure.

At a 0 percent discount rate, thin overlays are optimal for some not-failed condition states at the early years, even in Year 1. At 6 percent upwards, thin overlays for not-failed condition states are deferred until after Year 7. Eventually, at 15 percent and up, thin overlays are restricted to the failed states.

Modified Rehabilitation Policies

As seen in the preceding sections, a rehabilitation-condition policy that stipulates overlays only when failed can be simplistic and may not be the optimal in many situations. The result is increased costs, as shown in Figure 5, when compared

TABLE 5 Optimal Policy for 40 percent Daytime Lane Closures

Year End	Pavement Structure	Alternative at Condition State D1	D2	D3	Year End	Pavement Structure	Alternative at Condition State D1	D2	D3
0	(0)	2	2	1	1-6	(0)	2	1	1
7	(0)	2	2	2	8	(0)	2	2	2
	(0,V1)	2	2	2		(0,V1)	2	2	1
	(0,V2)	2	2	1		(0,V2)	2	2	1
	(1,V1)	2	2	1		(1,V1)	2	2	1
	(1,V2)	2	2	2		(1,V2)	2	1	1
9	(0)	2	1	2	10-13	(0)	2	1	1
	(0,V1)	2	1	1		(0,V1)	2	1	1
	(0,V2)	2	1	1		(0,V2)	2	1	1
	(1,V1)	2	1	1		(1,V1)	2	1	1
	(1,V2)	2	1	1		(1,V2)	2	1	1

Notes: 1 denotes routine maintenance
2 denotes thin overlay

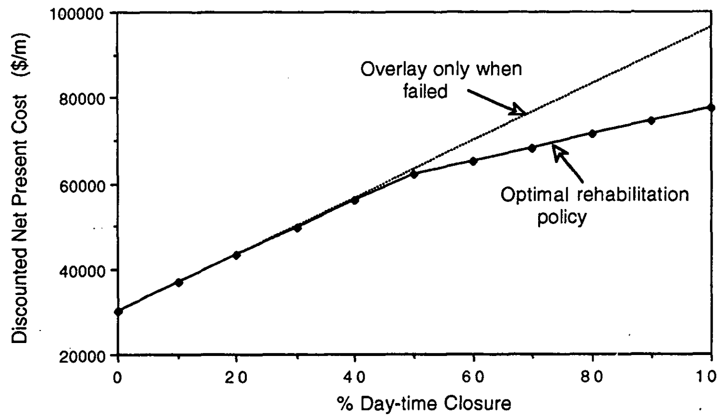

FIGURE 5 Effect of nonoptimal policy.

TABLE 6 Effect of Time of Lane Closures

Proportion of Day-time closures (%)	Discounted net present cost ($/lane-mile)	Number of not-failed states requiring overlay
0	30,130	0
10	36,750	0
20	43,370	0
30	49,830	4
40	56,170	15
50	62,100	22
100	77,380	44

TABLE 7 Effect of Discount Rates

Discount rate (%)	Discounted net present cost ($/lane-mile)	Number of not-failed states requiring overlay
0	62,330	19
4	56,170	15
6	52,450	11
10	46,060	4
15	39,650	0
20	34,740	0

with the optimal. Discounted net present cost departures from the optimal are more when excess user costs become more substantial, up to 25 percent higher than the optimal for the 100 percent daytime lane closure. The optimal strategy derived from the dynamic programming decision process can be very different from any heuristic-type rehabilitation-condition policy. In fact, a more restrictive rehabilitation-condition policy always incurs a discounted net present cost that is never less than the optimal unrestricted case.

CONCLUSIONS

The optimization is formulated in a dynamic programming model that incorporates a closed-loop decision process. The optimal policy obtained from the model always yields a discounted net present cost that is lower than any heuristic type rehabilitation-condition policy. The optimal policy will also be more cost-effective than one with imposed overlays regardless of pavement condition, which is characteristic of the open-loop decision process that most PMSs use.

The formulation of the states classification in the optimization model permits the individual modes of distress to be modeled without masking the individual defects by a composite index of pavement performance. It also provides the framework for using stochastic mechanistic distress submodels to derive the transition probabilities. In this way, the Markovian time-invariant assumption used in some PMSs is no longer necessary. In fact, for a mean of 3 percent annual growth in traffic, the results in the present study have shown that the transition probabilities do change, and by as much as 46 percent over the planning period.

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State Increment Optimization Methodology for Network-Level Pavement Management

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An optimization methodology is presented for program planning and budget allocation involved in network-level pavement management. It consists of three major components: (a) characterization of pavement condition into discrete states, (b) specification of treatment options for each pavement state, and (c) application of a linear programming technique for constrained optimization and development of the multiyear pavement program. Pavement sections having similar characteristics are classified into states that are defined on the basis of distress and nondistress factors. The network condition is represented as lane miles of pavement distributed among various states. Several treatment options are specified for each state; they are based on the information incorporated in the state definition. For each treatment applied, the time for a complete state transition (or increment) to occur is predicted from historical data and empirical knowledge. A linear program is formulated to model interactions between economic and engineering factors in an effective manner. It enables decisions about the type of treatment, timing, and magnitude of work to be made simultaneously. Both project- and network-level constraints can be imposed to develop a pavement program that meets specified requirements on condition and budget. The developed methodology has been implemented as part of the New York State Thruway Authority's pavement management system. An example is presented to illustrate the methodology and its usefulness to conduct variational analysis. It is concluded that the methodology can be applied to develop an effective multiyear pavement program and ensure optimal budget allocation for the entire network.

An important objective of a pavement management system (PMS) is to develop an optimization methodology useful for conducting pavement program analysis (*1*). The methodology should aim to provide a decision-making capability that enables highway managers to make rational, consistent, and cost-effective about the pavement network.

Most PMSs include specific methodologies for characterizing pavement condition, identifying treatment options, predicting condition, and evaluating the economics. A decision-making method (such as ranking or optimization) is necessary to integrate various aspects of these entities into a complete system useful for planning a pavement program. A review of some of the decision-making methods in a PMS was presented by Ravirala (*2*).

Presented is an optimization methodology that resolves the multiyear planning and budget allocation problem into two subproblems: a project-level planning problem, and a network-level constrained optimization problem. Specific objectives of the study are to

- Characterize the pavement condition using definitive parameters that quantify the distress and nondistress factors affecting the decision process,
- Develop an optimization methodology for multiyear pavement program planning, and
- Illustrate the developed methodology by applying it to analyze the New York State Thruway Authority's (NYSTA's) pavement network.

The project-level planning involves a series of tasks such as condition characterization, treatment options identification, prediction, and cost estimation. Pavement condition is characterized by defining states on the basis of distress and nondistress factors. Several treatment options are associated with each state. The consequences of applying a treatment are specified by predicting the time period over which a state transition would occur. The network optimization problem is formulated as a linear program that has cost minimization as the objective. Constraints related to network condition and annual maintenance budget are also specified. The linear program determines the lane miles of pavement in each state that should receive each of the possible treatment options.

The optimization methodology is illustrated by analyzing a portion of the NYSTA network. The influence of maintenance in prolonging pavement life as well as improving the ride quality is analyzed for 5- and 10-year periods. The changes in the multiyear pavement program are observed for a fixed annual budget, both with and without condition constraints.

CONDITION CHARACTERIZATION

Approach

Pavement condition can be characterized by classifying lane miles of sections into one of many discrete states. A state is a combination of specific levels of the variables (called the state variables) that completely describe the pavement condition. The values taken by each continuous state variable are generally divided into ranges. Each range is called a condition level for that measure. Such an approach to condition characterization was previously used in PMSs of other agencies (*3–6*).

States serve two important functions: (a) predicting condition through a state transition process, and (b) balancing

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the pavement program through proper distribution of the network among states. Pavement state transition is an event that describes the change of state (value of at least one of the state variables) for the pavement as a consequence of action or deterioration over time. The time over which the transition occurs is called transition time. The transition time corresponding to an action is a variable depending on the rate of deterioration after applying the treatment option. The pavement will transition to a specified state as a consequence of the action.

A balanced pavement program can be developed by establishing desirable management policies on the future condition of pavement network. This is achieved by grouping the states into broad classes such as good, fair, and poor. Threshold values are specified on the quantity of pavement that may belong to each class. For example, management may desire to maintain a certain percentage of the network in good states while limiting or gradually upgrading the poor-condition pavement. Thresholds can be specified for each year in the analysis to develop a maintenance and capital program that satisfies the management requirements.

Definition of Pavement States

Pavement states are defined according to three state variables, namely, pavement type, traffic volume, and distress measures. The pavement type parameter enables differentiation between types of distress and specification of appropriate treatments for increased accuracy of network-level analysis. Average annual daily traffic (AADT) is classified into three levels to differentiate between sections with different levels of traffic. The distress condition is summarized by developing indexes for three measures: structural rating, slab/surface rating, and joint/crack rating. A weighted average approach to calculating condition indexes was developed by Grivas and Schultz (*7*). Distress measures contributing to each index and specific levels defined for the state variables are as summarized in Table 1. There are 270 possible states obtained from combinations of variables at each level.

PROBLEM FORMULATION

Model Description

The multiyear pavement program development and budget allocation is modeled as a modified minimal network-flow problem (*8*). Pavement states act as nodes that are connected by links that represent various treatment options. Lane miles of pavement that transit through the states would constitute the flow. The modeling process can be described by the following five-step procedure:

1. Classification of each nominal pavement section into one of many states for the initial time period,
2. Identification of treatment options that drive the pavement from one state to another over a period of time,
3. Estimation of treatment costs and other resource requirements,
4. Specification of management condition goals and budget constraints, and
5. Formulation of a linear program.

The linear program determines the lane miles of pavement in each state that should receive each of the possible treatment options.

Treatment Options Identification

Identification of treatment options on the basis of engineering considerations is an important project-level requirement. Each

TABLE 1 Components of NYSTA Pavement States

VARIABLE	MEASURE	LEVELS	CODE
Pavement Type	-	Overlaid Concrete	O C
Traffic	AADT (per lane)	< 15,000 15,000 - 30,000 > 30,000	L M H
Structural Rating	Concrete • Transverse Jt. Spalling • Transverse Jt. Faulting • Slab Cracking Overlaid • All lane distresses	0 - 20 21 - 40 41 - 60 61 - 80 80 - 100	E G F P B
Slab/Surface Rating	Concrete • Slab Surface Defects • Slab Cracking Overlaid • Surface Defects • Rutting	0 - 33 34 - 66 67 - 100	E G F
Joint/Crack Rating	Concrete • Joint distresses Overlaid • Crack distresses	0 - 33 34 - 66 67 - 100	E G F

Rating: E = Excellent G = Good F = Fair P = Poor B = Bad

L = Low M = Medium H = High

pavement state is assigned several maintenance, rehabilitation, and reconstruction (MR&R) actions. Subsequently, it is necessary to specify the consequences of performing an action (or deferring it) in order to evaluate the differences between the cost-effectiveness of MR&R actions. In particular, two issues need to be addressed through proper planning: reduced maintenance costs due to higher investment, and extended life of a pavement due to preventive maintenance.

A novel method of specifying a suitable MR&R action is in the form of a control. A control is a conjunction of treatments planned and the resulting change in pavement condition as a function of time. A complete control is defined by specifying both the type of action and the subsequent time for state transition. An action must be chosen after observation of the pavement state; on the basis of only the state at that time and the action chosen, the probability density function corresponding to the transition time required to arrive at a future state must be specified.

The process of identifying treatment options for each state is facilitated by answering two types of questions: What is the expected time to arrive at a future state if an action is taken (for example, a partial restoration such as slab replacement)? and, What level of maintenance (type, magnitude, and frequency) needs to be planned to achieve certain performance (in terms of state transition)?

The time interval over which a given control is applied varies with each control. The interval is determined as the time required for a state to change by a specified increment. That is, instead of planning MR&R actions as one-time actions, they are planned as controls that correspond to short-term plans (or sequence of treatments) that achieve a certain performance. This unique feature incorporates a planning and prediction process that relies not only on historical data but also on engineering expertise. This method of planning MR&R actions as controls that achieve a complete state transition is called state increment control.

Linear Program Formulation

The linear program expresses the network optimization problem as linear functions of the decision variables. In a general form it is formulated as

Decision variable:

y_{ijt} = lane miles of pavement in state i that should receive action j at time t

Objective:

Minimize

$$\sum_{t=1}^{T} \sum_{i \in I} \sum_{j \in J_i} w_{ijt} y_{ijt} \tag{1}$$

where

T = number of time intervals in planning horizon (years),
I = set of condition states for pavement,
J_i = set of all actions for pavement in state i, and
w_{ijt} = present worth of expected cost for pavement in state i with action j applied in year t.

Subject to

$$\sum_{j \in J_i} y_{ij1} = L_{i1} \qquad \text{for all } i \in I \tag{2}$$

where L_{i1} is the lane miles of pavement entering state i at initial time.

$$\sum_{k=1}^{t-1} \sum_{i \in T_d} \sum_{j \in J_{id}} P_{ijd}^{t-k} y_{ijk} = \sum_{j \in J_d} y_{djt} \qquad \text{for all } d \in I \text{ and } t = 2 \text{ to } T \tag{3}$$

where

I_d = set of states that have some action leading pavement to state d,
J_{id} = set of actions that have transitions from state i to state d,
J_d = set of all actions for pavement in state d, and
p_{ijd}^{t-k} = probability that pavement in state i would move to state d at time t after receiving action j at time k.

$$\sum_{k=1}^{t} \sum_{i \in I} \sum_{j \in J_i} c_{ijk}^{t} y_{ijt} \leq B_t \qquad \text{for all } t \tag{4}$$

where c_{ijk}^{t} is the cost of action j for a lane mile of pavement in state i at time k (the decision is made at time k, and cash flow occurs at time t), and B_t is the funding available at time t.

$$\sum_{k=1}^{t} \sum_{i \in I} \sum_{j \in J_i} a_{ijc}^{t-k} y_{ijk} \geq L(I_c, t) \qquad \text{for all } c \text{ and } t = 2 \text{ to } T \tag{5}$$

where

a_{ijc}^{t-k} = probability that pavement in state i after receiving action j at time k would at time t be in some state belonging to class c;
I_c = set of states classified under class c;
c = 1, 2, 3 for good, fair, and poor, respectively; and
$L(I_c,t)$ = threshold for lane miles of pavement to be in state I_c at time t.

$$y \geq 0 \tag{6}$$

(i.e., nonnegativity of all y's)

Equation 2 provides the input corresponding to the initial time condition of the network. Initial condition is determined as lane miles of pavement distributed in various states. The left-hand side of the equation represents, for a particular state, a summation of the total lane miles that can receive various actions in the first year. This is equated to the actual number of lane miles present in that state during the first year.

Equation 3 imposes network length conservation during transitions between states. The summation on the left side represents the total quantity of pavement entering a particular state and time. The network length is conserved by equating the left-hand term to a summation totaling all lane miles leaving the state.

Equation 4 imposes budget constraints for each year in the analysis period. The left side represents a summation of total expected treatment cost incurred during a year. This is constrained to be no more than the specified MR&R budget for that particular year.

The objective of Equation 5 is to group different states into good, fair, and poor and denote them by a class, I_c. This constraint controls the amount of pavement that is allowed in each class per year; it corresponds to management goals for overall network condition. The left side is a summation of total lane miles belonging to a particular class at a given time. This is constrained to be no less than certain lane miles of pavement desired to be in that class. The direction of inequality can be reversed for undesirable classes. This will constrain the total lane miles in undesirable classes (e.g., poor) to be no more than a specified amount.

Variable Coefficient Determination

The coefficients for variables in the objective function are determined as present-worth costs of planned treatment options. Figure 1 illustrates state transition and cash flow for a control action. The probabilities are indicated for state transition to occur over a 3-year period. The costs incurred during each year are determined according to the cash flow. The expected cost associated with choosing an option is calculated by summing over transition timesthe product of yearly costs and the probability of pavement remaining in the current state.

The cost coefficients for the budget constraint depend on both the year in which the decision is made and the year in which the cash flow occurs. The formula for calculating these costs is given as

$$c^t_{ijk} = c_{ijq} \times (1 + \text{inf})^t \times \left(1 - \sum_{r=0}^{q} p^r_{ijd}\right) \quad \text{for any future state } d \qquad (7)$$

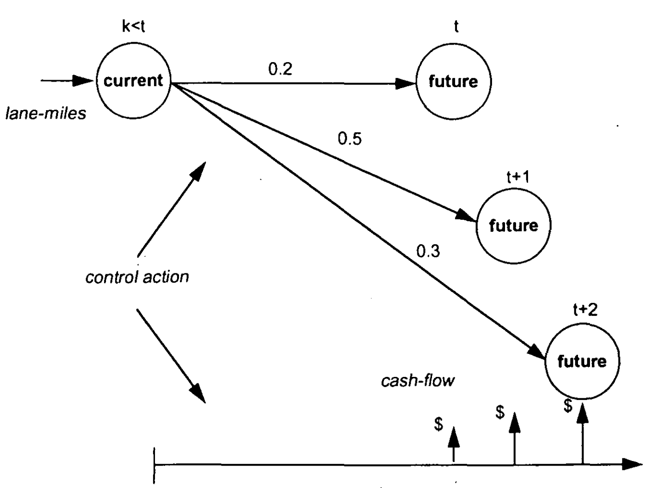

FIGURE 1 Illustration of state transition.

where $q = t - k$. The decision is being made in year k, and the cash flow is occurring in year t. The first factor is the cost of treatment option j applied after q years to pavement at state i. The second term is an inflation factor, and the third is the probability of pavement remaining at state i in year q. This probability is determined by summing the transition probabilities for previous years and subtracting that value from 1. Figure 1 is again helpful in understanding the derivation of these coefficients. For example, by time t there is only an 80 percent probability of pavement remaining in the current state because there is a 20 percent probability of transitioning to a future state in the preceding year. Only the probability of remaining in the current state is used in determining the costs associated with each year.

The coefficients for the condition constraint are calculated in a similar manner. The formula used is given as

$$a^q_{ijc} = \begin{cases} \left(1 - \sum_{r=0}^{q} p^r_{ijd}\right) & \text{for } i \in I_c \text{ and any } d \\ \left(\sum_{r=0}^{q} p^r_{ijd}\right) & \text{for } i \notin I_c \text{ and } d \in I_c \end{cases} \qquad (8)$$

Constraint 8 is concerned with the amount of pavement in a particular condition class at time t. The appropriate coefficient is calculated as one of two cases: the probability of pavement remaining in states belonging to the class, or the probability of pavement moving from a different class into states belonging to the class.

ILLUSTRATIVE EXAMPLE

Condition Data

The NYSTA network is divided into nominal sections of approximately 1 lane-mi based on 1991 survey data. Each section has at least the following properties: (a) pavement type, (b) AADT, (c) structural rating, (d) surface/slab rating, (e) crack/joint rating, and (f) length (lane miles). The problem size is reduced by analyzing only the overlaid pavement with less than 15,000 AADT per lane (this helps in presentation of results). Table 2 gives the classification of network inventory as distributed among various states. Column 1 has a five-letter code to denote each state with specific levels to the five state variables. Column 2 indicates the total lane miles of pavement classified into each state.

Treatment Options

Treatment options are identified for each pavement state. Each option is defined as a control action that consists of several specific treatment methods. According to the current practices of NYSTA pavement management, a set of eight treatment methods was identified:

- Do nothing (DN),
- Routine maintenance (RM),
- Preventive maintenance (PM),
- Heavy maintenance (HM),
- Resurfacing (RS),

TABLE 2 Assumed Distribution Among States

STATE	LANE-KM (in 1991)
11BFF	679.6
11BFG	356.8
11BGF	85.1
11BGG	37.0
11EEE	57.1
11FEE	0.0
11FFF	0.0
11FGF	0.0
11FGG	9.5
11GEE	14.5
11GGF	0.0
11GGG	0.0
11PEE	0.0
11PFF	1.6
11PFG	67.1
11PGG	15.3

1 km = 0.6 mi

- Rehabilitation of 4-in. overlay (RH1),
- Rehabilitation of 6-in. overlay (RH2), and
- Asphalt reconstruction (RC).

Preliminary cost estimates were obtained from construction and maintenance records of the most recent projects.

The notion of complete state transition control facilitates specification of treatment methods that vary (in type, frequency, and magnitude) from year to year. For example, routine maintenance that corresponds to fill-type patching may vary in terms of type (Grade 1 or 2 asphalt material), frequency (once or twice a year), and magnitude (square yards of patching). Although cost of applying maintenance (such as routine and preventive maintenance) can vary from state to state, such distinction is not made at this early stage.

Treatment options suitable for each state were assumed following simple guidelines developed by consulting NYSTA personnel. Sixteen states were developed, and each state was assigned at least two options. A sample of treatment methods and control action unit costs is given in Table 3. Columns 1, 2, and 3 indicate the current state, action number, and future state, respectively. Column 4 indicates treatment information for each control action. Specific treatment methods are planned for each year that the pavement remains in the current state.

Transition Probabilities

Transition probabilities associated with each state and control combination are based on condition data supplemented with engineering expertise accumulated over years of experience with NYSTA pavements. A computer program has been developed to display sequentially the information associated with each control action and enable the experts to predict the transition times. Each control action leads the pavement from one state to another over a random length of time. For each control action three transition times are specified:

- Lower limit (LL),
- Most common length of time (CL), and
- Upper limit (UL).

These transition times are used to construct a triangle of unit area representing the probability density function for the transition time. Table 4 presents a summary of the transition probabilities derived using the transition times predicted for some of the controls.

Variational Analysis

The linear programming problem is formulated and solved for a fixed annual budget, with and without condition constraints. First, the problem is solved considering only the budget constraints with an analysis period of 5 years. Second, additional constraints are introduced on the network condition in order to satisfy the long-term goals of management. Finally, the analysis period is extended to 10 years and only the budget constraints are considered. The problem may be extended to conduct a more rigorous analysis depending on the availability of data and the ability to define appropriate budget and condition constraints.

Results

Five-Year Analysis

Case 1: Budget Constraints Only An annual budget of $30 million is specified. The results from the linear program are given in Table 5. Major work including rehabilitation and

TABLE 3 Treatment Methods and Costs of Control Actions

Current State	Action Number	Future State	Treatment Information Code_Year Of Action	Unit cost ($1000/Lane-km)
OLBFF	1	OLEEE	RH1_1	39.77
OLBFF	2	OLEEE	RC_1	93.21
OLBFF	3	OLGEE	RH2_1	59.65
OLBFG	1	OLBFF	DN_0	0.00
OLBFG	2	OLBFF	PM_1, HM_2	14.29
OLBGF	1	OLBFF	DN_0	0.00
OLBGF	2	OLBFF	PM_1, HM_2	14.29
OLBGF	3	OLBFG	HM_1	9.32
OLBGG	1	OLBGF	DN_0	0.00
OLBGG	2	OLBFF	PM_1, PM_1	11.18
OLEEE	1	OLGGF	RM_2, RM_3	4.35
OLEEE	2	OLGGG	PM_2, PM_3	11.18
OLEEE	3	OLGEE	RM_1, RM_2, PM_2	9.32

TABLE 4 Example Transition Times and Probabilities for Control Actions

Current State	Action Number	Future State	Transition Times (Years) LL	CL	UL	Transition Probabilities Yr(1)	Yr(2)	Yr(3)	Yr(4)	Yr(5)
11BFF	1	11EEE	2.00	2.50	3.00	0.00	0.00	1.00	0.00	0.00
11BFF	2	11EEE	2.00	3.00	3.50	0.00	0.00	0.67	0.33	0.00
11BFF	3	11GEE	1.00	2.00	3.00	0.00	0.50	0.50	0.00	0.00
11BFG	1	11BFF	1.00	2.00	2.50	0.00	0.67	0.33	0.00	0.00
11BFG	2	11BFF	2.00	3.00	4.00	0.00	0.00	0.50	0.50	0.00
11BGF	1	11BFF	1.00	1.50	2.50	0.00	0.67	0.33	0.00	0.00
11BGF	2	11BFF	2.00	2.50	3.50	0.00	0.00	0.67	0.33	0.00
11BGF	3	11BFG	1.00	1.50	2.00	0.00	1.00	0.00	0.00	0.00
11BGG	1	11BGF	1.00	1.50	2.50	0.00	0.67	0.33	0.00	0.00
11BGG	2	11BFF	1.50	2.50	3.50	0.00	0.50	0.50	0.00	0.00
11EEE	1	11GGF	2.50	3.50	4.00	0.00	0.00	0.67	0.33	0.00
11EEE	2	11GGG	3.00	4.00	5.00	0.00	0.00	0.00	0.50	0.50
11EEE	3	11GEE	1.50	2.50	3.00	0.00	0.67	0.33	0.00	0.00

resurfacing is scheduled for 1045.9 lane-km (649.9 lane-mi) over a 5-year period. RH1 has been chosen as the optimal treatment for all lane miles of pavement in state OLBFF. DN_0 was chosen for 356.8 lane-km (221.7 lane-mi) of pavement in state OLBFG. Do nothing as opposed to preventive and heavy maintenance causes the pavement to deteriorate faster to state OLBFF. Consequently, rehabilitation is necessitated by 1993 and 1994. This indicates that maintenance is more expensive than do nothing because of the insignificant gain in pavement life with maintenance. Clearly, this result is a direct consequence of data used. But if funds were to be insufficient in 1993 and 1994, a maintenance action in 1991 would probably be chosen to defer the major work until funds are available. Such decisions are of great significance to pavement managers.

It is interesting that relatively good pavement (in states OLFGG, OLGEE, etc.) received the do nothing option whereas pavement in fair condition (in states OLPFF, OLPGG, etc.) received maintenance actions. This indicates that the appropriate time to conduct maintenance is when the deterioration (not just the rate of deterioration) is high enough that the maintenance effort will be cost-effective. In other words, maintenance at an early stage will decrease the deterioration rate but probably not improve the condition significantly. On the contrary, maintenance at a later stage can improve the condition as well as decrease the deterioration rate and consequently extend the life relatively more. Again, this result is a direct consequence of the data used.

Case 2: Budget and Condition Constraints In Case 2 additional constraints on future pavement condition are imposed. Four condition classes are established: safety with excellent and fair ratings, and ride quality with excellent and fair ratings. Threshold values are specified for the final year to target an increase in total lane miles distributed among each of the four classes.

The results of this case are presented in Table 6. Both RH1_1 and RH2_1 have been chosen as the optimal treatments for pavement in state OLBFF. In contrast with Case 1 (in which mostly do nothing was chosen), most of the pavement in other states received routine, preventive, or heavy maintenance. This is expected since the condition constraints achieve the targeted goals that correspond to increased lane miles among excellent and fair classes of safety and ride quality. The total expected present worth cost for the 5-year program increased from $46 million to $57 million.

It is necessary to note that the programmed lane miles correspond to a sequence of treatments (defined as part of the control). For example, in Table 6, 57.1 lane-km (35.5

TABLE 5 Results of 5-Year Analysis (Case 1)

Current State	Ln-km (in 1991)	Optimal Treatment	Programmed Lane-km 1991	1992	1993	1994	1995
OLBFF	679.6	RH1_1	679.6	-	238.0	118.8	-
OLBFG	356.8	DN_0	356.8	-	-	-	24.6
		PM_1, HM_2	-	-	85.1	-	-
OLBGF	85.1	DN_0	-	-	-	57.1	29.1
		HM_1	85.1	-	24.6	-	-
OLBGG	37.0	DN_0	37.0	-	10.1	6.1	0.5
OLEEE	57.1	RM_2, RM_3	57.1	-	-	679.6	-
OLFGF	0.0	RS_1	-	-	6.3	3.2	-
OLFGG	9.5	DN_0	9.5	-	-	-	-
OLGEE	14.5	DN_0	14.5	-	-	-	-
OLGGF	0.0	DN_0	-	-	-	38.1	19.0
OLGGG	0.0	DN_0	-	-	-	6.3	3.2
OLPEE	0.0	DN_0	-	-	-	6.3	3.2
OLPFF	1.6	HM_1	1.6	-	-	-	-
OLPFG	67.1	RM_1, RM_2, RM_3	67.1	-	-	-	-
OLPGG	15.3	RM_1, RM_2	15.3	1.6	-	-	-

TABLE 6 Results of 5-Year Analysis (Case 2)

Current State	Ln-km (in 1991)	Optimal Treatment	Programmed Lane-km				
			1991	1992	1993	1994	1995
OLBFF	679.6	RH1_1	428.4	-	46.0	166.9	143.9
		RH2_1	251.2	-	-	-	-
OLBFG	356.8	DN_0	69.0	-	-	-	-
		HM_2	287.7	-	85.1	-	-
OLBGF	85.1	DN_0	-	-	-	-	11.9
		PM_1HM_2	-	-	24.6	12.4	-
		HM_1	85.1	-	-	-	-
OLBGG	37.0	DN_0	37.0	-	17.9	36.9	14.0
OLEEE	57.1	RM_1RM_2PM_2	57.1	-	-	428.4	-
OLFGF	0.0	RM_1	-	-	-	6.3	-
		RS_1	-	-	-	-	3.2
OLFGG	9.5	PM_1PM_2	9.5	-	-	-	-
OLGEE	14.5	RM_1PM_2PM_3	-	-	163.7	-	-
		DN_0	14.5	-	-	144.7	-
OLGGG	0.0	DN_0	-	-	14.5	-	-
OLPFF	1.6	HM_1	1.6	-	-	-	-
OLPFG	67.1	RM_1HM_2RM_2PM_3	67.1	-	-	-	-
		RM_1RM_2RM_3	-	-	15.3	-	-
OLPGG	15.3	RM_1RM_2	-	1.6	-	-	-
		DN_0	15.3	-	-	-	-

lane-mi) in state OLEEE are programmed to receive routine maintenance for the first year and both routine and preventive maintenance during the second year. Although nothing appears in the column corresponding to 1992 (Table 6), treatments are still planned to be performed during that year.

Ten-Year Analysis

Ten-year analysis is a simple extension of Case 1 in 5-year analysis. This case presents the long-term effect on the overall costs. The results are given in Table 7. Increase in analysis period caused insignificant changes in pavement program. This indicates that the long-term economic trade-offs (for this simple case) are less significant than the economic trade-offs between pavement sections at different condition levels and their treatment options. Indeed, it highlights the need for conducting a network optimization.

DISCUSSION OF RESULTS

Pavement States Definition

The definition of pavement states on the basis of pavement distress condition will facilitate the project-level planning process. Important advantages of defining pavement states according to condition measures include the abilities to (a) provide meaningful information to assess the extent of pavement damage, and (b) correlate states with other engineering factors and establish broad classes to which various states may belong. The former is essential to define specific treatment options. Specificity in identified treatment options would alleviate the variability in cost estimation and prediction. States would also allow better estimates for extent of repair based on actual distress condition.

Grouping of states into classes considering safety, ride quality, and so on will enable management to define specific con-

TABLE 7 Results of 10-Year Analysis

Current State	Ln-km (in 1991)	Optimal Treatment	Programmed Lane-km									
			1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
OLBFF	679.6	RH1_1	679.6	-	238.0	118.8	-	16.4	12.1	4.0	-	-
OLBFG	356.8	DN_0	356.8	-	-	-	24.6	-	-	-	23.7	11.6
		PM_1, HM_2	-	-	85.1	-	-	-	-	29.8	-	-
OLBGF	85.1	DN_0	-	-	-	57.1	29.1	-	-	-	-	-
		HM_1	85.1	-	24.6	-	-	29.8	23.7	11.6	5.5	3.4
OLBGG	37.0	DN_0	37.0	-	10.1	6.1	0.5	8.5	3.9	3.2	4.2	2.1
OLEEE	57.1	RM_2, RM_3	57.1	-	-	679.6	-	294.8	147.1	16.4	16.4	12.1
OLFGF	0.0	RS_1	-	-	6.3	3.2	-	-	-	-	3.2	1.6
OLFGG	9.5	DN_0	9.5	-	-	-	-	-	4.8	-	-	-
		DN_0	-	-	-	-	-	-	-	9.7	-	-
OLGEE	14.5	DN_0	14.5	-	-	-	-	-	-	-	-	-
OLGGF	0.0	DN_0	-	-	-	38.1	19.0	-	453.7	251.7	221.9	202.8
OLGGG	0.0	DN_0	-	-	-	6.3	3.2	19.0	-	-	-	-
OLPEE	0.0	DN_0	-		-	6.3	3.2	-	-	-	-	3.2
OLPFF	1.6	HM_1	1.6	-	-	-	-	-	-	-	-	-
OLPFG	67.1	RM_1, HM_2, RM_2, PM_2	67.1	-	15.3	1.6	-	-	-	-	-	-
OLPGG	15.3	RM_1, RM_2	-	-	-	-	-	4.2	4.2	1.1	-	-
		DN_0	15.3	-	-	-	-	-	-	-	-	-

dition goals to be targeted. For example, pavement states may be classified as comfortable or uncomfortable by correlating a road user's perceptions of ride quality with pavement states. Then the management can impose a constraint to maintain less than a certain number of lane miles in uncomfortable states for each year in the planning horizon. A target level of network ride quality may also be specified for each year in the analysis period.

Some of the disadvantages in basing pavement states on condition measures include the relatively large number of states, the difficulty in defining state transitions, and the analytical complexity. Because of the large number of states (270), the overall size of the problem (considering several alternatives to each state and several years in the analysis period) is relatively large. Considerable effort may be required to implement the whole system.

Treatment Options Identification

Identifying suitable treatment options requires engineering expertise. It is essential to communicate clearly the pavement distress condition to the experts. As discussed, condition states are an effective way to assess the extent of pavement damage and assign several treatment options for each state. (Note that the treatment options are irrespective of individual sections classified into each state.) Each treatment option is planned using the state increment control approach as a short-term action.

The state increment control approach incorporates planning that relies not only on historical data but also on engineering expertise. Condition data alone are often insufficient to predict condition as a consequence of action. Alternatively, empirical knowledge applied using state increment control can facilitate both planning and prediction.

Nominal Sections Classification

Classifying nominal sections into pavement states can be improved on the basis of the specific characteristics of each nominal section. For example, consider an unrated (existing or new) 20-lane-mi section that is undergoing repair or construction (which may take 2 years). Such a section can be specified to reach the state of OLEEE in the third year.

In general, not all network inventory needs to be classified into states for the first year in the analysis (as done in the illustrative example; see Table 2). A more appropriate classification of the inventory can be achieved by analyzing the condition data and the maintenance status of each nominal section.

Linear Program Formulation

The optimal decisions at various levels of pavement management are mostly dictated by the trade-offs between economic and engineering factors. These factors exhibit subtle interactions since the decisions are bound by constraints. Hence, simultaneous determination of the treatment, timing, and magnitude of work is an important part of the decision process. The presented decision-making methodology achieves this task using a linear programming formulation.

The emphasis in the presented methodology has been on aggregating pavement sections of similar condition characteristics. Aggregation into states according to lane miles significantly reduces the complexity of the problem. It enables the application of a relatively simple linear programming technique for constrained optimization. In contrast, dynamic and integer programming have several limitations. In specific, it is very difficult to deal with multiple constraints in a dynamic programming application. It limits the control over decisions as needed for network optimization. Often large-scale integer programming problems are very difficult to formulate as they require alternatives to individual projects over multiple years (*9*). In practice it may be difficult to develop methodologies for treatment identification and cost estimation that are needed to support such formulations. Lack of explicit condition measures and a prediction model makes the decision process subjective. To minimize the subjectivity, it may be necessary to identify a few well-understood characteristics and achieve consistency in evaluation (in which case aggregation may be a better approach). The size of an integer programming problem could be significant for large networks, adding computational complexity. The number and nature of constraints on the problem (depending on the number of pavement characteristics and project interactions considered in the decision process) can add to the complexity.

Analytical Aspects Evaluation

The presented optimization methodology recognizes the role of project-level analysis as planning treatment options to each state while applying engineering expertise in prediction process. The management specifies the threshold condition levels and available budget for the entire network. A linear programming formulation is used to best allocate the budget and determine a feasible multiyear pavement program.

The objective function provides an economic comparison of the alternatives in terms of expected long-term costs. Minimizing costs rather than maximizing benefits alleviates the problem of finding a "correct" benefit value function. Specifying appropriate condition goals and constraints is essential to obtaining effective results in either case.

Life-cycle cost analysis is traditionally conducted on an individual project basis without explicit consideration of pavement condition. Consequently, costs and savings incurred between two different projects cannot be directly compared (a value judgment of other factors is necessitated). In contrast, state increment control approach explicitly considers the pavement condition at each stage. The differences in the costs as well as other aspects related to states (such as condition, safety, and ride quality) may be integrated into network-level analysis for constrained optimization. Once the management goals and constraints are explicitly stated, trade-offs between projects are essentially captured without the need for value engineering judgments.

The current formulation can provide answers to many "what if" scenarios. For example, the implications of maintaining less than 10 percent of the network in "unsafe" states compared with that of 20 percent that might boost the rehabilitation program could be considered in terms of increased costs (or increase in savings if user costs as a function of safety are included in the objective function). And variations in the pavement program as the budget limitations vary may be observed through sensitivity analysis.

SUMMARY AND CONCLUSIONS

The network optimization methodology presented in this study is an essential requirement for program planning and budget allocation involved in NYSTA's PMS. Preliminary investigation was conducted to identify the most important parameters that quantify distress and no-distress factors affecting the decision process. Specific values taken by these parameters describe the pavement condition state. The network condition was represented as lane miles distributed into various states. This was achieved by dividing the network into nominal sections of varying lengths and classifying each section into one of the states according to condition characteristics. Each state was assigned several treatment options that transform the pavement condition over time. The transitions between states are defined to model the consequences of each treatment option. A linear programming application was used for constrained optimization and determination of an optimal pavement program. An illustrative example was presented with details on data collection, network constraint formulation, and variational analysis for obtaining desirable results. The results demonstrate the ability of the mathematical model to provide answers to network-level pavement management questions.

From the research and developed methodology, the following conclusions are drawn:

- Representing network condition as lane miles of pavement distributed among various states reduces the problem complexity, thus enabling application of linear programming optimization techniques that are simple.
- Specifying treatments as controls that achieve a complete state transition enables the use of historical data and engineering expertise in the prediction process.
- Modeling interactions between economic and engineering factors (e.g., cost, budget, condition, timing of MR&R) is essential to evaluate the consequences of decisions in a effective manner.
- Using the illustrated optimization methodology may help to develop an effective multiyear pavement program and ensure optimal budget allocation for the entire network.

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Ranking Versus Simple Optimization in Setting Pavement Maintenance Priorities: A Case Study from Egypt

ESSAM A. SHARAF

The success of a maintenance management system depends largely on the efficiency of the maintenance program that is produced. An efficient maintenance program is the program that identifies what maintenance action to be taken and where and when to apply it so that the most cost-effective results are obtained. The process used to answer these questions is called the maintenance priority setting. Three priority-setting techniques are presented along with the results of their applications on the data collected from the Egyptian road network. The first technique is a simple ranking based on current year condition data. The second is a modified ranking technique that considers the future condition of pavement sections, and the third is a near optimization, one that considers both time (current and future) and space (entire network). A comparison of the techniques in terms of network condition over time and budget deficit is presented. The results indicate a considerable difference in future network performance under the three techniques, with the optimization technique producing the best results.

A typical pavement maintenance management system (PMMS) would consist of several components, including

- Network identification and coding,
- Inventory of network physical features,
- Network condition assessment,
- Maintenance needs assessment,
- Comparison between needs and available resources to establish priorities,
- Production of the maintenance program, and
- Monitoring the execution of the program.

In fact, these components should cover three basic responsibilities of a decision maker: the abilities to

1. Describe the current condition of the network;
2. Select the best maintenance program (i.e., which maintenance action to do and where and when to do it, so that a maximum utilization of the available resources is achieved); and
3. Monitor the execution of the maintenance program.

With that in mind, the process of setting the maintenance priorities is of utmost importance to the entire PMMS process. This may be because the priority setting is the step after which a final decision is to be made on the maintenance program to be executed. In addition, and even more important, the quality of the priority setting directly influences the effectiveness of available resources, which, in most cases, is a prime goal of a decision maker. The massive efforts typically allocated to the phases of data collection and needs assessment may very easily be wasted if the appropriate priority schemes are not applied.

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Priority-setting techniques as used in the PMMS cover a wide spectrum of methods and approaches ranging from simple priority lists based on engineering judgment to complex network optimization mathematical models. In most cases, the effectiveness of the techniques, particularly in the long range, is directly proportional to the complexity of the scheme.

The degree of complexity, or comprehensiveness, of a priority-setting scheme is generally a function of the time and space dimensions when dealing with the network condition as shown in the following (*1–5*):

Time Dimension	*Space Dimension*
Current year	Section by section
Future years	Simultaneous consideration of all sections
– End of analysis period	
– Each year in analysis period	

For instance, the simplest form of priority-setting schemes as used in PMMS would be the one that considers only the current year condition on a section-by-section basis. On the other hand, a complex scheme would be the one that considers the yearly condition of the sections comprising the network in a simultaneous manner so that the effect of changing the condition of a section on the rest on the network could be assessed. Several levels exist in between these extremes.

A key issue in this concern is the choice of the appropriate priority-setting scheme. It is not necessarily that the most comprehensive one will be the best to use, at least from the point of view of the decision makers. Data availability plays an extremely important role in such selection, particularly in developing countries. In fact, the titles of some such techniques may discourage the decisions makers from proceeding. Therefore, it is of paramount importance to take a special care when introducing such techniques and concepts to decision makers.

The purpose of this paper is to demonstrate the use and the expected benefits of three priority-setting techniques: (a) a simple ranking based on a pavement's current year condition, (b) a ranking based on a pavement's current and future conditions, and (c) a near optimization technique. Although the three techniques could be classified as simple ones, it is believed that they are appropriate for use in developing countries, particularly at the early stages of applying PMMS.

BACKGROUND

In 1986–1988 a network study was conducted in Egypt with the aid of a team of international consultants through a project funded by the World Bank (*6*). The intent of the study was to provide the basis of a framework for decision making in road maintenance, one based on a ranking methodology that relies on an objective approach with regard to need and standard rather than custom and practice.

The study undertook the normal steps of a PMMS as described earlier. Network identification and coding in terms of links and sections was completed and followed by a comprehensive inventory of the physical features of the network and traffic volumes. In addition, a pavement evaluation in terms of a visual inspection was performed. Finally, an intervention logic was developed to assess maintenance needs on the basis of the collected pavement condition data (Table 1).

After these basic steps, a ranking model using the current pavement condition was developed. This model was used to identify sections to be included in the current year's maintenance program. This model proved to be insufficient in several ways, as will be discussed later. A special research study was initiated at Cairo University, Egypt, to improve the model (*7,8*). This research resulted in two other models. The ranking model and the other two models will be discussed in the following sections. The three models will be referred to hereafter as

- Model 1: ranking model developed by the 1986–1988 Egypt road network study.
- Model 2: modified ranking model developed by a special research study, Cairo University.
- Model 3: a near optimization model developed by a special research study, Cairo University.

The three models are based on the condition data and the intervention logic developed by the 1986–1988 study. This, in fact, was necessary to demonstrate that the differences between the models could be attributed to the priority techniques rather than to the condition data or the intervention logic.

MODEL DESCRIPTION

Model 1

The intervention logic presented in Table 1 was used to determine the appropriate maintenance treatment for each section. To establish a priority measure, a treatment index associated with each section was calculated as follows:

$$\text{priority index} = \frac{\text{defect length}}{\text{traffic factor} * \text{defect factor}}$$

The traffic factor took on values of 0.1, 0.5, and 1.0 for average daily traffic levels of less than 2,500 vehicles per day (vpd), between 2,500 and 10,000 vpd, and more than 10,000 vpd, respectively. The defect factor, on the other hand, was assigned to each section on the basis of the defect type and the required treatment, as presented in Table 2.

The priority index for the entire section was then calculated as the sum of the priority indexes for both the highway and the shoulder. This way, the higher the traffic level is and the more severe the defect, the higher the section index.

After the calculation of section indexes, sections were ranked in descending order according to the index values. The resulting list was considered to be the priority list and was con-

TABLE 1 Maintenance Treatment Intervention Levels

MAINTENANCE TREATMENT	INTERVENTION LEVELS	DISTRESS SEVERITY
Rehabilitation	- alligator cracking > 0.5 km - alligator cracking > 3500 sq.m. - open potholes > 0.5 km - reflection cracking < 10 m.	- High Severity - High Severity - Medium & High - High Severity
Reshape and Overlay	- rutting > 0.5 km. - old patches > 50% and bad ride quality	- Rut Depth > 0.2 cm - High Severity
Overlay	- old patches > 50%	- High Severity
Surface Dressing	- lean texture (bleeding)	- High Severity
Edge Patching	- edge fretting > 1.0 km.	- High Severity
Shoulder treatment	- low shoulder > 2.0 km.	- Shoulder Drop > 5 cm
Recurrent Maintenance	- for pavement distresses with severity levels or quantities less than those shown above	

TABLE 2 Assignment of Defect Factor

Defect	Treatment	Defect Factor
Open potholes	Rehabilitation	0.10
Alligator cracking	Rehabilitation	0.15
Reflection cracking	Rehabilitation	0.20
Rutting	Reshape and overlay	0.30
Old patching	Overlay	0.50
Lean surface texture	Surface dressing	0.70
Edge fretting	Edge patching	1.00
Low shoulder	Shoulder works	1.00

verted to a costed list by using the appropriate maintenance treatment unit costs. The top sections with total cumulative costs equal to the allowable budget were selected for the current year's maintenance program.

Model 2

Both the condition survey data and the rating algorithm used in Model 1 were used in this model to determine the required maintenance treatment and the calculation of the priority indexes. Model 2, however, differs from Model 1 in two ways. First, pavement sections were arranged according to road type and traffic level into four classes. This classification was then used to determine the average treatment ages. An average age of a treatment is the number of years after which a new (or renewed) section would require that treatment. For example, 15 years was found to be the average age for rehabilitation for pavement sections in a desert environment subjected to heavy traffic. The corresponding average age for sections in an agricultural environment (roads surrounded by canals, drains, or agricultural fields) is about 5 years. The second difference is that predetermined budget shares were reserved for different maintenance treatments (rehabilitation, overlay, and surface dressing).

The mechanism of this ranking model could be summarized as follows:

- Surveyed sections were grouped according to the four classes mentioned.
- The average ages of treatments were used to identify sections requiring different treatments.
- According to the identified treatments, section priority indexes were calculated and sections were ranked in a descending order of importance according to the calculated priority indexes.
- A costed list was produced for each treatment using the appropriate unit costs.
- The reserved budget share for a specific treatment was distributed on sections with highest-priority indexes that require that treatment. This was repeated for other treatments.
- The process was repeated for each year in the analysis period. At the beginning of each year, section ages were updated as follows: (a) sections selected for last-year programs were assigned ages equal to zero, and (b) ages of other sections were increased by 1 year.

The main output of this model was in the form of a yearly maintenance program including the locations (sections) and the suggested maintenance treatment.

Model 3

Model 3, the annual optimization model, was considered to be a direct extension to the ranking Model 2. The mechanism of this model is identical to that of Model 2. The only difference between the two models is in the way that the budget share of a specific treatment is distributed among the candidate sections. In Model 2, only the top sections in a treatment priority list are selected, but in Model 3, a simple optimization problem is solved to select a set of sections (projects) such that maximum section priorities are achieved.

For a specific year and specific treatment with budget share identified the following formulation was used to select the optimal set of sections:

Maximize

$$\sum_{i=1}^{n} a_i x_i$$

subject to

$$\sum_{i=1}^{n} c_i x_i \leq B \qquad x_i = 0 \text{ or } 1$$

where

n = number of projects in need of specified treatment,
a_i = priority index of ith section,
c_i = treatment cost of ith section, and
B = budget share of specified treatment.

This way, three optimization problems were run, one for each treatment for each year in the analysis period. The set of candidate sections for different treatments and the priority indexes used in this model were provided by running Model 2, and the resulting list of sections and their indexes were then used as input to this model.

COMPARISON OF MODELS

The three models were compared to evaluate their efficiency. First the results of applying the three models on Egypt's road network will be presented, then their relative efficiency and possible reasons behind their differences will be assessed.

An analysis period of 5 years was assumed. It was thought that longer periods would lack accuracy in prediction. Besides this, 5 years is a typical planning period in Egypt. The analysis period therefore was considered from 1987 to 1991. The results of the 1987 Egypt network condition survey were used in all models as the basis for identifying the 1987 maintenance needs (*6*). To evaluate the efficiency of the models, two main indicators were considered:

1. The yearly budget deficit, which indicated the difference between the cost of maintenance actions required to fully upgrade the network to a perfect condition and the available budget.
2. The yearly deficient portion of the network, which represented the general condition of the network in terms of the

percentage of the total network that was in need of major maintenance (rehabilitation or overlay).

Generally, the higher that either of the two indicators is, the less efficient the model.

Figure 1 shows the resulting deficit values under each of the three models, and Figure 2 shows the deficient portions of the network. It is obvious that Model 3 produced the best results, followed by Models 2 and then 1. The deficit and deficient portions increased over time under Model 1, whereas they decreased under the other two models. The results of Model 1 indicated an increasing gap between the desired and actual conditions, a situation in which actual would never catch desired. This is, however, a typical result of using such year-by-year and section-by-section simple ranking methods, as will be discussed. On the other hand, the other two models showed a situation in which a continuous improvement in the outputs can be achieved. For instance, under Model 2, a first-year deficit of about £E243 million (£E5.36 = $1.00 U.S., 1993) has improved over the 5 years to a value of about £E132 million. The corresponding values under Model 3 are £E243 million and £E101 million, respectively. The same trend can be observed for the deficient portions, where under Model 2, an initial value of about 35 percent deficiency has improved to about 17 percent at the end of the analysis period. The corresponding values under Model 3 are 35 and 14 percent, respectively. In the following paragraphs, an interpretation of these results will be presented.

The poor performance of Model 1 may be because

- The model is good for 1 year only (current year) since it did not consider future condition of the pavement sections. This has created a situation in which it was very difficult to introduce project timing in the process.
- The model is based on a section-by-section approach that ignored the relative effect of selecting a section for maintenance, on the network condition.
- The model produced one priority list with only the top sections are eligible for selection, which has led to a situation in which the available budget was consumed by sections in need of heavy maintenance (rehabilitation), leaving other sections with moderate need (overlay or surface dressing) for further deterioration. This produced the endless cycle mentioned earlier.

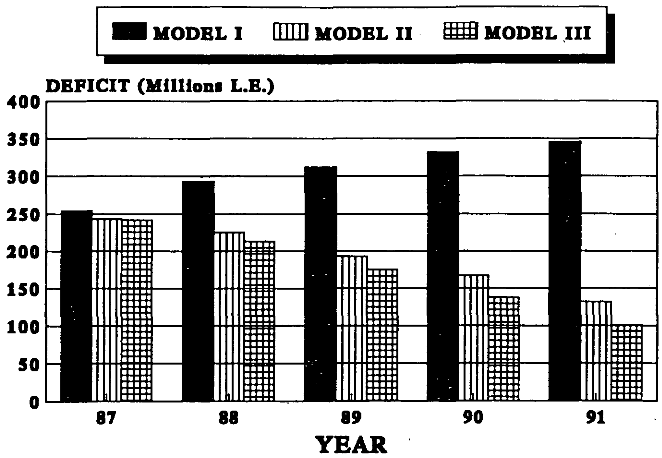

FIGURE 1 Budget deficit values.

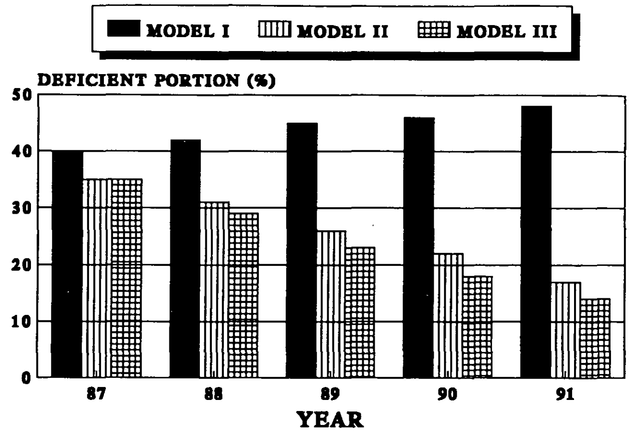

FIGURE 2 Network deficient portions.

The relative improvements in the outputs of Models 2 and 3 were basically due to the avoidance of some of the shortcomings of Model 1. For instance, in Model 2, the average age intervention logic allowed the consideration of project timing. And reserving predetermined budget shares for different treatments allowed the possibility of selecting sections in moderate need of maintenance instead of leaving them for further deterioration. One basic disadvantage of Model 2, however, was the section-by-section approach in which only the top sections on the list of each treatment consumed the reserved budget share. This disadvantage was, however, avoided by using the simple optimization solution to select the best set of sections within each treatment list so that maximum sum of priorities was achieved. This way, section selection was not constrained to the top.

SUMMARY AND CONCLUSIONS

Three priority-setting techniques were presented in this paper: a simple ranking technique based on first-year condition, a modified ranking technique based on first- and future-year conditions, and a simple annual near optimization technique. The data used in the comparison of the three techniques were obtained from a comprehensive survey of the Egyptian road network. The results indicated the superiority of the optimization model in terms of improved budget deficit and network condition over the analysis period.

Although the techniques can be classified as simple ones, the optimization model with its relative complexity is a suitable means for roadway agencies in developing countries—or for agencies in developed countries in their early stages of applying a PMMS—by which to allocate available funds. The data requirement for such techniques is minimal and can be easily collected and monitored.

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Objective-Oriented Approach to Fulfilling the Need for PMS Pavement Structure Survey Data in Japan

M. SEKIGUCHI, E. FUJITA, M. INAGAKI, AND T. MCGREGOR

The pavement management system (PMS) is considered to be of significant value in the effective management of road systems. Large amounts of data related to road surface condition, pavement structure, mechanical properties, and such must be stored in the PMS to establish and render the system useful. The common methods used to determine pavement structure—trial digging and large-diameter boring—are destructive methods that supply only local information about the pavement structure. A rapid and wide-ranging survey system is required in order to add functional value to the PMS. An integrated system using a combination of ground penetrating radar (GPR) and the borehole camera (BHC) has been developed in Japan by focusing sharp attention on how such a system must perform. GPR supplies overall pavement structure information, and the BHC provides an accurate image of layer thicknesses and pavement material composition at a broader range of points. By combining the GPR overall data and the BHC specific location data into a single vehicle-mounted survey system, an accurate profile of the pavement structure can be successfully created. The system proved useful after evaluation of the trial period in terms of survey functionality and practical accuracy. Introduction of the system is considered to increase the accuracy of the PMS and thus of the formation of pavement management decisions.

It is a fact of life that roads, after construction, must be periodically repaired to maintain a suitable traffic flow condition. Because maintenance records are often inaccurate or hard to find, the maintenance history is likely to be an ineffective repair strategy development tool. Knowledge of a pavement's structure is necessary for designing a repair strategy. Accurate structure information is also needed to evaluate the increasingly popular falling weight deflectometer (FWD) data, to calculate future failure curves, and to add accuracy to a pavement management system (PMS) (*1*).

Recently, the importance of a PMS, which provides pavement surface and structure information for intelligent pavement decisions, has been growing worldwide (*2*). Current methods used to determine pavement structure are trial digging and core boring, but these methods give only limited information. To ensure the integrity of the PMS, a large amount of data taken over the road network must be acquired in a short period. For this purpose, the development of a more effective and convenient method has been anticipated in the industry. The Japanese government confronted the problem by designing a system that met certain predetermined objectives.

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OBJECTIVES IN SYSTEM DEVELOPMENT

During the development of the new system, the following priorities and objectives were established: the system must

- Be minimally destructive,
- Cause minimal interference with traffic flows,
- Provide a high degree of accuracy in the determination of layer thickness and material identification, and
- Be able to collect a large amount of data in a short period.

DESCRIPTION OF SURVEY SYSTEM

The survey system was created by combining ground penetrating radar (GPR) and a borehole camera (BHC) in a vehicle. The vehicle (Figure 1) was designed to use these two technologies conveniently as one operation (*3*). GPR meets the objectives of being a nondestructive tool for surveying a large area as quickly as possible, and the BHC was chosen to provide pinpoint accuracy in determining layer thickness and composition. Mounting the survey equipment on a vehicle was necessary in order to avoid obstructing traffic flows and increase efficiency.

GPR

In a GPR system, an electromagnetic pulse is radiated through an antenna to the pavement surface, where it continues to travel underground (*4*). It is partially reflected at the boundary between two layers, whose dielectrical properties are a little different from each other. This boundary is called an interface. The remaining radar energy propagates through successive layers, showing signals of interface as it strikes each boundary.

From this process the antenna receives a series of reflected pulses that represent each boundary. The pulses are repeatedly transmitted through the pavement. While the survey vehicle travels over the pavement, these pulses, which reflect interfaces, create a constant stream of radar reflection profiles. The vehicle drives at 30 to 40 km/hr. At this speed, the

FIGURE 1 Survey vehicle.

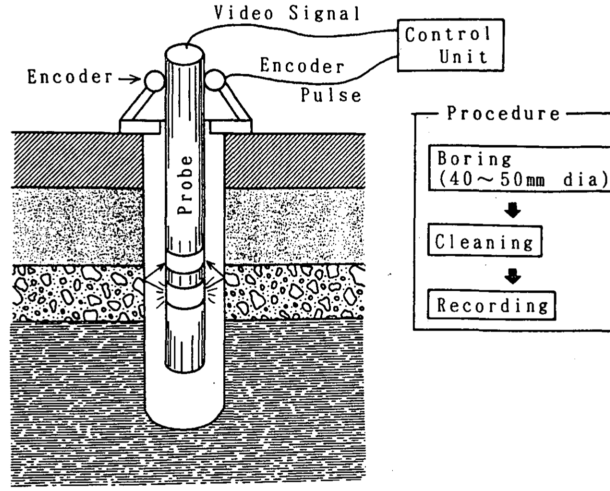

FIGURE 2 BHC acquisition drawing.

operators can cover a substantial length with minimal traffic obstruction.

The pulse length has been adopted as 1 nsec to obtain both the best available resolution and sufficient penetration. GPR, however, cannot measure the absolute layer thickness and it cannot determine material composition, because the velocity of an electromagnetic wave in a medium depends on the dielectric constant of the material. GPR cannot determine its own dielectric constant, so that constant must be determined by another method.

Borehole Camera

To supplement the limitations of GPR, the BHC has been introduced (*5*). The data acquisition process requires a borehole, which is destructive, but because of the small diameter of the probe, the borehole is limited to no more than 4 cm in diameter. Such small-diameter borings are considered minimally destructive to the overall pavement structure.

The hole reaches into the pavement subgrade, which in most cases in Japan is approximately 1 m from the surface. Before inserting the probe, the wall of the hole must be cleaned by a heavy-duty vacuum cleaner wand insertion using a water flush to remove any mud generated by the boring process; this provides a clean target for the image acquisition process. The diameter of the probe is 2.5 cm.

A full-angle view (360 degrees) of the wall is taken by the video probe, and this NTSC (National Television Standards Committee) image is digitized and recorded on magnetic tape. Figure 2 shows a sketch of the BHC data acquisition portion of the survey. BHC survey allows the thickness and the composition of the layers to be determined visually. After taking the BHC record, the hole is repaired with a fast-cure concrete and epoxy-mixed asphalt.

EVALUATION OF FIELD TRIALS

The suitability of this system needed to be proved in several actual pavement structure survey trials. In the evaluation period, three items were carefully evaluated.

Speed of Survey Vehicle

The survey vehicle travels at 30 to 40 km/hr. Profile information, which is derived from the displayed chart paper data, is made with every 10-m plot. However, plotting accuracy may decrease at higher speeds. To determine the effect of speed on the plotting accuracy, a test was carried out on Road 8 in Tokyo.

Six cases were investigated, at speeds of 0, 5, 10, 20, 30, and 40 km/hr; these speeds were considered in the comprehensive trial. Layer thickness observed under each survey speed has been compared with the stationary vehicle mode. The results are shown in Figure 3, which indicates the correlation of layer thickness between the moving vehicle versus zero speed. The allowable range of thickness deviation is practically established at ± 2.5 cm for asphalt and at ± 5.0 cm for subbase and others. The result shows that 100 percent of asphalt and 95 percent of crushed stone subbase are within allowable limits when surveyed at speeds under 5 km/hr, as are 92 percent of asphalt and 95 percent of crushed stone subbase under 40 km/hr. A slightly greater accuracy was obtained at slower speeds. It is believed that positioning accuracy is mainly related to this phenomenon. The results also showed better accuracy on shallower layers and an increase in thickness error in direct proportion to the depth of the medium. It is thought that this occurs because the subbase is not as smooth as the asphalt layers.

Material Properties

Layer thickness is calculated by a calibration method using a selected dielectric constant. The dielectric constant is 1.0 for air, and average values appear to be approximately 5.0 for asphalt and 8.0 for subbase in actual experience. These values usually experience a range of deviation to some extent. If deviation is quite large, GPR would not be used effectively for this purpose.

The relationship between the GPR data and the BHC data was investigated. It should be noted that zero-speed GPR data was used to eliminate the speed effect, as indicated in

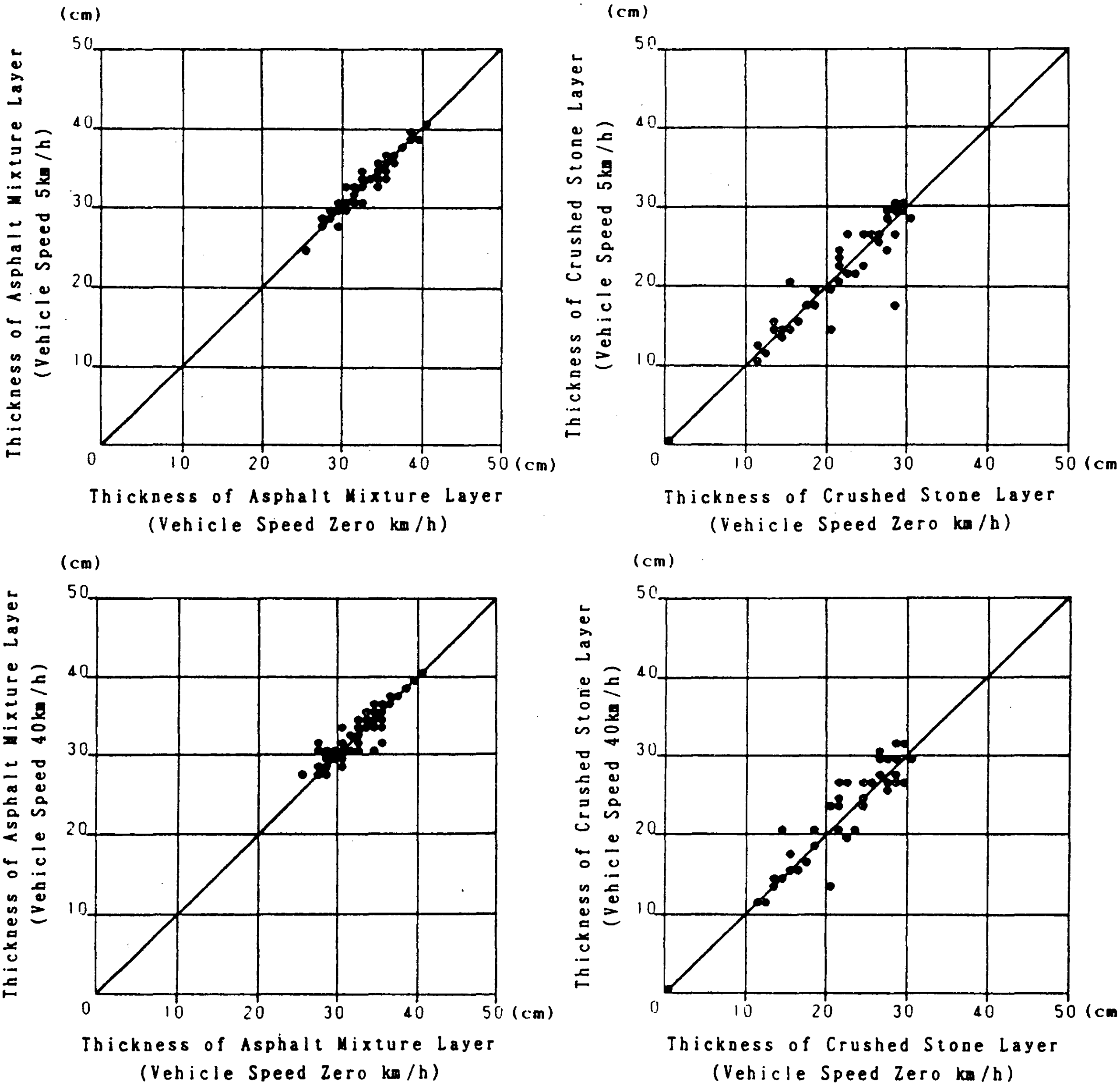

FIGURE 3 Correlation between vehicle speed and system accuracy.

the preceding section. The results are shown in Figure 4, which indicates that 91 percent of asphalt and 82 percent of subbase are within allowable limits. Deviation of material properties is not considered to have a practical effect on the result.

Combined Accuracy

In an actual investigation, layer thickness will be observed as a result of the combined condition of speed and material property effects. Though these effects occur at the same time, a simply combined value is considered to determine the minimum value. The result is that 84 percent of the asphalt and 78 percent of the subbase were within allowable limits. It shows that the system is practically suitable for application to the pavement structure survey.

Dielectric constant determination sometimes turns out to be higher or lower than expected, probably because of material mixture at a boundary or local changes of thickness. The calibration constant should be determined carefully by averaging or by adopting regional values to cancel as much error as possible.

APPLICATION

The pavement structure survey was carried out in the Tokyo metropolitan area. The survey distance was approximately 60 km.

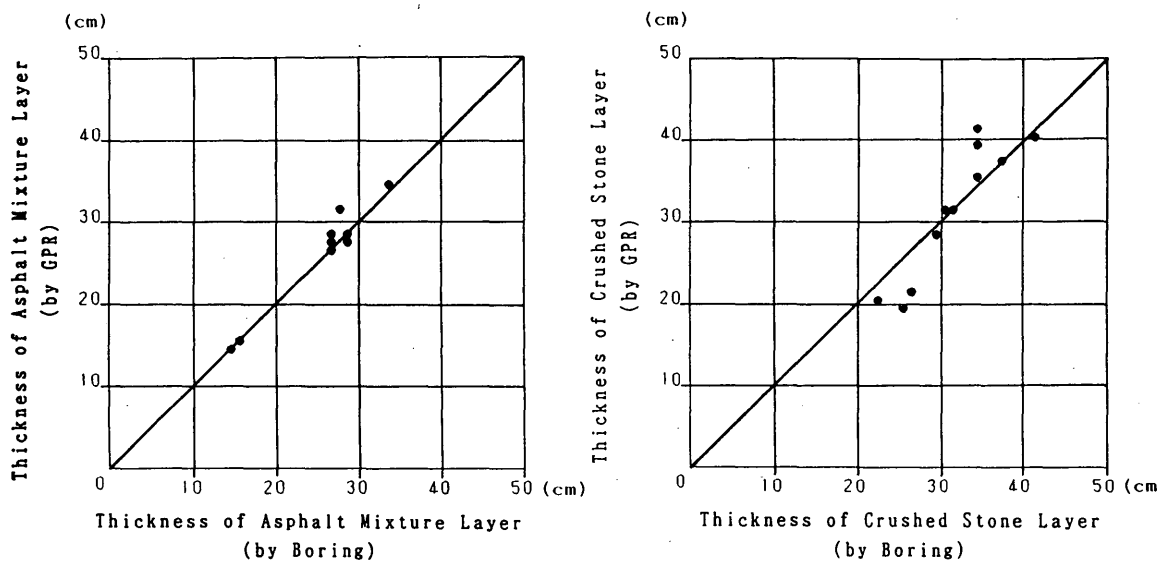

FIGURE 4 Relationship between BHC and GPR data.

Preparation

Within the target area, the locations of buried facilities such as water, gas, electricity, sewer, and communication lines must be identified so that they will not be damaged during the BHC portion of the survey.

GPR Survey

The GPR survey was carried out first. The vehicle speed was about 40 km/hr, because that speed proved sufficient to avoid traffic disruption. A sample of acquired GPR data is shown in Figure 5. After the site survey, all the data were divided into segments. Segmentation depends on the view of experts who distinguish some kind of critical pavement differences, such as the number of layers, relative layer thickness, and signal intensity. Distribution of segment lengths is illustrated in Figure 6. The average segment length was 440 m. It is preferable to bore one hole per segment, except for considerably similar segments. In this case, based on the segmentation, 50 BHC survey points were selected. To evaluate the result, 70 large-diameter boring survey points also were selected, some of which were at the same point as the BHC points.

BHC Survey

Before the BHC survey, all the selected investigation points were marked with spray paint. It is considered good practice for the surveyor to ask the owners of the buried facilities whether it is safe to dig around the points. If they recognize

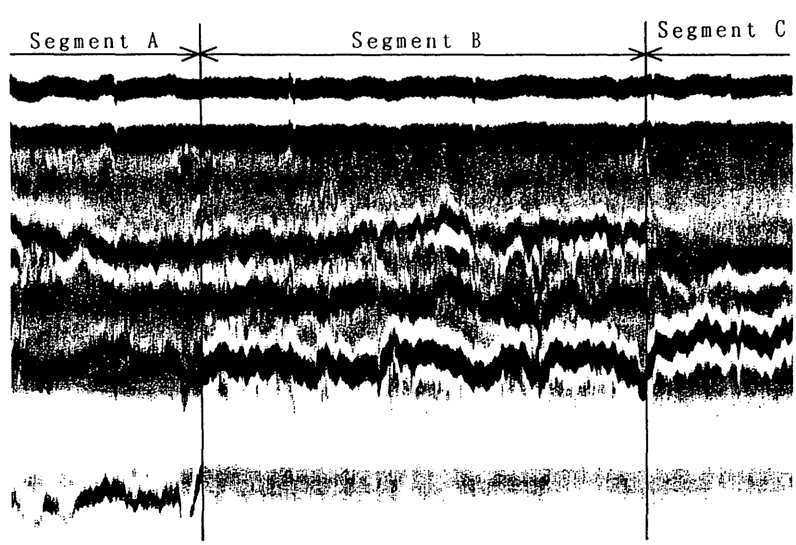

FIGURE 5 Sample of GPR data.

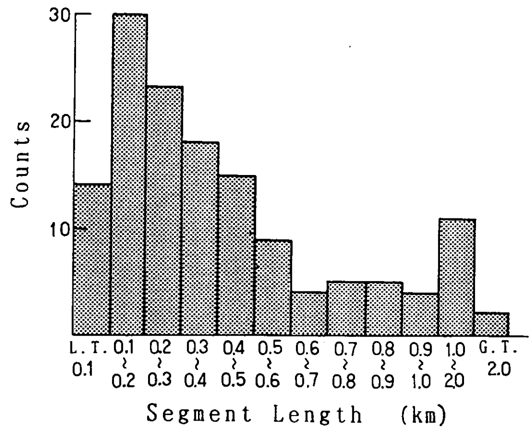

FIGURE 6 Distribution of segment lengths.

any problem, the boring point might be shifted a couple of meters.

When a BHC survey starts, traffic control must already be in place. After traffic control was in place in this survey, it took 30 min to bore the hole to a sufficient depth and 30 min to record the video data. Total work time per BHC record in the field was approximately 90 min, including the time it took to move to the next location. A record acquired by BHC survey is shown in Figure 7. Pavement structure is clearly observed in the picture. Figure 8 compares BHC survey results with large-diameter boring results. The thickness observed by both methods was very consistent. After the BHC record was taken, the hole was repaired by fast-cure concrete and epoxy-mixed asphalt.

Data Analysis

The GPR record was analyzed to make a profile in reference to the BHC record. A sample plot is illustrated in Figure 9. The upper part is the profile, and the lower part is the av-

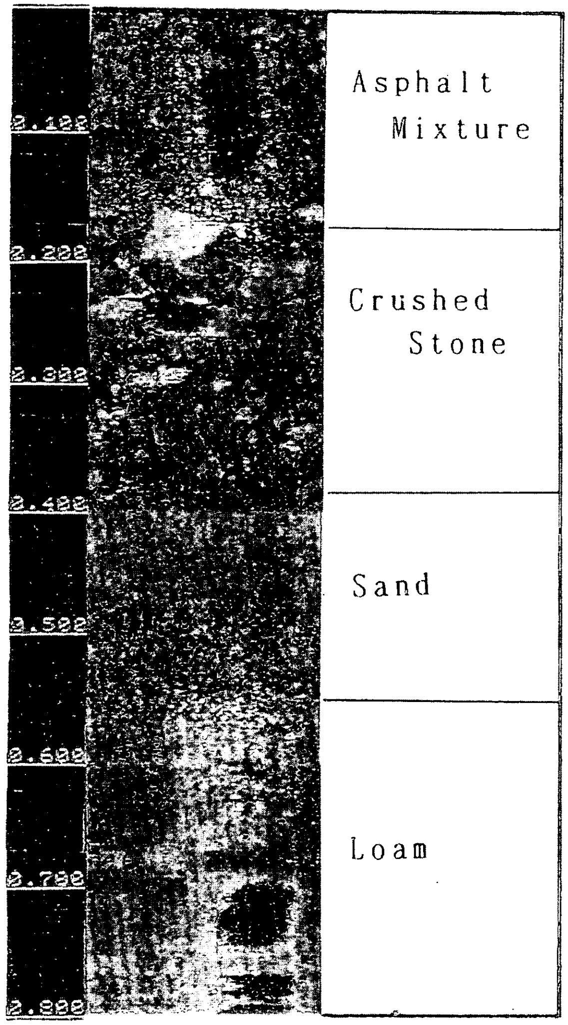

FIGURE 7 Sample BHC survey record.

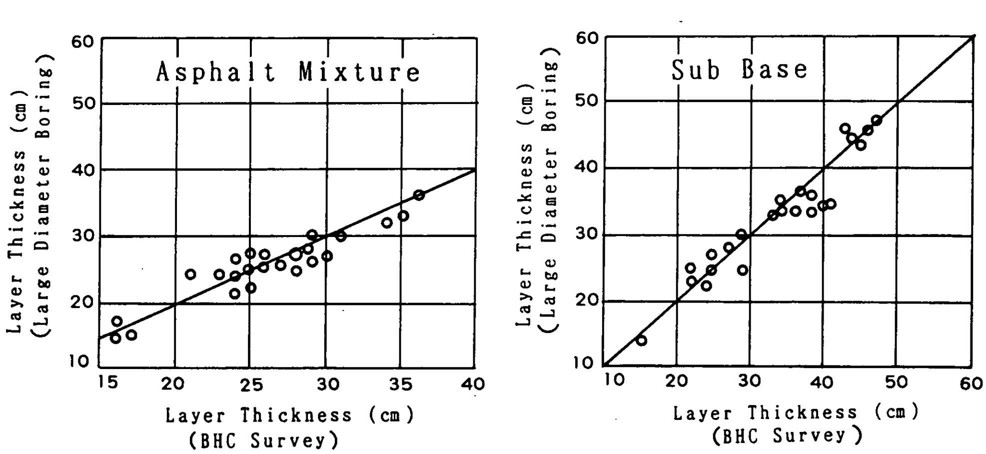

FIGURE 8 Comparison of BHC and core data.

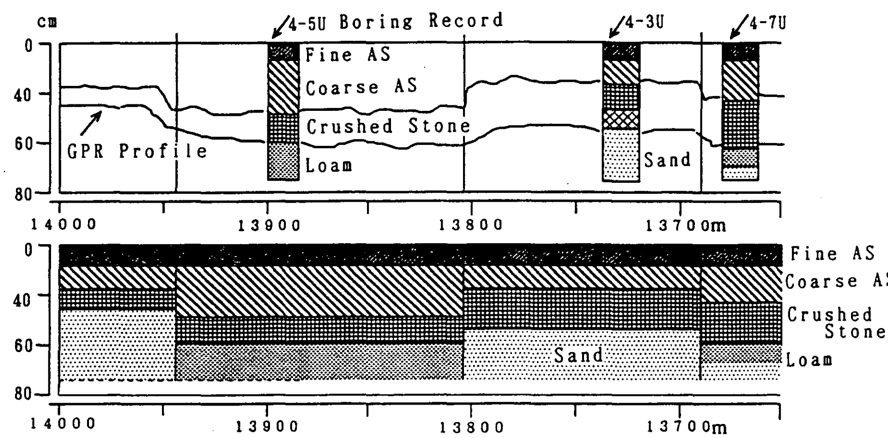

FIGURE 9 Software-generated profile of GPR and BHC data: *top*, profile of pavement structure; *bottom*, averaged profile.

eraged profile. Ultimately, the averaged profile is stored in the data base of the PMS.

CONCLUSION

The Tokyo metropolitan government is now establishing an extensive PMS. Various kinds of data are being stored: road surface condition survey data taken by the road surface observation vehicle, mechanical properties taken by FWD, and so on. A system was envisioned that would provide pavement structure data within certain operational parameters. The survey system of combined GPR and BHC mounted in a designed vehicle proved useful in determining the pavement structure. The advantage of this system is to add functionality that enables a wide-ranging survey to be performed in less time than the more common methods.

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There Are Two Categories of PMS Analysis Methods: Reactive and Generative

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

Two categories of pavement management system (PMS) analysis methods are proposed: reactive and generative. These terms are derived from two basic methods of thinking: event and systemic. Event thinking takes complex problems and breaks them down into a linear chain of events to make analysis more manageable. Event thinking focuses on detailed complexity and limits learning to event explanations. These characteristics are most useful for project analysis. Systemic thinking considers complex problems to have three levels of explanation: event (reactive), pattern of behavior (responsive), and systemic structure (generative). Systemic structure explanations are root cause explanations for the patterns of behavior observed. The reasons that root cause explanations are so important is that only they address patterns of behavior at a level at which behavior can be changed. These characteristics are most useful for network management. The standard structure for PMS conducts analysis as a linear chain of events whose products are not effective for managing large networks. The reason is that users cannot learn beyond the event explanation of their reactive analysis products. To be generative, PMS analysis methods must produce all three levels of explanations. This in turn makes it possible to manage networks by providing the information needed to control long-term funding levels and network condition, monitor the efficiency and effectiveness of proposed preservation programs and staff activities, and learn how economic efficiency can be improved by administrative and technical means.

This paper is based on management principles advocated by Senge in *The Fifth Discipline* (*1*). Senge, who is director of the Systems Thinking and Organizational Learning Program at MIT's Sloan School of Management, indicates that we are living in an increasingly complex world for which our formal education in linear thinking is no longer reliably effective. Complex systems, such as pavement management, require a shift from linear thinking toward systemic thinking. Linear thinking is most effective for solving problems that consist of detail complexity. But Senge indicates that there are two types of complexity: detailed and dynamic. In situations such as managing networks, for which cause and effect are subtle, effects over time are not obvious and the same action has different effects in the short and long runs, as in the case of using all short life treatments; here we have dynamic complexity. Conventional forecasting, planning, and analysis methods that are based on linear thinking are not equipped to deal with dynamic complexity. To deal effectively with dynamic complexity we must shift from linear to systemic thinking (*1*).

Network management is a complex issue that consists of the following levels of explanation: events (reactive), patterns of behavior (responsive), and systemic structure (generative). This paper proposes that the needs of network and strategic management systems are best served by analysis methods designed specifically for dynamic complexity, whereas project- and network-level analysis needs are best served by methods designed specifically for detail complexity. But the complex array of details that characterizes any management system distracts us from seeing patterns of behavior and the interrelationships among projects, preservation treatments, annual programs, networks, and trunkline systems. It appears, from proposed pavement management system (PMS) research needs, that it is necessary to devise increasingly complex solutions to increasingly complex management problems. The essence of systems thinking is seeing interrelationships rather than linear cause-effect chains—processes of change rather than snapshots (*1*). Systemic thinking methods simplify managing complex systems such as pavement networks, because they free us from detail complexity and help us see the deeper patterns behind the events and because they provide the ability to identify, understand, and control the vast array of interrelationships and patterns of change associated with pavement preservation.

This paper proposes that there are two categories of analysis methods for management systems—reactive and generative—and that AASHTO's PMS guidelines include only the reactive category needed for the detail complexity of project- and network-level analysis (*2*). It is also proposed that generative analysis methods are needed to provide pattern of behavior and root cause explanations needed for network and strategic management. The characteristics are described of generative analysis methods that differentiate them from the reactive methods that are in prevailing use. Network management primarily consists of dynamic complexity, and project and program development primarily consists of detail complexity. For this reason, two management systems are proposed, one for managing networks and one for managing programs. This necessitates linking the two systems with program development constraints.

DERIVATION OF TERMS

Event explanations are based on linear cause-effect thinking that focuses on breaking complex problems into smaller, easier-to-manage components that have less complex solutions (*1*). Senge refers to this process of thinking in terms of events as reactive. When reactive thinking is applied to developing analysis methods for management systems, the results are a series of component parts, the objective of each

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component being to reduce the number of variables with which the next analysis component must deal. An important characteristic of reactive thinking is the usual assumption that patterns of behavior are known. Event explanations are the most common in contemporary culture, and that is why reactive management prevails (*1*). For this reason, analysis methods that are reductive and based on event explanations are referred to as event or reactive analysis methods.

Pattern of behavior explanations focus on seeing longer-term trends and assessing their implications (*1*). They suggest how to respond to shifting trends over a longer term. Systemic structure or structural explanations focus on the underlying causes of patterns of behavior. The reason that structural explanations are so important is that only they address the underlying causes for patterns of behavior at a level at which behavior can be changed. Therefore, structural explanations are generative because only they enable us to create our own future. For this reason, analysis methods are referred to as generative when they deal with total systems, when they establish the patterns of behavior, and when they identify the underlying or root causes of patterns of behavior.

PAVEMENT MANAGEMENT VIA LEARNING OR NONLEARNING SYSTEMS

Learning systems provide all three levels of explanation (event, responsive, and generative), and nonlearning systems typically provide only event explanations. Therefore, management systems can be divided into learning and nonlearning systems, as illustrated in Figure 1. Learning systems consist of two separate systems. Policy makers and planners use the network management system to plan strategy, make policy, set the budget, monitor staff activities and programs, and control future network condition and funding requirements. Technical staffs use the program management system to select the combination of projects and treatments that meet program development constraints and maximize benefits at least program cost. The two systems are linked by program development constraints.

Nonlearning systems consist of network- and project-level analysis. Network-level analysis is used to determine network condition and the location of possible preservation projects. Project-level analysis is used to select the best treatment for each project. An optimization or ranking procedure is used to identify the best projects for the annual preservation program. This standard structure has no link between network and project levels that enables long-term control of network condition, budget requirements, and benefits. Furthermore, the operating characteristics of learning and nonlearning systems are totally different. For example, network management systems require detailed pavement condition data consisting of an inventory of each occurrence of distress by type, severity, and extent for 100 percent of the network. In contrast, network-level analysis needs only generalized pavement condition data based on a well-designed sampling plan.

LEARNING SYSTEMS	NON-LEARNING SYSTEMS
•	•
•	•
NETWORK MANAGEMENT SYSTEM	NETWORK LEVEL ANALYSIS
•	•
CONSTRAINTS	•
•	•
PROJECT MANAGEMENT SYSTEM	PROJECT LEVEL ANALYSIS
•	•
•	•
GENERATIVE ANALYSIS	REACTIVE ANALYSIS

FIGURE 1 Two fundamental categories of PMS.

REACTIVE AND GENERATIVE ANALYSIS METHODS

Reactive analysis methods used for nonlearning PMSs include the following:

- Combined index for pavement condition
- Project life-cycle cost analysis
- Network-level analysis to identify maintenance, rehabilitation, and reconstruction (MR&R) projects
- Project-level analysis to identify best MR&R treatment projects
- Design service life estimates and pavement condition assessments based on different criteria
- Decision trees to select treatments
- Expert systems to select treatments
- Optimization to provide "the Optima program"
- Optimization based on selected projects and the best treatment for each project
- Requirement for an operational PMS staff
- Performance model for each pavement classification
- One level of optimization
- Duplication or replacement of pre-PMS program and project development process

The purpose of these methods is to formalize, essentially duplicate, and perhaps extend the pre-PMS project and program development process. When implemented, PMSs based on reactive analysis methods usually become an integral part of the project and program development process.

Generative analysis methods are listed in the following:

- Separate remaining service life indexes for roughness, rut depth, friction, and distress
- Detailed pavement condition data required for 100 percent of the network
- Network analysis based on project analysis of 100 percent of the network
- Network life-cycle cost analysis
- Network strategy analysis
- Automated project analysis
- Design service life estimates and pavement condition assessments based on same criteria
- Feedback process conducted by pavement research staff
- Network, MR&R program, and project performance based on
  - Percentage of length in acceptable condition
  - Remaining service life

- Three levels of optimization
  - –Network: maximize condition/cost
  - –R&R program: maximize benefits
  - –Project: minimize cost
- Performance model for each uniform section
- Not a duplication or replacement of the pre-PMS program and project development process

Their purpose is to establish the patterns of behavior observed for each network, to determine the underlying causes for these patterns, to control long-term (20 to 40 years) network performance and funding requirements, and to provide monitoring information. Their primary products are program development constraints that enable policy-level control of long-term network performance, funding level, economic efficiency, and monitoring capability. Any network preservation program that may be proposed by the technical staff must comply with the constraints set for that network. The quality of proposed programs is based on quantified measures of efficiency. The role for generative PMS analysis for network management systems is illustrated in Figure 2 and explained elsewhere (*3*).

IMPORTANCE OF SYSTEM CHARACTERISTICS

It should be important when developing a new PMS, or evaluating an existing one, to decide what characteristics the system should possess. The characteristics could then be used as constraints for selecting and developing the analysis methods.

Table 1 presents the characteristics that describe the products of generative and reactive analysis methods. If the first PMS development step were to select the reactive or generative product characteristics, more than likely the generative would always be selected. The point is that regardless of whether our thinking focuses on structural or event explanations, most of us prefer the product characteristics of generative analysis methods. However, when developing analysis methods, normal habits of linear event thinking prevail and the system ends up providing products like those on the right side of the table.

The following sections provide specific characteristics that differentiate reactive and generative analysis methods for pavement management.

Learning or Nonlearning Products

The use of reactive analysis methods explains only events such as network condition, project condition, and best project treatment. Learning is usually considered a research function, not a function of management systems. Reactive systems are usually designed to formalize the project and program development process. In this way they are parallel to and aide and improve the program and project development process. Reactive analysis includes various decision support methods such as decision trees and expert systems that are used to replace an individual's subjective opinion. However, according to diffusion of innovation concepts (*4*), reactive analysis methods should provide little relative advantage over pre-PMS program and project development methods.

Generative analysis for network management requires automated project analysis of all uniform performing sections within each network. This requires complete high-quality pavement condition, cost, and physical inventory data so that the automated project analysis products are accurate. Automated project analysis of the entire network for all feasible preservation treatments provides a huge pool of information from which application software systems are able to provide information needed to answer any conceivable question about network preservation. Automated project analysis provides what Senge refers to as leverage.

The term leverage means "seeing where actions and changes in structures can lead to significant, enduring improvements" (*1*). The objective of generative analysis is to provide the

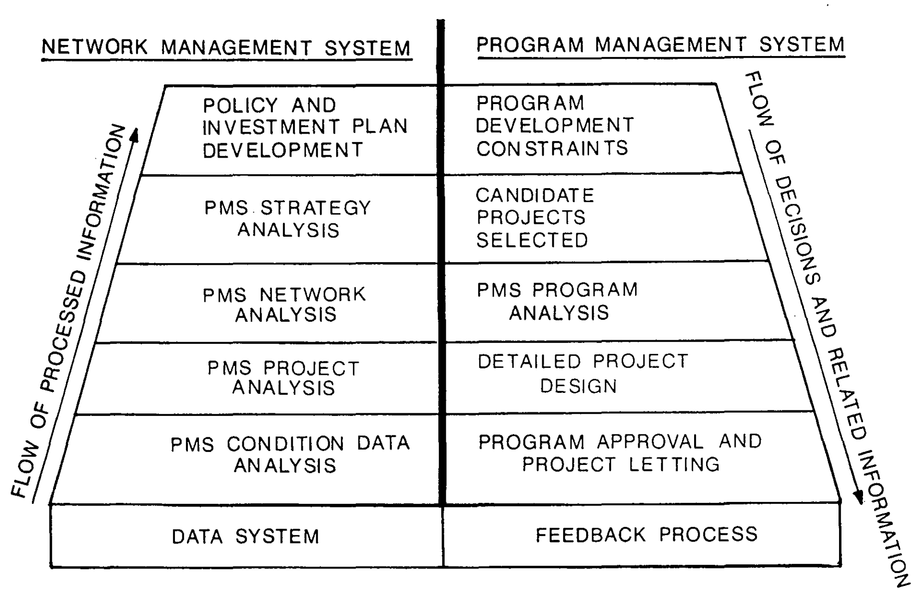

FIGURE 2 PMS structure for which decisions flow top down.

TABLE 1 Characteristics of Generative and Reactive PMS Analysis Methods and Their Products

Analysis Method	
Generative	Reactive
Learning	Nonlearning
Proactive	Reactive
Innovative	Noninnovative
State of the art	State of the practice
Generic	Agency-specific
Flexible	Inflexible
Simple	Complex
Decisions flow from top down	Decisions flow from bottom up
Cost-effective	Not cost-effective
Maximizes benefits	Does not maximize benefits
Truthful	Superficial

ability to see where high leverage changes are possible. For example, network (or program) life-cycle cost analysis (*5*) and strategy analysis (*6,7*) provide the information needed to see how improvements in long-term network condition can be accomplished with no increase in funding. This ability to learn from the system is illustrated elsewhere (Novak et al., unpublished data, 1993).

Generative analysis products are used by upper managers to learn what pavement preservation funding level would be required to meet long-term network condition objectives at the lowest network life-cycle cost. All three levels of explanation are needed for such a learning process. Examples of each level are as follows:

1. Event explanations include network condition and the lane-mile cost of available projects of a given cost-effective range.
2. Pattern of behavior explanations include network remaining service life and the long-term network condition resulting from a given network strategy (a network strategy is the lane-mile length and the average design service life of the annual program).
3. Underlying or root cause explanations include lane miles of pavement designed to be moved from each lower to each higher remaining service life category and the primary cause of network deterioration.

The information from generative analysis can be used with other information to arrive at informed decisions that enable accomplishments such as controlling and creating future network condition and funding streams, improving economic efficiency, reducing administrative overhead cost, determining the research projects that would most improve economic efficiency and program benefits, and analyzing the cost-effectiveness of staff activities such as pavement research and cost estimation.

Generative analysis methods are used by technical staffs to learn, via the feedback process, how to improve the accuracy of estimates such as project cost, benefits, and design and remaining service life. Feedback is a data processing activity that provides processed information that technical staffs need to improve economic efficiency. For example, research can use the primary cause of network deterioration as its primary research effort. If research is successful, it should lengthen network remaining life, improve network condition, and increase funding efficiency. The cost-effectiveness of the research staff is a function of the cost of research and the dollar value of the improved economic efficiency that it has produced.

Network management based on generative analysis provides relative advantage over the pre-PMS project and program development process. Advantages include a simplified and accelerated learning process, better communications between technical and manager staffs, the direct use of technology to attenuate the effects of inadequate revenues, and funding efficiency that is controlled by administrative users and improved by technical users.

Proactive or Reactive Products

Proactive refers to the ability to provide upper managers with the information needed to create the desired future network condition, associated funding streams, and investment benefits. Proactive, as used in this paper, does not refer to the aggressiveness with which agency problems (both internal and external) are attended to. To be proactive, a PMS must provide for decisions to be based on all feasible alternatives and to flow from the top down, for monitoring capability to ensure that constraints are followed, and for feedback to compare actual with estimated results. Generative analysis for network management provides the ability to control future network condition and funding streams rather than react to them. This gives managers (users) a relative advantage over pre-PMS methods since the long-term outcome of any feasible alternative funding or preservation scheme can be readily displayed by means such as simple bar charts and a network analysis chart (*8,9*). The technical staff also gains relative advantage in that the management system becomes a means by which to learn, to communicate with upper managers, and to use technology directly to improve economic and benefit efficiencies.

Generative analysis methods provide managers, designers, materials engineers, cost estimators, and research personnel with processed information and data that indicate what must be done to improve economic efficiencies, benefits, and effectiveness of available funds. The products of linear event analysis methods create a climate of compliance for which individuals pursue narrow goals. Generative analysis methods enable individuals to see beyond their self-interest and to have new energy and commitment to organizational learning and improvement.

Measurement of Economic Efficiency

Generative analysis methods are capable of measuring economic efficiency. Efficiency measures provide relative advantage over pre-PMS methods because managers are given the means to learn how to maximize network condition and the benefits derived from available funds. For example, few managers realize how nonuniform pavement performance is and how much this effects economic efficiency. Efficiency measures enable their users to comply with the economic efficiency objective of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA).

Efficiency measures also furnish monitoring capability and supply the reasons that proposed programs are not as efficient and effective as desired. The value that users derive from generative analysis methods can be measured in terms of the improvement in funding efficiency that takes place over time. Reactive analysis methods cannot provide this capability. The reality of reactive analysis methods is that they do the best they can with the data available; however, it is not known just how good that is.

Innovative or Noninnovative Products

Innovative refers to the ability of generative analysis methods to accommodate and facilitate creativity and innovation in the development and improvement of analysis methods and in the use of its products to develop alternative funding schemes. Noninnovative refers to products of reactive analysis methods that generally parallel the agency's pre-PMS methods. No new information, no new products, and no new means to use the analysis products to develop more creative and innovative preservation programs or funding schemes are offered. Nor do reactive methods give technical staffs the means to be more creative or innovative. The ability of generative analysis methods to enable all staff levels to be more creative and innovative gives them a relative advantage over current practice and reactive analysis methods.

Users of network management products are upper managers and planners, whereas appropriate technical staffs have responsibility for the quality and completeness of the data and analysis methods used and their products (*3*). For network management based on generative analysis methods, the three key variables are cost, design and remaining service life, and lane-mile length. These variables have equal meaning and importance to technical, planning, manager, and policy staffs; hence, communication among these staffs is simplified and improved. Even the influence of a material property as obscure as effective porosity can be traced to funding efficiency. Likewise, managers can trace less-than-desirable funding efficiency down to underlying causes such as bases that are subject to seasonal softening and their corresponding effective porosity.

To try out reportedly creative and innovative ideas should not be confused with actually being creative and innovative. Reactive analysis methods can accommodate working with and incorporating materials, methods, and ideas reported to be creative or innovative. However, reactive analysis products are not intended to foster creativity or innovation. Users of reactive analysis methods must therefore either comply or not comply with their analysis products. And since reactive analysis methods generally formalize and centralize the program and project development process, Kalia indicates that this generally inhibits innovation (*10*).

State-of-the-Art or State-of-the-Practice Technology

State-of-the-art technology refers to an analysis method's ability to use technology to improve economic efficiency. Reactive analysis many employ high-technology analysis methods such as linear programming to find optimal combinations of projects for the annual program. However, the use of high-tech methods does not necessarily result in good products or the efficient use of available funds. Generative analysis methods are designed so that the technical staffs can at any time improve the accuracy and reliability of the analysis methods. To do this, an agency's pre-PMS era pavement research, pavement design, cost estimating, and materials staffs must, in the post-PMS era, have full-time responsibility for the PMS feedback process (*3*). Research efforts can then be directed at problems that would make greatest improvements in economic efficiency and program benefits. Furthermore, unless automated project cost and design life estimates are reliable and accurate, it would not be possible to use generative analysis methods. Therefore, cost estimators and pavement designers must track the accuracy of cost and design life estimates and take corrective action as needed.

The exclusive use of reactive analysis methods stagnates the agency with state-of-the-practice technology and the associated inability to actually use technology to improve economic efficiency. This has created a problem that is pointed out by Hudson and Haas (*11*). Their concern is that "pavement management implementation experience suggests that many of the same problems found in PMS in the 1970s still exist in the 1990s." This reflects the nonlearning nature of PMSs that are now in use and that are based on reactive analysis methods.

Generic or Agency-Specific Analysis Methods

Network management based on generative analysis methods requires the following variables: lane-mile cost, design and remaining service life, and lane-mile length. It is not directly involved in the selection of treatments, projects, and programs. For these reasons, generative analysis methods are not agency-specific but generic, and they deal only with the objective aspects of network management. Generative PMS analysis methods are explained by Kuo et al. (*6*), and their generic nature is demonstrated by Novak et al. (unpublished data, 1993;*12*). To be generic, the analysis method must consist of only application software. A customized utility software system is needed to adapt application software products to each agency.

Generative analysis products for network management are used primarily by upper managers to develop preservation program constraints and to review the economic efficiency of proposed programs. This use ensures compatibility with the agency's existing operating procedures and organization and avoids problems with detail complexity that is associated with decisions about which treatments and projects to select, political and demographic considerations, and factors related to priorities and ranking methods that cannot be quantified.

Flexible or Inflexible Analysis Methods

Network management based on generative analysis methods is flexible since upper managers are totally unrestricted by it. The analysis products simply provide the outcome of any alternative funding or network preservation scheme. For example, what estimated reduction in administrative overhead cost should occur as a result of increasing the average remaining life of a network by 2 years? Or, what would be the best possible network condition that could be maintained with a given funding stream? Flexibility is also enhanced because decisions flow from the top down, as shown in Figure 2. Reactive analysis tends to be inflexible because alternatives are narrowed down until there are few left. Generative analysis allows projects and treatments to be selected in any way the agency chooses. The quality of proposed programs is measured in terms of their economic efficiency and the quantified benefits they provide.

Generative analysis methods are flexible because during the preprogram development process, managers can inquire into any proposed funding or preservation scheme and evaluate its pluses and minuses. Once the program constraints that provide the desired long-term network condition and funding stream are identified, the technical staffs are free to assemble alternative programs in any way they think is most appropriate. This in turn provides competitive opportunities. Upper managers have efficiency measures to determine the quality of proposed programs and to evaluate the cost-effectiveness of their staff. For example, a district whose preservation program's funding efficiency is 40 percent would require in-depth review if the efficiency of another district's program was 60 percent.

Simple or Complex Analysis Methods

Network management based on generative analysis methods provide simplicity not possible with reactive analysis methods. Much of the effort that goes into reactive methods deals with reducing the number of variables that must be considered in the next analysis step, in managing large volumes of data that are used for reference purposes and establishing performance curves, and in overcoming problems caused by not having complete, high-quality data for network and project analysis. Generative analysis requires more and higher-quality data on pavement condition, unit cost, and physical inventory than are used for reactive analysis. These data requirements are necessary to automating project analysis and getting accurate products. But this simplifies everything else. The many computations needed for generative analysis can be made in seconds, thanks to the brute-force capability of modern computers. And the problem of storing huge quantities of data products is avoided by converting the data to various matrices that are used for strategy analysis (*6*).

Generative analysis methods simply forecast the outcome of any given decision. Agencies can then use the system to track real outcome with that forecasted by the analysis method. This form of trialability (*4,10*) enables users to continue with current operational procedure while gaining experience and an understanding of its products and forecasting capability. New operational procedures can be phased in as a result of the learning process afforded by the trialability of generative analysis methods.

Generative analysis methods require the use of remaining service life (*13*) because it is a pattern of behavior of projects, programs, networks, and total systems and it must be controlled. The use of remaining service life provides simplicity because it has a linear relationship with time, because it provides a measure of the network's condition (percentage of network in poor condition is the same as the percentage of network with zero remaining service life), and because it simplifies relating the impact of alternative treatments, projects, and programs on the long-term condition and funding needs of the network. Network or program life-cycle cost analysis (*5*) is less complex than project life-cycle cost analysis and provides the following advantages:

- Managers have greater flexibility when establishing budgets and network condition objectives,
- It demonstrates how preservation programs can be made more economically efficient, and
- It provides the ability to measure funding efficiency and benefits of alternative preservation strategies and programs.

Network strategy analysis (*6,7,11*) provides a simple means to evaluate the network patterns of behavior resulting from any feasible alternative network strategy or funding scheme over a 40-year (or more) analysis period.

Top-Down or Bottom-Up Flow of Decisions

The generative analysis methods used for network management are indicated on the left side of Figure 2, and the agency-specific program management system is on the right side. The left side activities are automated, administrators and planners are its users, and program development constraints are its products. Constraints are program cost, design service life, lane-mile length, and benefit priorities. The right-side activities are conducted in conformance with program constraints using any methods that the agency desires. The quality of proposed programs is measured in terms of their efficiency and the benefits that they provide. This is a top-down program development process that is explained in more detail elsewhere (*3*).

Current thinking is that policy-level activities can be based on low-quality, incomplete pavement information with an increasing need for quality and completeness at the project and then research levels. The advantage of incomplete information of low quality is the freedom to do as consensus agrees is best with little fear of accountability. If revenues fall short of needs, this same incomplete, low-quality data can be used to justify proposals to increase revenues without fear that outside review could successfully challenge them. It is not that state highway agencies (SHAs) will try to raise revenues without justification. Instead, it is far more difficult to make more efficient and effective use of available funds than it is to secure needed revenue increases based on current state-of-the-practice methods and consensus. And nonlearning organizations are compelled to seek easy solutions to difficult problems.

Whereas most upper managers would probably prefer a top-down flow of decisions, such a flow brings with it accountability, greater responsibility, and the need for more technical and management skills. Reactive systems are, after all, designed to continually narrow down the alternatives. Managers generally do not realize how much this reductive process diminishes their decision-making prerogatives.

For example, in Michigan the director needed to establish the funding level for pavement preservation. Subordinates provided the following three alternatives: a budget level that (a) provided good system condition but required serious underfunding in other categories, (b) allowed adequate funding of the other categories but would cause the system to deteriorate to unacceptable levels, or (c) provided acceptable system condition and was affordable. Which funding level would you select, and who really made the decisions?

Good or Poor Cost-Effectiveness

Generative analysis enable users to determine the cost-effectiveness of the PMS and its various support activities. The automated project analysis method generates the huge pool of data previously described. This data source provides the total possible benefits and associated costs available within the system. It is similar to determining the total energy available in a unit of gasoline and using that as a basis for determining the efficiency of alternative engine designs. Many practical constraints prevent us from doing that which is theoretically possible. In addition, the ratio of that which is theoretically possible to that which is proposed to be done is a measure of efficiency. If, through the use of generative analysis methods, it is found that efficiency can be improved, the amount of improvement can be converted to the dollar value derived. Hence, the value or cost-effectiveness of generative analysis is easy to determine.

This is not so for reactive analysis methods. Likewise, the dollar value of improvements developed by research, design, cost estimating, materials, and so on can be calculated on the basis of the effect they have on funding efficiency. For example, if research enables funding efficiency to be improved by 1 percent, the value of that research is equivalent to 1 percent of the cost of the annual preservation program. This same idea applies to the other activities involved in the feedback process. The generative PMS also gives policy makers the reasons that proposed programs are not 100 percent efficient. The dollar value of a generative management system is measured in terms of the improvement in funding efficiency that it provides. Management systems based on reactive analysis provide little opportunity to quantify their dollar value or their return on the agency's investment in its development and operation.

Does or Does Not Maximize Benefits

The network management system's automated project analysis computes the benefits derived from all feasible treatments for all uniform sections that make up the network. For the program management system shown on the right side of Figure 2, a PMS program analysis software system is provided to enable the engineering staff to assemble and rank combinations of projects and treatments that best maximize benefits. To do this, users list enough projects for three or more annual preservation programs, and the software system assembles alternative programs that meet program development constraints and maximize benefits and then places them in rank order. This procedure is presented in the AASHTO guidelines (*2*) and is explained in detail in a paper now in preparation for the Third International Conference on Pavement Management. The abilities to maximize benefits, control long-term network condition and funding requirements, and learn how to further improve program benefits are not possible with reactive methods.

Reactive methods convert benefits to dollar value and discount them to their net present value. This is an inflexible approach since managers cannot emphasize different benefits in line with current social, economic, and political needs. As a result, management prerogatives are diminished since the benefits provided by selected programs are limited to that which the preselected candidate projects and treatments will provide. Furthermore, nothing will be learned of the relationship between the benefits provided by alternative programs and those that are technically possible.

Truthful or Superficial Products

It is important that management systems not deceive their users. The questions for which honest answers are needed address what is really going on out there and what will really happen if this or that alternative is chosen. Senge indicates that a commitment to the truth does not mean seeking the truth, the absolute final word, or ultimate cause. Instead, it means a relentless willingness to root out the ways in which we limit or deceive ourselves from seeing what is, and to continually challenge our theories of why things are the way they are. In this respect, generative analysis methods provide observability (*4,10*)—that is, the effects a given long-term funding stream and network strategy have on network condition can be observed each year and evaluated for accuracy, reliability, and the like. In this way, the reliability and accuracy of past decisions can be monitored and used to improve the current decision-making process.

Reactive analysis methods provide superficial products that are considered safe and acceptable: the projects, treatments, condition, and programs. The joy of reactive systems is that decisions can be made on the basis of data so general that outside sources cannot use the data later to question the wisdom of agency policies and objectives. The nature of management systems based on reactive analysis is in complete contrast to the accountability possible with learning systems and their generative analysis methods. Generative analysis methods cannot be developed and operated on superficial definitions and generalized data. Few agencies may wish to have the results of their decisions publicly scrutinized, which may eventually happen with the use of generative analysis methods. So what incentives are there to change the way we think about complex pavement management issues in order to make better use of federal funds?

SUMMARY

Based on learning organization principles presented by Senge (*1*), SHAs have one of two alternatives: to continue using the standard structure for PMS and reactive analysis methods, or to include generative analysis methods for network and strategic management purposes and reactive analysis for project- and network-level analysis needs. The second alternative requires two management systems—a network management system (for policy makers) and a program management system (for technical staffs)—and a link between the two systems referred to as program development constraints (see Figure 2). Reactive analysis methods are intended for analytical problems that involve detail complexity—that is, problems that can be solved by breaking them down into a linear chain of events and then solving each event independent of the other. However, this paper points out that when network management is conducted in this way, learning cannot go beyond event explanations. Policy makers are then left with no means to control future network condition and budgets; hence, their only choice is to react to the event explanations of the PMS analysis methods. This is the reason for referring to analysis methods that provide only event explanations as reactive.

Networks are systems that have dynamic complexity. For networks, the long-term cause-and-effect relationship between their performance and annual preservation programs are subtle, and because the same action can have different results in the short and long runs, there is dynamic complexity. Systemic thinking is that for network management, it is necessary for the analysis method to provide pattern of behavior and systemic structure (root cause), as well as event explanations. Pattern of behavior explanations focus on seeing longer-term trends and assessing their implications. Network remaining service life distribution is a pattern of behavior required for network management. It must be established on the basis of complete, accurate, and reliable pavement condition data. Systemic structure (root cause) explanations are the least common in pavement analysis and the most powerful. They focus on identifying the causes of the observed patterns of behavior. The reason root cause explanations are so important is that only they address the underlying causes of patterns of behavior at a level that behavior can be changed. When PMS analysis methods provide all three levels of explanation, policy makers then have the information needed to control their future in terms of network performance, budget requirement, and benefits.

The Michigan Department of Transportation's PMS is a network management system that is based on the generative analysis methods described in this paper. The system has been approved by FHWA and has been recognized to be ideally suited to the strategic planning process required by the ISTEA legislation. Because of the monitoring capability of network management systems, it is not necessary to have a program management system. An agency's current project and program development system could continue to be used in conjunction with the network management system.

CONCLUSIONS

1. Project and network level analysis are complex problems consisting primarily of detail complexity for which reactive analysis methods and their event explanations are well suited.
2. Network and strategic management are complex issues consisting primarily of dynamic complexity for which analysis methods must provide event, pattern of behavior, and root cause explanations.
3. The standard structure for PMSs described in the AASHTO's guidelines offer direction for developing analysis methods that deal with the detail complexity of managing projects and networks; however, they provide little guidance for developing analysis methods that deal with the dynamic complexity of network and strategic management systems.
4. It should be necessary for SHAs to use generative analysis methods, a network management system, and program development constraints, if policy makers are to control future network condition and funding requirements, improve the economic efficiency of available funds, and monitor the efficiency and effectiveness of their PMSs and the subordinate staffs involved in project and program development.
5. Generative analysis methods require complete, accurate, and reliable data on pavement condition, physical inventory, unit cost, and pavement design.

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Development of Delaware Department of Transportation Pavement Management System

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Preservation of the statewide roadway network, consisting of the vast majority of Interstate primary, secondary, and local roads in the state, is a key responsibility of the Delaware Department of Transportation (DelDOT). Because the pavement surfaces are (a) the primary link between the roadway network and the efficient movement of goods and services, (b) the portion of the network most visible to the traveling public, and (c) the most significant functional and structural components of the network, a systematic approach to their management is needed to provide the engineering and analysis tools required by decision makers. The process is described by which DelDOT pavement management activities were upgraded to be consistent with the FHWA Pavement Design Policy and the AASHTO *Guidelines for Pavement Management Systems*. The customized DelDOT pavement management system is designated the Pavement Management and Planning program; key features of it include unique and unambiguous milepoint referencing, dynamic segmentation, a decision-tree process to priority rank capital improvement and rehabilitation projects for annual programs, interim pavement performance forecast models based on currently available data, multiyear planning capabilities to forecast conditions and needs, and color graphics and mapping capabilities to illustrate current pavement conditions and projected conditions for various program scenarios.

The Delaware Department of Transportation (DelDOT), Division of Highways, is responsible for the maintenance of 7668 km (4,765 mi) of the 8666 km (5,385 mi) of public roads in the state. Of this mileage, 3561 km (221 mi) are multilane highways. Only 2348 km (1,459 mi) are eligible for some type of federal financial aid. Most of the necessary funds for construction, reconstruction, rehabilitation, and maintenance of these roads are allocated from the Delaware Transportation Trust Fund.

The statewide roadway network represents a tremendous investment. The preservation and management of these facilities are vital to the economy of the state and a key responsibility of the department. Increases in traffic, both in numbers of vehicles and in wheel loads, along with rising costs and reduced resources result in a significant challenge to administrative and engineering personnel. Because pavement surfaces are (a) the primary link between the roadway network and the efficient movement of goods and services, (b) the portion of the network most visible to the traveling public, and (c) the most significant functional and structural components of the network, their preservation and management at performance levels appropriate for desired service are major activities of the department. The changing emphasis from new construction to maintenance, rehabilitation, and reconstruction (MR&R) of existing pavements must be addressed.

A systematic approach to the management of pavements is needed to provide the engineering and economic analysis tools required by decision makers in making cost-effective selections of MR&R strategies on a network basis. Such an approach has come to be known as a pavement management system (PMS). The overall benefits attained from implementing a PMS include the planning and conduct of MR&R activities in a timely manner to preserve pavement surfaces and to provide for the most effective and efficient use of available highway funds. As described in the FHWA *Federal-Aid Policy Guide*, "the analysis and reporting capabilities of a PMS are directed towards identifying current and future needs; developing rehabilitation programs; priority programming of projects and funds; and providing feedback on the performance of pavement designs, materials, rehabilitation techniques, and maintenance levels" (*1*).

In response to an invitation from DelDOT, Pavement Consultancy Services, a division of Law Engineering, Inc. (PCS/Law), submitted a technical proposal for development and implementation of the Delaware PMS. The objective was to provide DelDOT with state-of-the-art tools for cost-effective management of the entire network of paved roads and streets under its jurisdiction.

PROJECT APPROACH

Significant project concepts that enhanced prospects for the timely accomplishment of objectives included the following:

- All activities were planned and conducted as a team effort involving DelDOT and PCS/Law project personnel. This approach provided for the accurate and realistic interpretation of Delaware PMS needs and the training of DelDOT personnel as work progressed. It also facilitated implementation by maintaining the cooperation of administrative and engineering personnel.

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• Funding, administrative, and operational constraints were recognized, and corresponding short-term, intermediate, and long-term objectives were identified early in the project. Accomplishment of short-term objectives produced early benefits and provided support for future enhancements to accomplish the intermediate and long-term needs and desires of the department.

• The pavement management software was developed and organized as a set of individual modules that can easily be replaced and updated. This approach facilitated the incorporation of advances in data collection and programming technology as well as intermediate and long-term enhancements.

• The initial phase of the project concentrated on determining DelDOT pavement management needs, desires, and anticipations at all levels from top administrative officials to district maintenance engineers. A detailed work plan for future conduct of the project was prepared on the basis of the established goals of the department.

PHASE 1 RESULTS

On the basis of an extensive review of documents and information obtained by many interviews and visits with DelDOT personnel during Phase 1 of the project, it was apparent that substantial elements of a conventional pavement management process were already being used for the selection and prioritization of projects for inclusion in the annual Highway Capital Improvements Program (CIP) and MR&R program. The existing process, however, was time-consuming because the required information was in different data files and analysis required both manual and computer efforts. Major deficiencies included the inability to forecast pavement conditions and needs and the lack of graphic reporting capabilities.

The needs, desires, expectations, suggestions, and objectives identified from interviews with both headquarters and district personnel were grouped into two categories: those appropriate for a conventional PMS and those beyond the scope of such a program. On the basis of a careful evaluation of the needs and desires that could realistically be accomplished with the available data, developed to implementation stage immediately, and completed within available time and funds commitments, recommendations were presented in the Phase 1 report for short-term, intermediate, and long-term objectives. The short-term objectives were those recommended for accomplishment during Phase 2 of the project. The intermediate and long-term objectives were proposed for accomplishment as supplemental projects.

PHASE 2 WORK PLAN

The detailed work plan for Phase 2 of the project focused on the overall objective of developing and implementing a customized DelDOT computer software package consisting of a user-friendly data base and analysis and reporting modules with emphasis on flexibility to permit ease of modification and updating.

The detailed project work plan prepared at the conclusion of Phase 1 identified the following activities to be completed under Phase 2.

Customized Delaware PMS

A major project activity was the development of a customized computer software package consisting of a user-friendly data base and analysis and reporting modules.

• Data base: the pavement management data base is the repository for all pavement and related information required to conduct the desired analysis and reporting activities. To establish the operational data base, a data base structure responsive to DelDOT needs must be determined and access and query routines for entering, examining, and editing the data base contents must be developed.

• Analysis and forecasting modules: the PMS must contain a powerful and versatile set of analysis and forecasting tools related to pavement condition, traffic, rehabilitation needs, and budget estimates. Specific modules developed for DelDOT included pavement analysis, traffic analysis, pavement condition forecasting, project ranking and prioritization, and multiyear budget projections.

• Report generation: the primary output (useful end products) from an operational PMS are various types of planning, priority ranking, scheduling, forecasting, and budgeting management reports. The DelDOT program contains a robust set of reporting options including tabular summaries, bar and pie charts, line graphs, and color-coded maps.

Implementation and Training

An important project goal was the maintenance of close coordination and communications between the project staff and DelDOT personnel during the program development, resulting in the implementation's being a continuous process. The implementation activities also included a review and evaluation of data collection equipment and procedures plus final software documentation and DelDOT personnel training.

DELDOT PAVEMENT MANAGEMENT FEATURES

The customized pavement management software developed under the project has been designated the DelDOT Pavement Management and Planning (PMAP) program. Customized PMAP features include:

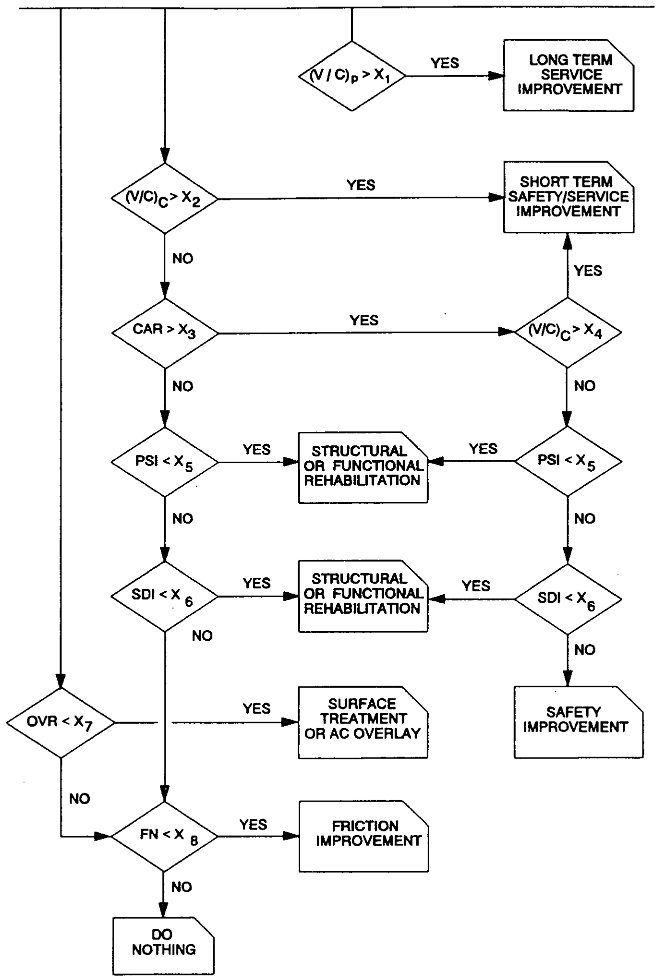
• A decision-tree process to priority rank capital improvement and rehabilitation projects for annual programs (Figure 1),

• Development of interim pavement performance models based on currently available DelDOT data,

• Multiyear planning capabilities to forecast needs and condition trends, and

• Production of color graphic and map reports illustrating current pavement conditions and projected conditions for various programming scenarios.

This paper describes the PMAP development process including road referencing, segmentation, performance models, safety and service improvements, data collection evaluation, and reporting capabilities.



Legend: $(V/C)_P$ = volume-to-capacity ratio, planning (gravity) model projection for long-term planning
$(V/C)_C$ = volume-to-capacity ratio, current for short-term program development
CAR = critical accident ratio (number of accidents per 0.3 mi road section per year in relation to average number of accidents per 0.3-mi section of similar traffic and classification)
PSI = present serviceability index (AASHO Road Test model at 80 percent ride/roughness and 20 percent distress)
SDI = surface distress index (subjective visual rating of combined distress types)
RCI = ride comfort index (use with SDI to determine PSI)
OVR = overall condition rating (local surface-treated roads only)
FN = friction number (measured with locked wheel friction tester at 40 mph)

FIGURE 1 Delaware PMAP decision tree process.

Road Reference System

A unique and unambiguous milepoint referencing system for all roads in the DelDOT network is critical to successful operation of PMAP. At the beginning of the project there was a general perception that the existing maintenance road number milepoint referencing system would serve this purpose. However, substantial inconsistencies were found in the use of the system in the various data sources such as inventory, traffic, and condition surveys. The same physical location was not always identified by the same milepoint, roads were missing from some data sources, directionality was not always identified, and treatment of divided and multilane roads was ambiguous.

To address these problems, a new standardized scheme for specifying road references was developed and implemented as part of this project. The road reference scheme was designed to satisfy the following four objectives: to (a) provide an unambiguous milepoint location reference along the roadway, (b) permit tracking of the changes or "evolution" of milepoint references over time due to alignment and other modifications along the roadway, (c) maintain compatibility with field milepoint measurements, and (d) retain consistency with current DelDOT practice to the maximum extent possible.

The first objective simply requires that there be an unambiguous, well-defined, unique correspondence between roadway milepoint references and the corresponding geographic location along the roadway alignment. This clearly is the first and critical requirement for any road reference scheme. In the Delaware PMS this is accomplished for each roadway via a roadway milepoint table that defines the complete one-to-one correspondence between reference milepoint values in the forward and reverse directions along the roadway.

Unfortunately, roadway alignments do not stay the same over time. As a consequence of road extension, curve straightening, changes from undivided to divided travel, and other construction activities, the road alignment and its associated milepoint references will change or "evolve" over time. It must always remain possible to relate historical inventory, accident, traffic, and other data to the new milepoint references in effect after construction (assuming that the historical location remains on the roadway alignment after construction). In the Delaware system, this is done via a set of milepoint evolution tables that document the historical changes in milepoint references along the roadway and permit mapping of past milepoint references to current milepoint locations.

Dynamic Segmentation

In addition to unambiguous referencing, a PMS must include some form of road segmentation for the organization of the various pavement attributes in the data base, the forecasting of attributes, and the prioritization of rehabilitation needs. The pavement network must be subdivided into homogeneous segments/lengths within which all relevant attributes such as pavement type and design, traffic, condition, subgrade and materials characteristics, and climatic conditions are treated as uniform. The values of these attributes vary along each roadway, and in many instances the attribute values also vary over time. As a consequence, the total number of segments will become quite large and the length of each segment quite small.

Dynamic segmentation, originally developed in the context of geographic information systems, is an alternative for organizing pavement network data that eliminates many of the problems inherent in the fixed segment approach and that can be applied generally to any highway pavement network that is uniquely referenced by road number and milepoint location (*2–4*). Given these advantages, dynamic segmentation was the clear choice for implementation in PMAP. In dynamic segmentation, each pavement attribute or set of related attributes is associated with a variable length segment of pavement by specifying a road number and beginning and ending milepoints. Beginning and ending points will generally

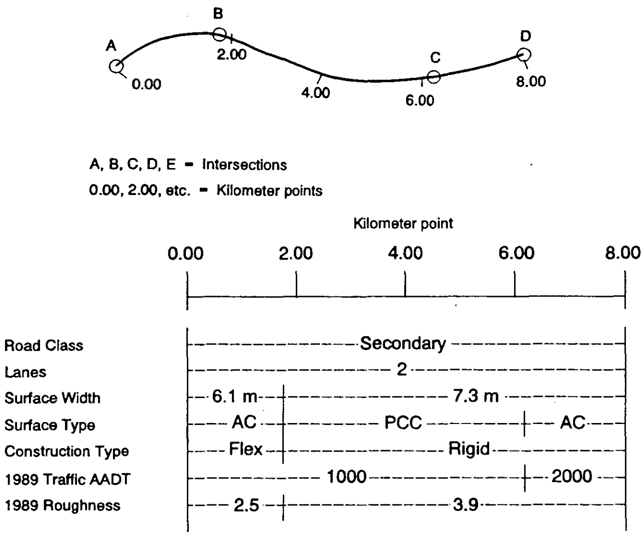


FIGURE 2 Example of dynamic road segmentation: *top*, Road 491; *bottom*, strip charts.

be different for different sets of attributes. The concept is best illustrated by an example.

Consider the hypothetical Road 491 sketched in Figure 2. (For simplicity, only a limited set of data are included in this example; for a real road, a much larger set of attributes would be stored for each segment.) The road is approximately 8.05 km (5.1 mi) long. It contains four intersections, labeled A through D in Figure 2. Road 491 was originally built as a two-lane farm-to-market road, with original construction consisting of a flexible asphalt concrete (AC) pavement with lane widths of approximately 3 m (10 ft). The length between Intersections B and D was later reconstructed as a rigid portland cement concrete (PCC) pavement with lane widths of approximately 3.6 m (12 ft). Later still, the length between Intersections C and D was overlaid with AC. Traffic and roughness are measured at 2-year intervals, 1989 being the most recent measurement year. Roughness is measured using automated equipment that records data at 0.3-km (0.2-mi) intervals. Strip charts showing the variation of an example set of pavement attributes along the length of the road are included in Figure 2.

Table 1 presents a summary of the inventory data for Road 491. Two segments are required to store the inventory data because of the change in lane widths at a point 1.93 km (1.22 mi) from the beginning. Table 2 gives the structural data for Road 491. Three segments are required here because of the change in construction type at 1.93 km (1.22 mi) and the change in surface type at 6.1 km (3.8 mi). Note that the breaks for the structural data segments need not match the breaks for the inventory data segments.

The traffic data for Road 491 are presented in Table 3. Only two segments are required because there is only one change in traffic volume along the length of the road. Note that the date of the traffic data measurement is also included in the data base to permit storage of a historical series. Table 4 gives a summary of the roughness data for Road 491. Roughness is quantified in terms of an arbitrary index ranging from 0 (poor) to 5 (good). The automated equipment used to measure roughness collects data at 0.2-mi intervals. Two segments of uniform roughness values are determined from the measured data points. The roughness values will vary along the length of the road, but the data can be aggregated into segments along which the roughness is "uniform" within a specified tolerance band.

The variable length segmentation given in Tables 1 through 4 for each set of attributes permits the pavement attributes to be organized in a more natural structure than is possible with conventional approaches. The segmentation for any one set of attributes is not necessarily congruent to the segmentation for any other set of attributes. Each set of attributes with its corresponding segmentation would be stored as a separate table under a relational data base scheme. Each table would be indexed using a sorted concatenation of the road number plus the year, if appropriate (e.g., for traffic and roughness), and the beginning milepoint.

For budget forecasting, however, the analysis algorithms still require segments along which *all* primary pavement attributes are uniform (secondary attributes—for example, curbing—can be allowed to vary along a segment). Logical segments meeting this requirement can now be easily constructed on the fly from the variably segmented pavement attribute data; this is the "dynamic" part of the dynamic segmentation scheme. Each component data table (Tables 1 through 4) is scanned to generate a master list of segment breaks for the road; this master list defines the analysis segments, and this set of segments remains in effect throughout the analyses and forecasts (and, in fact, until updates to the data base dictate regeneration of the segments). A summary of the three segments for this example is found in Table 5 (we assume here that all pavement attributes in Tables 1 through 4 are primary attributes for determining segment breaks—that is, there are no secondary attributes). Note that for each segment, all primary pavement attributes are constant.

In reality, only the segment location reference data (road number plus beginning and ending points in Table 5) would need to be stored separately for each segment, because the attribute data (all data to the right of the vertical bar in Table 5) can be extracted directly from the component data tables (Tables 1 through 4).

TABLE 1 Inventory Data, Road 491

Road Number	Kilometer Point Begin	Kilometer Point End	Road Class	Lanes	Surface Width (meters)
491	0.00	1.93	Secondary	2	6.1
491	1.93	8.05	Secondary	2	7.3

TABLE 2 Structural Data, Road 491

Road Number	Kilometer Point Begin	Kilometer Point End	Surface Type	Construction Type
491	0.00	1.93	AC	Flexible
491	1.93	6.12	PCC	Rigid
491	6.12	8.05	AC	Rigid

TABLE 3 Traffic Data, Road 491 (1989)

Road Number	Kilometer Point Begin	Kilometer Point End	Year	AADT
491	0.00	6.12	1989	1000
491	6.12	8.05	1989	2000

TABLE 4 Roughness Data, Road 491 (1989)

Road Number	Kilometer Point Begin	Kilometer Point End	Year	Roughness Index
491	0.00	1.93	1989	2.5
491	1.93	8.05	1989	3.9

Pavement Performance Models

The ability to develop annual programs based on current information and to prepare multiyear plans requires the use of forecasting models and curves. Families of models are generally developed for combinations of pavement types and designs, traffic levels, subgrade and materials characteristics, and environmental conditions. Development of the most reliable and specific models requires extensive inventory, design and construction history, condition history, climatic, and traffic data. Some of these data may be difficult to obtain, particularly the numbers of equivalent single-axle loads (ESALs) that the pavement has been subjected to since construction or rehabilitation. PMAP contains models for projecting traffic and forecasting the structural and functional condition of each pavement section based on the best available data for Delaware. Figure 3 is a family of interim models and curves to forecast surface distress index (SDI) for various types of pavements and age since new construction or last rehabilitation. Because of limitations on available data, these interim models do not distinguish between different traffic/ESAL levels. However, all PCC pavements are on roads with medium to high traffic levels, and all surface treatment pavements are on roads with low traffic levels. Provisions are included to consider low, medium, and high traffic/ESAL levels by the feedback process as future data are collected and entered in the data base.

Development of annual pavement reconstruction, rehabilitation, and resurfacing projects has been based on condition data that were 2 or more years old and might have contained some inaccurate information because previously programmed projects were not completed as programmed. And previously programmed projects were eliminated from the prioritized list manually. The PMAP program automatically updates the most recently collected traffic and pavement condition data to the program year using the forecasting models. It also automatically includes forecasted condition data for previously programmed sections. Provisions are being made for an up-to-date field inventory of completed rehabilitation and resurfacing projects by laptop computer to record actual rather than programmed details of the project.

Safety and Service Improvements

A major function of a PMAP is the capability to priority rank pavement reconstruction and rehabilitation projects for the next annual program and to forecast needs for multiyear planning purposes. The DelDOT Office of Planning prepares a Capital Improvement Program (CIP) each year that becomes the basis for the next fiscal year legislative allocations for all transportation projects. The CIP identifies multimodal planning studies; corridor/noncorridor road improvements; bridge replacement and rehabilitation projects; railway improvements; public transportation projects; pavement reconstruction, rehabilitation, restoration, and resurfacing projects; safety and drainage improvements; and aeronautics projects. Although this project was initiated to develop and implement a customized DelDOT PMS, PMAP as developed is much broader in scope. Besides ranking pavement projects, it identifies corridor/noncorridor road service needs on the basis of traffic projections and safety improvement projects. The road service needs are based on volume-to-capacity ratios computed for each segment using traffic capacity of the segment. The safety improvement projects are identified by the critical accident ratio, defined as the number of accidents per 0.3-mi road segment per year divided by the average number of accidents per 0.3 mi per year of similar traffic and road classifications. Combining these functions in PMAP avoids the programming of a pavement section for major reconstruction

TABLE 5 Pavement Management Segments, Road 491 (1989)

PMS Segment	Road Number	Kilometer Point Begin	Kilometer Point End	Road Class	Lanes	Surface Width (meters)	Surface Type	Const'n Type	AADT	Roughness Index
121	491	0.00	1.93	Sec'y	2	6.1	AC	Flex	1000	2.9
122	491	1.93	6.12	Sec'y	2	7.3	PCC	Rigid	1000	3.9
123	491	6.12	8.05	Sec'y	2	7.3	AC	Rigid	2000	3.9

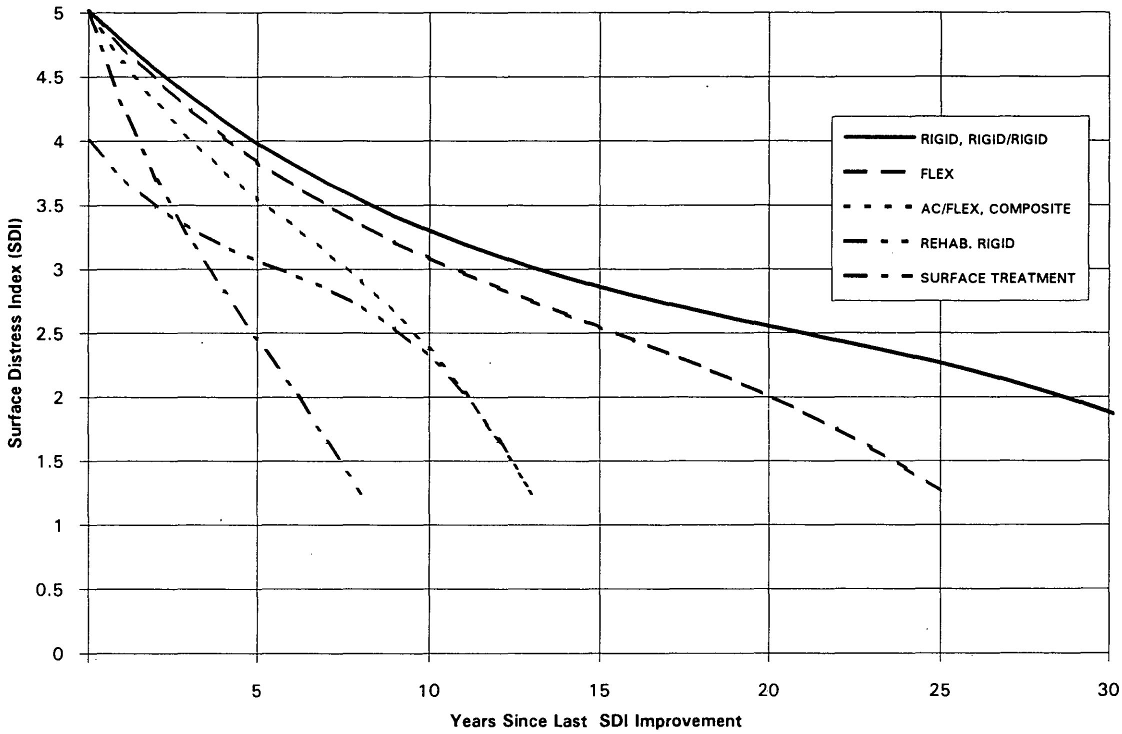


FIGURE 3 SDI forecast models.

or rehabilitation that will soon be in need of service or safety improvements.

Data Collection Evaluation

The reports produced by the PMAP are tools used by decision makers. The usefulness of these tools is greatly influenced by the quality of the data collected, entered in the data base, and used in the analysis process. The PCS/Law project included an evaluation of the current DelDOT pavement condition data collection equipment and procedures. This evaluation noted that pavement surface friction is collected with a locked-wheel trailer of the type specified by AASHTO 242-90 and that the resulting data are suitable for use in the PMAP. Pavement surface distress data are collected by visual (windshield) survey continuously. The distress type, severity, and extent are entered in a computer program by laptop keyboard and a SDI computed as a 0 to 5 statistic. The SDI data are adequate for use in the PMAP now, but use of video equipment for collection and analysis of pavement surface distress should be considered in the future.

The project identified the primary pavement condition data collection concern as the need for improved ride/roughness data for operation of the PMAP. Existing DelDOT data are the ride comfort index (RCI) collected by use of a trailer with a single accelerometer and a computer program for producing the RCI values of 0 to 5. A review of all RCI data collected in 1990 and 1991 indicates that more than half of all pavement sections in Delaware have an RCI of 1.5 or less. On the 0 to 5 scale, the 1.5 value would be a very rough riding pavement, but this is not consistent with the subjective evaluation of most roads in Delaware.

To provide some background information on ride/roughness data collection equipment and procedures, a small correlation study was conducted cooperatively between DelDOT, the University of Delaware, and PCS/Law. Thirteen pavement sections of various types and ride/roughness levels were selected for the study. Ride/roughness data were collected on each section with the North Atlantic Region Strategic Highway Research Program (SHRP) profilometer, the PCS/Law South Dakota type profiler, and the DelDOT trailer-mounted equipment using both the single and double accelerometer modes.

Various forms of international roughness index (IRI) statistics were computed from the profile data collected by all three pieces of equipment. Ride number (RN) values were also computed from the profile data collected by the SHRP profilometer and PCS/Law profiler. It should be noted that all profile data were subjected to a 152-m (500-ft) wavelength filter. The spacing between the wheelpath sensors is 165 cm (65 in.) on the SHRP profilometer and 175 cm (69 in.) on the PCS/Law profiler.

The accelerometers are mounted at the midpoint between the wheels of the DelDOT equipment, resulting in the IRI values' being based on a half-car simulation. The IRI values from profile data collected by the SHRP profilometer were calculated as left wheelpath and right wheelpath values using a quarter-car simulation and the mean of the two values used as the section IRI.

Table 6 contains the computed data for the test sections. The RCI values are computed from DelDOT data equipment operated in a single accelerometer mode and currently used by DelDOT as the pavement ride/roughness rating. Section 13 is an asphalt surface treatment pavement, and Section 16 is a rehabilitated concrete pavement. Profile data were not

TABLE 6 Computed Ride/Roughness Values

Type	Sec.	PI	RN (Eq. 1) (SHRP)	RCI (Eq. 2) (PURD)	IRI (PURD)	Mean IRI (LAW)	Mean IRI (SHRP)
Concrete	1	0.0385	2.09	0.526	4.72	4.28	5.49
	2	0.0290	2.54	1.694	3.11	3.36	3.35
	3	0.0336	2.32	0.710	3.87	4.20	4.01
	4	0.0340	2.30	0.589	3.06	3.44	3.22
	5	0.0070	4.19	3.600	1.34	1.56	0.85
Asphalt	6	0.0067	4.22	4.221	1.64	2.27	0.87
	12	0.0103	3.88	2.808	1.39	1.44	1.48
	14	0.0285	2.59	0.823	3.27	3.95	3.38
Overlay	7	0.0194	3.16	1.242	2.40	2.83	2.48
	10	0.0360	2.20	0.187	2.68	3.28	2.65
	11	0.0662	1.17	0.002	3.46	4.17	4.23
	13	0.0435	1.88	0.225	3.77	4.96	4.77
	16	0.0262	2.72	0.543	3.84	---	4.02

collected on Section 16 by PCS/Law equipment. Calculated values are generally based on an average of five sets of data per site. As indicated in Figure 4, there is reasonable good general correlation between IRI values computed from the pavement profiles collected by SHRP and PCS/Law equipment and the DelDOT equipment.

For use in the DelDOT PMAP program, it is recommended that the RN statistic for pavement ride/roughness replace the RCI currently used. Both RCI and RN are 0 to 5 statistics.

The following table contains correlations between RN and IRI on the basis of very limited data collected and analyzed for this paper:

Pavement Ride and Roughness Range	*RN*	*IRI*
Smooth	5.0 to 3.5	0 to 2.0
Medium	2.4 to 2.5	2.0 to 3.4
Rough	< 2.5	> 3.4

It is recommended that a more extensive correlation study be conducted using future data from the PMAP data base. The same values were calculated from the PCS/Law profiler data plus a half-car simulation IRI. The RN values were computed using the model

$$RN = -1.74 - 3.03 \log (PI)$$

from *NCHRP Report 275* (*5*), in which

$$PI = \{[RMS(P_r)]^2 + [RMS(P_l)]^2\}^{0.5}$$

where

RMS = root mean square of vertical acceleration,
P_r = measured displacement amplitude of right wheelpath for pavement wavelengths 0.5 to 2.4 m (1.6 to 8 ft), and
P_l = measured displacement amplitude of left wheelpath for pavement wavelengths 0.5 to 2.4 m (1.6 to 8 ft).

A Fourier analysis process was used to remove pavement profile wavelength content shorter than 0.5 m (1.6 ft) and longer than 2.4 m (8 ft).

It is acknowledged that this is a very limited study. However, Figure 5 does indicate the good correlation between RN

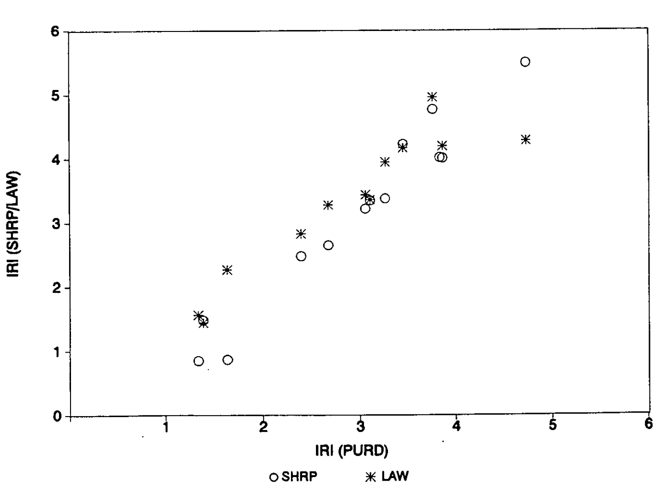


FIGURE 4 IRI from SHRP and PCS/Law equipment profiles versus IRI from DelDOT equipment profiles for all pavement sections.

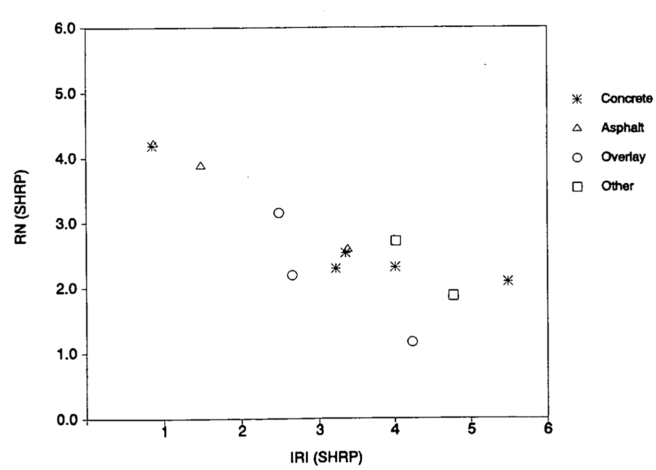


FIGURE 5 IRI versus RN values computed from SHRP profiles.

values, shown by previous NCHRP research to be highly correlated with mean panel ratings (the public perception of the ride quality) and IRI values, particularly at RN values of 2.6 and greater for different pavement types.

Report Generation

Standard reports (e.g. inventory data, budget analysis results) can be generated as tabular summary (spreadsheet) or detailed reports. In addition to standard reports, PMAP provides the capability to create "custom" reports interactively and to save the user-defined custom report formats. Any data element within the data base can be included in a custom report, and all formatting details (column headings, field widths, number of decimal places, etc.) are obtained from the PMAP data dictionary. All tabular reports can be viewed interactively on the screen, sent to the printer, and saved on disk in a form suitable for export to other programs or to other computer systems, including decentralized systems located within individual maintenance districts.

A wide range of color pie charts, bar/column charts, and *x-y* graphs can be generated within PMAP. The user has complete flexibility in defining charts: the user selects the subset of road segments to be displayed, the chart type, the parameters to be displayed along the various axes, and the details of the chart format (colors, headings, etc.). For pie charts, one data element is selected from the list, and the user specifies whether the pie chart is computed in terms of centerline miles, lane miles, or surface area (in either absolute units or percentage terms). For bar/column charts, multiple data series can be displayed on a single chart; one data element is selected for the category (horizontal) axis, and a second is selected to define the series.

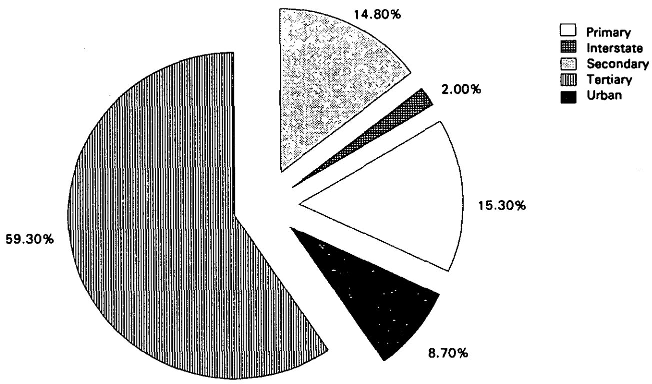


FIGURE 6 Federal aid classifications, all maintenance roads.

X-y graphs are typically used in PMAP to display forecast costs or conditions as a function of time. All charts can be viewed on the screen, sent to a color printer (e.g., HP PaintJet), and saved for later viewing or printing. Figures 6 through 8 are examples of color graphics produced by PMAP (reproduced in black and white in this paper).

Detailed color-coded maps summarizing any data in the PMAP data base can be quickly generated and examined using PMAP. The user selects the subset of road segments to be displayed, the data attributes to be displayed (e.g., "roughness index" and "AADT"), and other formatting details. Up to two attributes can be specified for each map: the first is displayed using color, and the second using line width. Complete zooming, panning, and labeling capabilities are provided. In addition, the PMAP user can point to any road segment with the mouse and activate an inquiry window summarizing the characteristics of that segment. Maps can be viewed on the screen, sent to color printer or a plotter, and saved to later viewing, printing, or plotting. Optional capa-

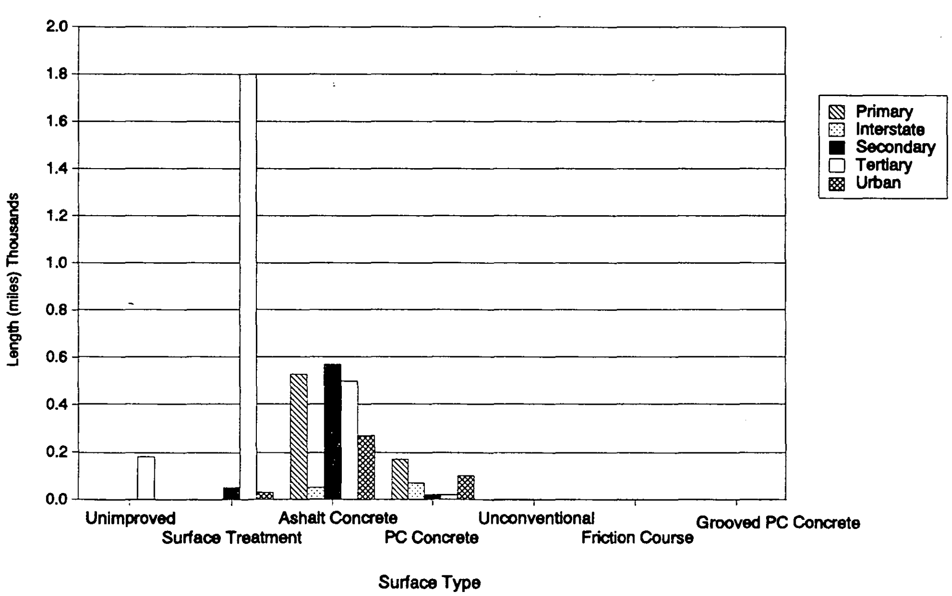


FIGURE 7 Miles of surface type by federal aid classification, all maintenance roads.

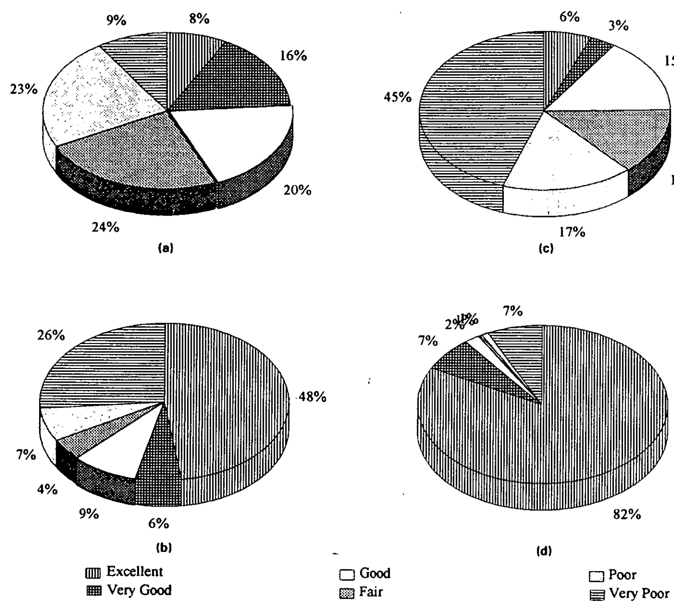


FIGURE 8 SDI, Maintenance Area 8, current condition and example funding scenarios: *a*, 1992 SDI; *b*, 1996 SDI, reallocation of funds; *c*, 1996 SDI, planned budget; *d*, 1996 SDI, increased budget.

bilities include the export of maps to other graphics and mapping programs (e.g., AutoCAD, MapInfo).

ACKNOWLEDGMENTS

This paper describes a project conducted for DelDOT under the FHWA Highway Planning and Research program. Scott D. Rabinow and Naim L. Bentahar, PCS/Law, were extensively involved in the PMAP software customization. Paul Smith, DelDOT, prepared the field test plan for the pavement roughness correlation study. The SHRP North Atlantic Region Coordination Office collected pavement profile data for use in the roughness correlation study. Data analysis for the roughness correlation study was conducted by the Delaware Transportation Center, University of Delaware, by Perincherry Vijayakumar under the direction of Robert Nichols.

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Seasonal Truck Volume Patterns in Washington State

MARK HALLENBECK

For many years states have collected year-round traffic volume information at specific sites in order to measure traffic patterns as they change throughout the year. Because of a lack of data that showed otherwise, it was assumed that all vehicle types had seasonal patterns that were similar to those observed for total traffic. In the 1980s advances in computer and sensor technology allowed states to expand this traditional data collection process to include volumes by vehicle classification and truck weights. The preliminary results of an analysis of the seasonal volume patterns for different vehicle classes in the state of Washington are presented. In Washington trucks have very different travel patterns from automobiles. In addition, the various truck types often have different seasonal patterns. The factors that are needed to convert 24- or 48-hr weekday vehicle classification counts to average annual daily estimates of truck volumes by classification vary from truck type to truck type and from site to site. As might be expected, higher-volume roads have more stable seasonal patterns. Lower-volume roads show greater variability from month to month and from year to year. The stability of seasonal patterns is also affected by the volume of vehicles within specific vehicle classes. In general, the greater the volume within a specific vehicle class, the more stable that pattern is from year to year. The lower the volume in a particular class, the less stable is that pattern. Examples of the common types of truck volume patterns are shown, the effects of two vehicle classification schemes on the patterns observed are described, and the implications of those patterns on geometric and pavement design are discussed briefly.

This paper summarizes the seasonal truck traffic patterns that were discovered when traffic volume trends were examined at 26 sites in the state of Washington. The work is being performed as part of an FHWA project entitled "Getting Better Truck Flows and Loads for Pavement Management." The project should be completed by mid-1993. Data for this paper were collected with four-bin vehicle length classifiers at 23 sites and weigh-in-motion (WIM) scales at three sites.

Because of variations in the lengths of vehicles within specific FHWA vehicle classes, the four length classes do not directly relate to the 13 FHWA vehicle classes. The contents of the four length categories generally include the following vehicles:

Bin	*Vehicle Type*
1	Cars, pickups, and short single-unit trucks (< 26 ft)
2	Cars and trucks pulling trailers, long single-unit trucks, and recreational vehicles (RVs) (26 to 39 ft)
3	Combination trucks (39 to 65 ft)
4	Multitrailer trucks (> 65 ft)

One to four calendar years of data were available at each of the 26 sites.

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COMPARISON AMONG VEHICLE CLASSES

One of the primary objectives of this project is to determine whether truck traffic volumes follow seasonal patterns that are similar to automobile traffic. If trucks are different, are seasonal adjustments specific to each truck class necessary to accurately estimate annual average daily truck volumes? To answer such questions, the research team developed normalized seasonal traffic flows for all four vehicle length classes. The normalized flows were computed for weekdays (Tuesday through Thursday), weekends (Saturday and Sunday), and complete weeks. One factor was computed for each vehicle class for each month. The factors were developed in accordance with the procedures described in a recently accepted report from AASHTO (*1*).

For the sake of brevity, only the weekday patterns are discussed in this paper. The remaining factors will be discussed in the project report. Weekday patterns are discussed in this paper because state highway agencies most commonly convert weekday counts (lasting from 1 to 3 days) to average annual totals when estimating annual conditions.

The project findings reveal that the four vehicle classes collected by the permanent length classifying equipment have very different seasonal patterns, regardless of the volume or functional classification of the roadway or the geographic location of the site. In general, the longer truck categories show less seasonal variation (i.e., month-to-month changes in daily traffic volumes) than the short truck and automobile classifications. In addition, traffic volumes of Bin 2 vehicles (mostly larger single-unit trucks and RVs) tend to vary the most by season. This variance appears to be attributable to the recreational vehicles in this category.

Figures 1 and 2 illustrate the differences in seasonal truck volume patterns among vehicle classes. The monthly volume patterns on these charts, shown as the ratio of monthly average weekday volumes (MAWDT) to average annual daily volumes (AADT), are typical of the patterns found at many sites. The exact locations and sizes of seasonal peaks and valleys often shift from site to site, but the basic shapes of the four curves are reasonably similar.

The characteristics and magnitudes of the differences in seasonal volume patterns for the vehicle classes are discussed in the following.

GEOGRAPHIC AND FUNCTIONAL ROADWAY DISTRIBUTIONS

One of the findings expected from this study was that the functional classification of the road and the location of each

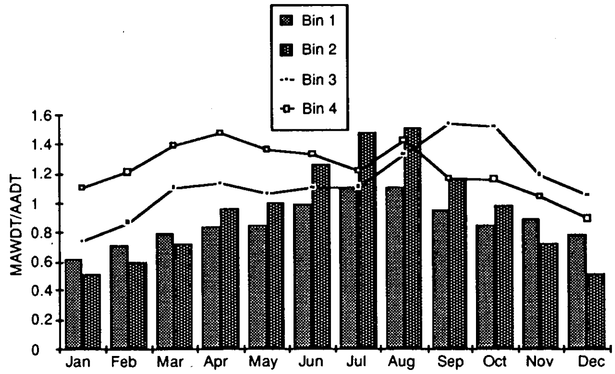

FIGURE 1 MAWDT/AADT for Site 61, 1991.

data collection site would significantly influence the traffic patterns observed at that site. This is indeed the case, and the findings are biased toward the geographic and functional distribution of the sites available for analysis.

Because 14 of the 26 sites (53.4 percent) are on the Interstate system, the project data base is heavily weighted toward the Interstate. The remaining sites are 10 principal arterials, 1 collector, and 1 minor arterial. Ten of the sites are within urban area boundaries; however, because of the relatively small sizes of some of these urban areas, some of these counters display traffic volume patterns that are more characteristic of rural recreational routes.

Because of the distribution of counter locations, the findings of this study are weighted toward the higher-volume rural roads in the state. Although a number of urban Interstate sites exist in the analysis data set, few urban arterial sections are instrumented with permanent vehicle classifiers. This reflects the fact that the Washington State Department of Transportation (WSDOT) operates few roads other than freeways in urban areas. (Local jurisdictions operate and maintain most urban arterials.) The overrepresentation of higher-volume rural roads in the analysis data base reflects WSDOT's concern with that group of roads. However, this distribution of equipped sites does limit the usefulness of conclusions concerning traffic trends on lower-volume rural highways and urban arterials.

In general, the higher the functional classification of the road is, the higher the traffic volumes in all vehicle classes. And the higher the traffic volumes are, the lower the variation in traffic levels from month to month and from year to year. Conversely, the lower the functional classification of the road is, the lower the traffic volume (particularly in the longer truck categories) and the more unstable the traffic volume pattern, both from month to month and from year to year. Some low-volume roads show reasonable stability in their traffic volume patterns, but a much greater degree of variation is present on these facilities.

The impact of location can also be seen in the traffic volume patterns observed in the data. For example, data from counters in areas subject to heavy recreational traffic show extreme seasonal patterns in Bin 2 vehicle volumes. Data from non-recreational sites may show minor volume increases in Bin 2 vehicles during peak recreational periods but not to the degree found at recreational sites. In agricultural areas, the longer truck categories show traffic volume peaks that are not present in other parts of the state.

The geographic influences change from one vehicle class to the next. For example, the recreational routes show increased automobile volumes (i.e., Bin 1) in the peak recreational periods, but not to the extent (in percentage terms) experienced by vehicles in Bin 2, which contains most of the recreational vehicles. Similarly, the two longer truck classes (Bins 3 and 4) are only minimally affected by the recreational peaks. Figures 1 and 2 show examples of these differences at two sites with fairly extreme seasonal variability.

The counter site that provided the data for Figure 2 is on a rural primary arterial near Washington's south-central border with Oregon. It displays the fairly high seasonality of the rural area, and the seasonal variation of longer truck classes (Bins 3 and 4) is much flatter than the seasonal variation for either automobiles or small trucks and recreational vehicles (Bins 1 and 2). The longer trucks counted at this site actually show a fairly high degree of variation in comparison with those counted at other locations because of an agricultural harvest haul that occurs in the late summer and early fall.

Figure 3 illustrates the volume patterns at a high-volume urban Interstate location. As expected, the seasonal volume patterns for all four vehicle classes show less month-to-month variation than those in the rural site in Figure 2, although recreational vehicle traffic still increases significantly during the summer months. At this urban site, the ratio of MAWDT to AADT for the two longer truck classes never falls below 1.0. This shows that the weekday traffic volumes tend to be

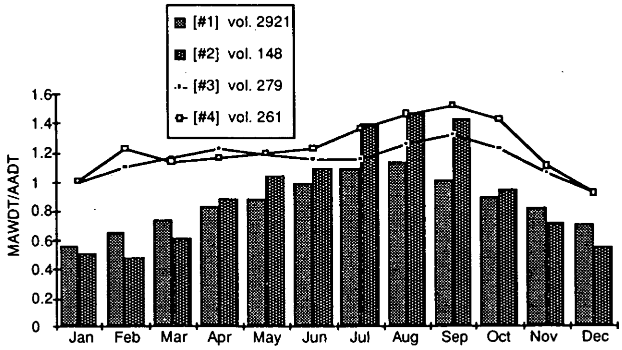

FIGURE 2 MAWDT/AADT for Site 41, 1991.

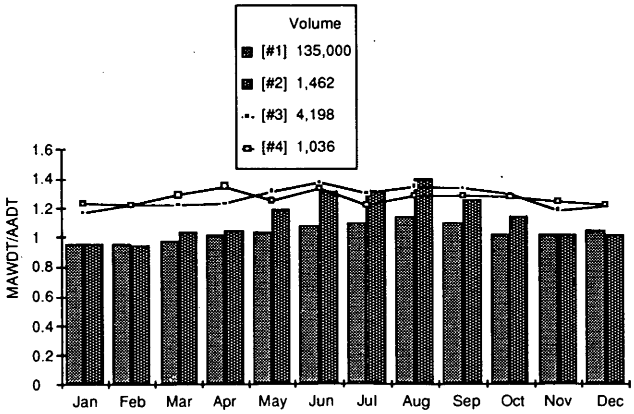

FIGURE 3 MAWDT/AADT for Site 809, 1989.

fairly constant throughout the year and that the weekday volumes tend to be consistently higher than the weekend volumes. Both of these facts are important in estimating average annual conditions from either weekday or weekend traffic counts.

Figure 4 illustrates some of the problems that occur when seasonal factors are calculated for lower-volume roads. In this case, the volumes of longer trucks are so small that relatively small changes in daily truck volumes cause the seasonal factor ratio (MAWDT/AADT) to reach fairly large values. For Figure 4, this ratio reaches 2.5.

With the low levels of traffic volume of a site like that of Figure 4 (AADT for Bin 4 is 14 vehicles per day), relatively small changes in volume significantly affect the computed seasonal factors. The results are highly variable seasonal factors, because the factors for any given month can be very different from year to year for a specific site, or between two similar sites. High variability complicates the search for groups of roadway sections that have similar traffic volume patterns, and it reduces the accuracy of AADT estimates produced with short-duration counts and seasonal adjustment factors. This problem is accentuated by larger classification schemes. That is, the FHWA 13-category classification scheme will produce a greater number of highly variable vehicle class seasonal factors than the four-length bin categories shown in Figure 4.

13-BIN VERSUS 4-BIN CLASSIFICATION SCHEMES

When the Strategic Highway Research Program introduced its revised traffic data collection plan, several state highway agencies indicated that they would use permanent 4- or 6-bin length classifiers to compute seasonal factors for 13-bin axle classification counts conducted with short-duration portable traffic counting equipment. This approach to seasonal factoring assumes that vehicles in the 13-bin axle categories follow volume patterns that are similar to the patterns found in the 4-bin data. It also assumes that all of the 13-bin truck categories fit cleanly into the length bins (i.e., that the axle bins are simply subsets of the length bins) and that each of the axle-based categories within a length bin follows the same pattern as that length bin (i.e., all of the FHWA categories that would be part of Length Bin 4 have similar seasonal patterns). All of these assumptions are also dependent on the length limits selected by each state for its length categories.

To test these theories, the seasonal volume patterns of the 13-bin vehicle categories were compared. WIM vehicle records were used to compute 13-bin daily volume records by vehicle class. These records were then used to compute monthly patterns based on the ratio of MAWDT/AADT. Figures 5 and 6 illustrate the resulting patterns for the truck categories (Bins 4 through 13) of the FHWA 13-bin classification scheme. As with the four length bins discussed, these patterns vary from site to site. Unfortunately, only three WIM sites in Washington had been operating for a complete year at the time of the analysis, so the differences in patterns caused by geographic and functional classification changes cannot be explored in this paper.

It is apparent from looking at Figures 5 and 6 that the 13 categories have very different seasonal patterns. This is particularly true if the categories containing RV traffic are compared with the categories that contain primarily commercial trucks. (In Washington, single-unit RVs tend to be classified as Axle Bin 4 "Buses," and vehicles pulling RV trailers tend to be classified as Axle Bin 8 "Four or Less Axle Combinations.") Recreational traffic has very high peaking characteristics, whereas commercial vehicle traffic has more consistent traffic volume patterns. Not surprisingly, the distribution of traffic between weekdays and weekends is also very different for the two types of travel.

For large commercial trucks (Axle Bins 9, 10, and 11, in particular), the seasonal factor MAWDT/AADT rarely falls below 1.0. This ratio reflects the fact that more commercial vehicle traffic occurs on the weekdays than on the weekends. Thus, even when some decrease in volume occurs in winter, the average weekday for the month is often higher than the average annual condition, which includes the lower weekend traffic volumes.

This phenomenon is not true for RV traffic. Much of the recreational traffic takes place on weekends, so with the exception of the summer months, the ratio of average monthly weekday to average annual condition tends to be less than 1.0. This pattern is illustrated best by Axle Bin 4 in Figure 5, which also shows an extremely high seasonal factor in July. This significant increase (the factor is greater than 4.0) is because of both the very large increase in RV traffic in the summer and the fact that the 4th of July holiday was on a

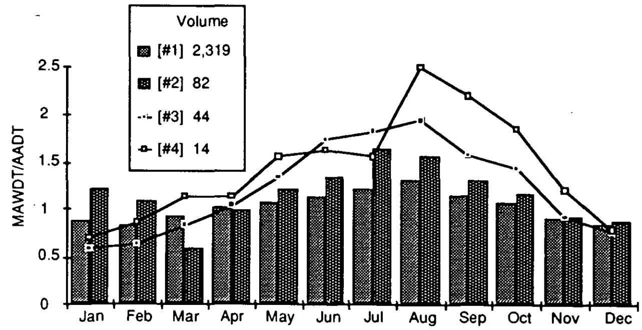

FIGURE 4 MAWDT/AADT for Site 820, 1990.

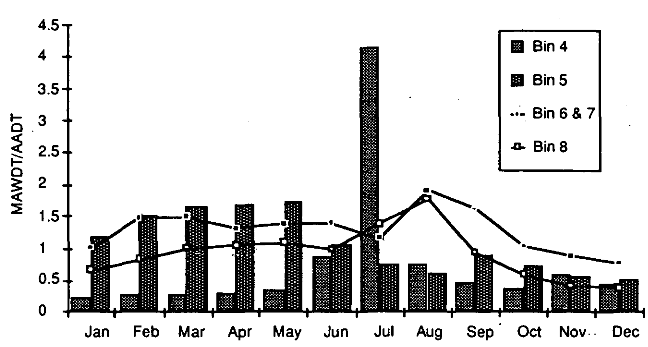

FIGURE 5 Seasonal traffic patterns of FHWA vehicle classifications for rural Interstate in western Washington, smaller truck categories.

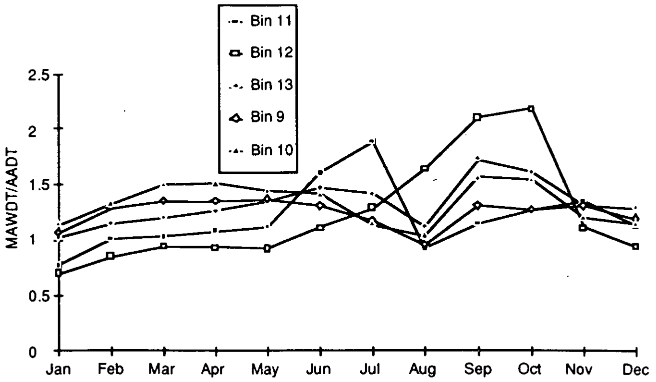

FIGURE 6 Seasonal traffic patterns of FHWA vehicle classifications for rural Interstate in western Washington, larger truck categories.

Thursday during the year illustrated. Thus, the "average" July weekday included a very high volume of RVs.

The very large seasonal pattern in this group is also attributable to the low volume of RVs on the average annual day: many RVs are normally on the road in the summer, but few are present during the rest of the year. Thus, the annual average volume is quite small. This makes the denominator in the ratio of MAWDT/AADT small, and consequently a relatively modest increase in traffic volumes can result in fairly large seasonal factors.

When these disparate vehicle class patterns are combined into a smaller number of categories (for example, the four length classes), the individual peak traffic movements shown in Figures 5 and 6 are "dampened"—that is, the monthly volume patterns change less from month to month. The large increase in RVs still produces a travel peak in July and August, but the MAWDT/AADT is much lower. Dampening occurs for two basic reasons. The first is that the patterns for different vehicle types have different peaks. Therefore, volumes for some vehicle types within a composite vehicle class increase, whereas others decrease or stay constant. Thus, in some cases, the absolute increase in traffic volumes is not as large as the increase for some vehicle types. The second reason is that even if the total volume increase is the same as or greater than that for any vehicle category, the combined vehicle group has a much higher total volume (the denominator in the ratio) than the individual vehicle category, and thus the computed factor is lower.

An example of the dampening effect can be seen in Axle Bin 8 in Figure 5. This axle bin contains both relatively large numbers of commercial vehicles (small tractor semitrailer combinations) and some RVs (primarily large vans and pickups pulling large trailers). The effect is that the lack of RVs in the winter months lowers the seasonal factor below 1.0, but the presence of commercial vehicles prevents that value from being very far below 1.0. In the summer, when large numbers of RVs are present, the seasonal factor increases well beyond 1.0, but because the volume of background commercial traffic is fairly large, the ratio of MAWDT/AADT (greater than 1.5) is considerably smaller than that in Axle Bin 4, the other vehicle class containing RVs.

The dampening effect can be significant if a vehicle class that is "different" from the other classes in its length bin is only a small proportion of the total volume for that combined classification. In this case, even an extremely large percentage increase in the vehicle category with smaller volumes is insignificant in comparison with the larger background traffic volumes. The result is a seasonal factor that reflects the total class, not the smaller vehicle category.

The primary drawback to this dampening effect is that it masks the actual vehicle patterns that are occurring on the road. But the dampening effect is not all bad. One of its significant advantages is that the seasonal factors for the larger vehicle categories tend to be more stable. Thus they are more capable of predicting total traffic volume. They simply do not estimate the vehicle mix within that volume with a high level of precision.

In summary, by combining specific vehicle types in larger classes, the volume patterns at a site tend to become more stable. However, this stability masks a variety of fluctuations in the volumes of specific types of vehicles. The end result is often a stable factor that does not accurately represent specific traffic volume patterns.

WEEKDAY VERSUS WEEKEND TRAFFIC

Another analysis examined the differences in traffic volumes between weekdays and weekends. This analysis included investigations of whether all classes of trucks had a specific weekly pattern and whether more truck traffic occurred on specific days. As indicated, the results showed that in most cases Saturday and Sunday traffic volumes differ significantly from weekday traffic volumes. In the majority of cases, weekday traffic volumes are higher than weekend volumes—especially for the longer truck classes, in which large commercial vehicles dominate. However, for classes with a high percentage of RVs, weekend volumes are consistently higher than weekday volumes.

As part of this analysis the project team also tried to determine the elements that constitute a "weekday." The researchers computed the average weekday three different ways, depending on the definition of the weekend/weekday split. (They computed and compared Monday to Friday weeks, Monday to Thursday weeks, and Tuesday to Thursday weeks.) The conclusion drawn from these analyses is that in some locations and in some months, the incorporation of either Monday or Friday in the weekday estimate is appropriate, and in other locations or months, traffic volumes on these days are statistically different from those of Tuesday through Thursday. For the sake of consistency, analyses performed for this paper assume that weekdays are only Tuesday through Thursday. This may be a conservative assumption, but the decision greatly simplified the performance of the analyses.

STABILITY OF FACTORS OVER TIME

The analysis of monthly to average annual traffic ratios over time showed that in general, the greater the traffic volume is on a road (or within a classification), the more stable the monthly ratio of weekday traffic to annual average condition. That is, average monthly traffic volume patterns for Interstates and heavily traveled principal arterials are reasonably

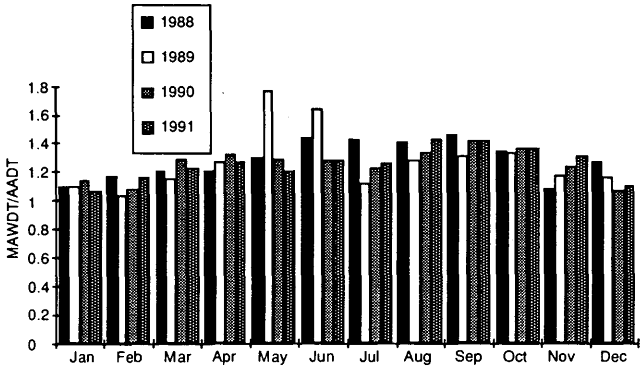

FIGURE 7 MAWDT/AADT for Bin 4 at Site 1, Interstate 5.

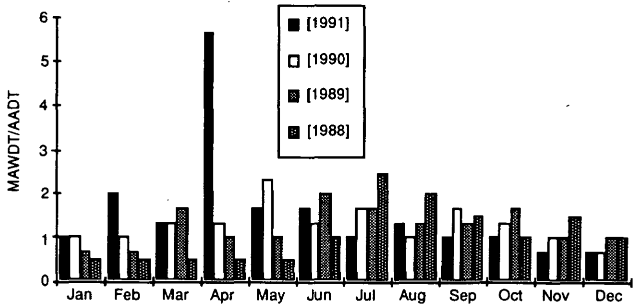

FIGURE 9 MAWDT/AADT for Bin 4, State Route 410.

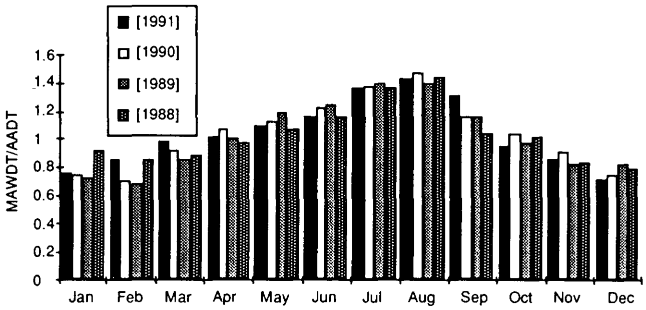

FIGURE 8 MAWDT/AADT for Bin 2 at Site 1, Interstate 5.

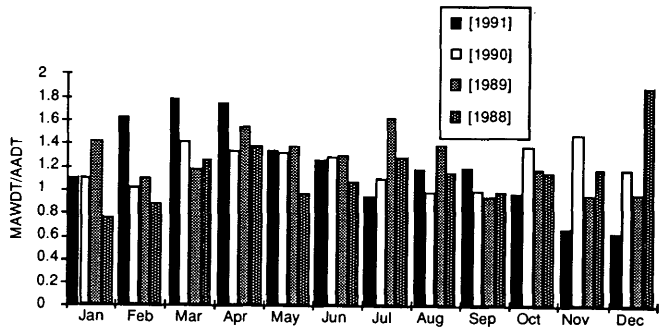

FIGURE 10 MAWDT/AADT for Bin 2, State Route 410.

stable over time (from year to year). Traffic patterns on lower-volume roads are often unstable from one year to the next. Although some low-volume sites have stable monthly factors, others have factors that vary considerably from year to year.

The actual monthly factors computed for low-volume roads may change significantly from one year to another, but the general volume patterns remain reasonably constant (for example, there is a consistent peaking pattern for each counting location that can be associated with summer or harvest period travel, but the timing and size of those peaks and valleys tend to vary from year to year.) The data also revealed that different roadways within the same geographic area or functional classification often have very different monthly factors, even though the shapes of their seasonal traffic volume patterns are similar.

Figures 7 and 8 show examples of changes in monthly to annual ratios from one year to another at a high-volume site. Figures 9 and 10 shows these ratios at a lower-volume site.

SUMMARY

The analyses described indicate that in most cases, an unadjusted 24-hr vehicle classification count is a poor estimate of average annual conditions. At most sites, an unadjusted 24-hr weekday count will consistently overestimate the average number of longer trucks traveling that road.

Except during the peak recreational travel periods, unadjusted weekday counts will significantly underestimate the average annual volume of RVs using the roadway. If counts are taken during peak recreational periods, weekday counts will overestimate the average annual RV volumes.

A comparison of the length Bin 1 patterns with the other three classifications shows that in most cases, the use of traditional seasonal factors to adjust short-duration truck volumes is inappropriate for estimating average annual truck volumes. The analyses show that during most portions of the year, the seasonal adjustments for different vehicle classes are different. Where the monthly adjustments for both total volume and individual vehicle classes are above or below 1.0, the factor based on total volume will improve the AADT estimate, although this improvement is rarely as good as that produced by a class-specific factor. Where one factor is above 1.0 and the other is below 1.0, the adjustment based on total volume will provide an estimate of total truck traffic that is worse than the unfactored volume estimate.

REFERENCE

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Revisions to Arizona Department of Transportation Pavement Management System

KELVIN C. P. WANG, JOHN ZANIEWSKI, GEORGE WAY, AND JAMES DELTON

The important aspects of the original network optimization system (NOS) used in the Arizona Department of Transportation (ADOT) are reviewed. The NOS has been an important instrument for the Highway Preservation Program since the early 1980s. However, no major updates have been conducted to the original NOS since its initial implementation. It was determined that there is a need to reevaluate the system since there are more than 10 years of pavement performance data available now and technology advancements in microcomputers. Several improvements should be made to the NOS model structure and the original transition probability matrices (TPMs). The factor of crack change was found to be insignificant in predicting the acceleration of pavement deterioration. Therefore, it was removed from the system. The effective rehabilitation actions were determined to be 6 instead of the original 17. In addition, new prediction models were established for all the road categories on the basis of the 13-year pavement performance data base in Arizona. The TPMs were modified with accessibility rules to improve the prediction of pavement performance. Pavement probabilistic behavior curves have been established and analyzed on the basis of Chapman-Kolmogorov equations. The new NOS structure improves the effectiveness and efficiency of the optimization. The enhanced NOS is implemented on a high-end microcomputer in the 32-bit operating environment. ADOT uses the new NOS to conduct financial analysis for more than 7,000 mi of highways with annual rehabilitation funding approaching $100 million.

A network optimization system (NOS) has been implemented by the Arizona Department of Transportation (ADOT) for more than a decade. It represented a significant advancement in applying operations research techniques to a pavement management system (PMS). An estimated $40 million was saved for the state of Arizona from 1980 to 1985 by using the results from NOS runs for the Highway Preservation Program (*1*). The capability of NOS to reliably conduct financial planning has been the driving force for ADOT's continued reliance on the instrument. This paper reviews the important aspects of the original NOS system and recommends revisions to NOS where deemed necessary. New transition probability matrices (TPMs) were developed to improve the reliability of the system. Pavement probabilistic behavior curves have been established and analyzed on the basis of Chapman-Kolmogorov equations. Accessibility rules were established to improve the Markovian prediction.

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INTERPRETATION OF ADOT PAVEMENT MANAGEMENT SYSTEM

In 1979 ADOT selected WoodWard-Clyde (WCC) to develop a PMS for the state highway network for programming and budgeting of highway preservation needs (*2*). The optimization procedure is unique among the existing PMSs. The elements of the ADOT PMS are the pavement management data base, the NOS, and report writing capabilities.

ADOT Pavement Management Data Base

The pavement management data base contains a record for each milepost of two-lane roads in the state and a record for each milepost in each direction for divided highways. There are 7,498 records, or sections, in the system. The fields in the data base contain location descriptors, pavement condition variables and historical information on traffic and maintenance. The pavement condition data include fields on the roughness, cracking, patching, rutting, flushing, and skid resistance. Each record contains the complete condition history of the section dating to the time when the data were first collected. The roughness data are collected with a Maysmeter and date to 1972. The cracking data are estimates of the percentage of the surface cracked and date to 1979. The data of rutting, patching, and flushes date from 1986, 1979, and 1979, respectively. The maintenance information includes fields for the most recent type of maintenance or rehabilitation project on the section.

NOS

The major features of the input data for NOS are the road categories, current condition of the pavements, TPMs, rehabilitation costs, infeasible actions, and condition standards. The output of the mainframe-based NOS enabled ADOT management to address the following questions on the basis of multiperiod NOS runs:

- What proportion of the pavements in each road category are expected to be in various condition states at the beginning of each time period?
- What is the most cost-effective rehabilitation program for the pavement network for each time period?
- What are the expected annual costs of pavement rehabilitation and routine maintenance?

Road Categories

Road categories are defined by

- Functional class—Interstate and non-Interstate;
- Traffic level—low, medium, and high;
- Region within the state—mountain, transition, or desert.

This produces 18 road categories. However, the low traffic level does not exist for the Interstate highways, thus the NOS uses 15 road categories. Road categories are treated independently by the NOS optimization procedure.

Condition States

The condition of the pavement network is defined in terms of the percentage of network that is in each condition state, defined as the following:

Factor	*Levels*	*Unit*
Roughness	<94, 94–142, >142	Maysmeter output (in./mi)
Cracking	0–10, 11–30, >30	Percentage of area
Cracking change	0–5, 6–15, >16	Percentage in 1 year
Index to first crack	1, 2, 3, 4, 5	N/A

The index to first crack was conceptually an estimate of the time between the construction or rehabilitation of the pavement to occurrence of the first crack. However, this index is used to select a TPM on the basis of the most recent rehabilitation. There are five levels of the index to first crack that are based on the type of rehabilitation treatment.

Roughness, cracking, and crack change are based on the observed condition of the pavement. There are 27 combinations of these factors. However, the combination of low-level crack and high level of crack change in 1 year is not feasible, resulting in 24 feasible combinations. When the five levels of index to first crack are considered, there are 120 combinations, as presented in Table 1.

Each pavement section in the network is placed in a road category and a condition state to define the characteristics of the population for the optimization process. Once these characteristics have been defined, the NOS operates with the percentages, or fractions, of pavements rather than considering specific pavement sections in the data base. For example, the NOS can determine the percentage of pavements in specific condition states that should be overlaid but not the specific sections of highway that need the treatment. NOS is capable of assigning actions to each mile in the system; however, this feature is not often used for project selection because of the impractical assignment of different actions to each mile.

TABLE 1 Condition State Numbering System

			INDEX TO FIRST CRACK, I_c				
R_o	C_o	C_p	INDEX 1	INDEX 2	INDEX 3	INDEX 4	INDEX 5
1	1	1	1	25	49	73	97
1	1	2	2	26	50	74	98
1	2	1	3	27	51	75	99
1	2	2	4	28	52	76	100
1	2	3	5	29	53	77	101
1	3	1	6	30	54	78	102
1	3	2	7	31	55	79	103
1	3	3	8	32	56	80	104
2	1	1	9	33	57	81	105
2	1	2	10	34	58	82	106
2	2	1	11	35	59	83	107
2	2	2	12	36	60	84	108
2	2	3	13	37	61	85	109
2	3	1	14	38	62	86	110
2	3	2	15	39	63	87	111
2	3	3	16	40	64	88	112
3	1	1	17	41	65	89	113
3	1	2	18	42	66	90	114
3	2	1	19	43	67	91	115
3	2	2	20	44	68	92	116
3	2	3	21	45	69	93	117
3	3	1	22	46	70	94	118
3	3	2	23	47	71	95	119
3	3	3	24	48	72	96	120

R_o: Roughness Level
C_o: Crack Level
C_p: Crack Change

Rehabilitation Actions

The NOS considers 17 rehabilitation actions, as given in Table 2. The first action is routine maintenance; it is assumed that all pavements that are not selected for a different rehabilitation treatment will receive routine maintenance. The second alternative, seal coat, is a preventive maintenance treatment and will not substantially improve the condition of a deteriorated pavement. The third treatment, asphalt concrete friction course (ACFC) is usually applied to improve skid resistance or roughness, although there will be a reduction in cracking also. The remaining treatments provide structural improvement. There are some differences in the actions available for the Interstate and non-Interstate roads. The costs of each of the rehabilitation actions are given in dollars per square yard (Table 2). They are updated annually or as needed.

TPMs

The performance model used in the NOS is based on TPMs. A transition probability, $p_{ij}(a_k)$, is the proportion of roads in

TABLE 2 Rehabilitation Action Table

ACTION	COST ($/SY)		I_c INDEX	TO STATES
	INTERSTATE	NON-INTERSTATE		
1. ROUTINE	0	0	*	1 - 24
2. SEAL COAT	1.19	1.20	2	25 - 48
3. ACFC	1.33	1.34	2	25 - 48
4. ACFC+AR	4.55	4.58	3	49 - 72
5. ACSC	2.59	2.61	2	49 - 72
6. AC +AR	8.96	9.02	3	49 - 72
7. 2"AC+FC	6.51	6.56	4	73 - 96
8. 2"AC+AR+FC	8.68	8.74	4	73 - 96
9. 3"AC+FC	9.10	9.17	4	73 - 96
10. 3"AC+AR+FC	11.27	11.35	4	97 - 120
11. #, **	5.19	6.42	3	97 - 120
12. #, **	10.44	9.02	4	97 - 120
13. #, ***	10.96	6.49	4	97 - 120
14. #, ***	11.86	8.46	5	1 - 24
15. #, ***	13.83	10.43	5	1 - 24
16. 4"AC+FC	11.69	11.77	5	1 - 24
17. 5"AC+FC	14.28	14.38	5	1 - 24

*: I_c in this category depends on the most recent action
ACFC is Asphalt Concrete Friction Course
AR is Asphalt Rubber
ACSC is Asphalt Concrete Surface Course
AC is Asphalt Concrete
FC is Friction Course
is removal-replace plus 2" AC for interstate with increasing removal-replace thicknesses
** is 2"AC plus Seal Coat, and 3"AC plus Seal Coat for non-interstate respectively
*** is removal-replace plus FC for non-interstate with increasing remove-replace thicknesses
I_c is the index to first crack

state i that move to state j in 1 year if the kth rehabilitation action is applied. It defines the probability of transition from one condition state to another in 1 year under one of the rehabilitation actions, including routine maintenance. The current matrix structure of transition probabilities in NOS consists of 15 road categories, 17 actions (including routine maintenance, seal coat, and 15 rehabilitation actions), and 120 states. The total number of matrices is 15 × 17 = 255.

All pavement sections, within a road category, are placed in 1 of the 120 condition states. However, since the index to first crack is based on the most recent rehabilitation action, a given condition state can transition to only 1 of the 24 condition states associated with the index to first crack in 1 year under routine maintenance, as given in Table 1.

The concept of the transition between condition states is shown in Figure 1. After construction or reconstruction, a pavement remains in Condition State 1 to 24 until an action other than routine maintenance is applied. Once a nonroutine maintenance action is applied, a new index to first crack is defined and the condition state of the pavement is restricted to 1 of the 24 condition states associated with that index. This structure prohibits a pavement that has received a nonroutine maintenance action from entering Condition States 1 to 24.

In the year in which a nonroutine maintenance action is applied, a transition matrix is used to predict the proportion of pavements in each of the 24 condition states associated with the index. Generally one would expect that a very high percentage of the pavements would be transformed to the best-condition state. For example, in Table 2 the index to first crack is 5 for a 5-in. overlay with a friction course. Table 1 shows that the condition states for this index are 97 to 120, with 97 being the best condition state. The probability approaches 1.0 that this treatment would result in a pavement in Condition State 97. Seal coat and friction courses generally will hide cracks for 1 or 2 years, but seal coat will not improve roughness. These treatments have an index to first crack of 2. The most probable condition states following these treatments are 25, 33, and possibly 41.

In summary, for each of the 15 road categories there is one TPM that is 120 × 120, grouped in five blocks of 24 × 24, for the routine maintenance action. In addition, there are 16 TPMs for the nonroutine maintenance actions; these matrices are 120 × 24.

For the development of the NOS, regression equations were derived from a sample of pavement performance data (*3*). TPMs could then be calculated from the regression equations.

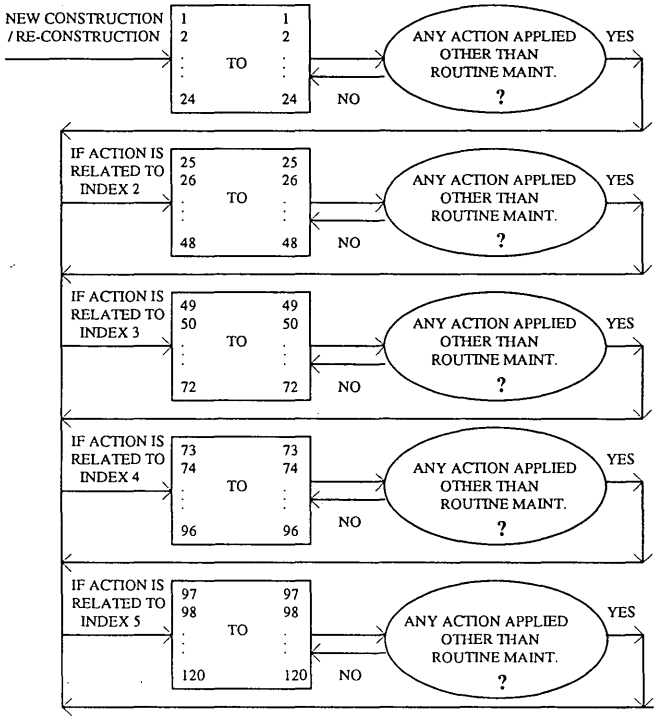

FIGURE 1 Flow chart of transition process of pavement condition states.

Regression equations were developed for changes in

- Roughness under routine maintenance,
- Amount of cracking of newly constructed roads under routine maintenance,
- Roughness following an overlay, and
- Cracking following an overlay.

The methodology of calculating transition probabilities for a given combination of traffic volumes and regional factors was detailed by Kulkarni et al. (*2*).

Infeasible Actions

Initial testing of the original NOS demonstrated that Rehabilitation Action 3, ACFC, was selected a disproportionate amount of the time. This was due to the inability of the TPMs to distinguish between the long-term performance of the ACFC and structural overlays. Therefore, the input to the program was modified to allow the user to prohibit the consideration of certain actions for certain condition states.

Condition Standards

Condition standards define acceptable levels of pavement condition to meet the needs of the traveling public; they are set by management policy. The user of the NOS inputs the minimum percentage of sections that should be in good condition and the maximum percentage that can be in poor condition for each of the traffic levels for Interstate and non-Interstate highways.

Optimization Algorithm

The NOS uses a linear optimization method coupled with the Markov chain concept for minimizing the overall costs of preserving the highway network to a set of specified standards over a planning period. The techniques of using linear programming and the Markov chain concept were initiated by Manne and by Wolfe and Dantzig in the early 1960s for large systems (*4,5*). They were subsequently adopted by WWC and ADOT to solve highway network investment problems (*2,6*). Hillier and Lieberman describe the basic model setup of this linear programming formulation (*7*). The transition process of pavement condition state conforms to the finite-state Markov chain process.

Two stages are needed to complete the optimization process. Let $w_{i,k}^l$ denote the proportion of roads of a given road category that are in condition state i at the beginning of lth time period of horizon T, and to which kth preservation action is applied. $w_{i,k}^l$ is time-dependent and reflects the behavior of the system in response to selected rehabilitation strategies; $w_{i,k}$ reflects the steady-state condition of the system under a fixed level of funding for rehabilitation and is therefore time-independent. The $w_{i,k}^l$ and $w_{i,k}$ are the two key variables in the process of setting up the short-term and long-term (steady-state) highway preservation policies. On the basis of the transition matrices and other constraints, $w_{i,k}^l$ and $w_{i,k}$ can be determined through the linear programming process. The core of the optimization model lies in the following two transition equations for the two stages of optimization, respectively:

First stage: steady-state problem

$$\sum_k w_{i,k} = \sum_{i,k} w_{i,k} \cdot p_{ij}(a_k) \qquad (1)$$

Second stage: multiperiod problem

$$\sum_k w_{j,k}^l = \sum_{i,k} w_{i,k}^{l-1} \cdot p_{ij}(a_k) \qquad \text{for } 1 < l \leq T \qquad (2)$$

RECOMMENDATIONS FOR IMPROVING SYSTEM

The development of the ADOT PMS was a significant advancement in using new technologies and was recognized nationally in 1982 (*6*). However, the current state of the art and data bases that have subsequently become available provide the means to revisit the original developments to determine whether revisions are warranted. Since the heart of the NOS analysis method is the TPMs, these are examined in this paper.

The regression equations were the basis for the generation of the TPMs. Because of inadequate data, sample data were used to build regression equations instead of using actual pavement performance data to generate transition probabilities. It was also assumed that the probabilistic behaviors of condition transition of pavements for both Interstate and non-Interstate highways were the same.

Four factors are used to determine pavement condition. Three of the four factors are related to pavement structural capabilities: percentage crack, crack change, and index to first crack. Only one factor, roughness level, is used as the measurement of ride quality. However, pavement rehabilitation strategies are dominated by ride quality rather than structural soundness (*8*). The review of the existing system also indicates that the NOS problem size is probably excessive. In addition, the existing levels for the boundaries defining condition states are not representative of the levels used by the engineering staff for determining rehabilitation needs or actions. Therefore, new levels of pavement classification are needed. In this paper, the TPMs were evaluated with respect to long-term behavior and new TPMs were developed on the basis of pavement management data base.

Under the original system, poor pavements can transition to good condition under routine maintenance. This unrealistic phenomenon is attributed to the assumption during the development of the original NOS that the transition probabilities conform to normal distribution. As shown in Figure 2, there is a probability, p_{ih}, under routine maintenance for a pavement, whose roughness value is within the medium roughness level, to transition to the low roughness level. Defining the transition probability for any pavement in the medium roughness level to transition to a low roughness level under routine maintenance requires integrating the specific probabilities, such as shown in Figure 2, within the limits that define the levels. This unrealistic behavior of transitioning to a lower roughness level under routine maintenance does not occur in the field during a long observation period, so accessibility rules were introduced to prohibit some of the transitions from occurring in the model.

Reducing Size of TPMs

When the structure of the condition states was set up in the early 1980s, there was little information on the crack change in the pavement management data base. During the development of the system it was assumed that crack change would play an important role in predicting pavement structural deterioration rate. However, examining the pavement performance data base shows that crack change of more than 5 percent is a rare event (Table 3). Only 4.2 percent of Interstate and 6.5 percent of non-Interstate sections had a crack change, from one year to the next, of more than 5 percent. In addition, the occurrences of crack changes over 15 percent occurred in less than 1 percent of the records.

Table 4 demonstrates that more than 5 percent crack change in one year does not indicate that there will be a high level of crack in the following year. This is in conflict with the concept that the rate of distress development increases as the pavement deteriorates. The failure of the data to demonstrate an increasing rate of deterioration could be attributed to the 5 percent level of crack change used in the analysis. However, the deviations of visual examination of percentage cracking can be as high as 5 percent at the same location either by different field crews or at different times within 1 year. This deviation can be even higher when the pavement is highly cracked. For example, when a pavement is 20 percent cracked, it is very possible that the visualized percentage crack range is between 15 and 25 percent. Therefore, the analysis based on the 5 percent level of crack is reasonable.

Further evidence is illustrated in Figure 3. The data of percentage crack for the Interstates were averaged on a yearly basis for 15 years. They show that there was an average of 4½ years between the rehabilitation and the occurrence of

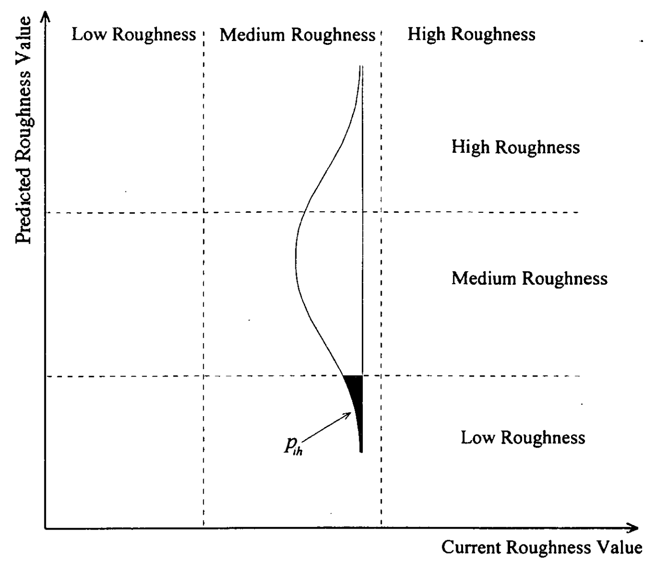

FIGURE 2 Concept of using normal distribution to define TPM for factor of roughness in original NOS.

TABLE 3 Percentage of Records in First Crack

Crack Change (%)	% of Total Record-Year (Interstate)	% of Total Record-Year (Non-Interstate)
0 to 5	95.90	93.50
6 to 15	3.80	5.90
Over 15	0.30	0.6 0

TABLE 4 Percentage of Records in Consecutive Multiyear Crack Change Over 5 Percent

Multi-Year Crack Change Over 5%	% of Total Record-Year (Interstate)	% of Total Record-Year (Non-Interstate)
Consecutively Two-Year	0.40	0.57
Consecutively Three-Year	0.03	0.06
Consecutively Four-Year	0.00	0.00

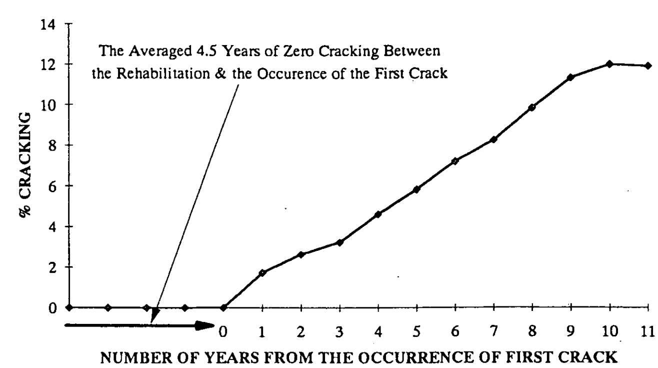

FIGURE 3 Average percentage cracking over time after rehabilitation for Interstate highways.

the first crack for the Interstate network. When the percentage crack increased over the next 11 years, as shown in Figure 3, the relationship of crack change over time is approximately linear. The rapid pavement structural deterioration after the development of first crack was not observed in this figure.

Therefore, from this analysis, it is evident that crack change, as defined in the existing system, is not an important indicator of the acceleration of pavement deterioration. The new structure of condition states without considering crack change is given in Table 5.

Reducing Number of Rehabilitation Actions

There are 17 rehabilitation actions in the original NOS. The initial concept of using this number of actions was to provide guidance for the pavement design process to select the "best" overlay design strategy. On the basis of the effectiveness of the action in the year of application, there are three categories of action: routine maintenance, light treatments, and heavy treatments. Light treatments have an index to first crack of 2 but the initial effect of the treatments in this category varies depending on the type of action—that is, a seal coat does not improve roughness but ACFCs and asphalt concrete surface courses improve roughness. Heavy treatments, with index to first crack of 3 to 5, have a high probability, approaching 1.0, of improving the pavement to the best condition state. In the NOS all actions with a particular index to first crack use the same transition probabilities under routine maintenance. Therefore, the NOS only models the behavior of five action groups under routine maintenance, one for each index to first crack. This restricts the NOS to selecting the least-cost actions for the heavy treatment categories. The difference in the initial condition within the light treatment category enables the NOS to distinguish between the effectiveness of the seal coats and the other light treatments. Therefore, the NOS can optimize only on 6 actions, not 17. Experience with running the NOS supports this conclusion. The infeasible actions input to the NOS were used to restrict the system's use of actions that were deemed inappropriate for certain condition states. Therefore, since the model can select between only six actions, the structure of the model can be simplified by eliminating 11 actions without compromising the effectiveness of the model. The new rehabilitation actions are presented in Table 6. Note that the new actions list does not distinguish between Interstate and non-Interstate.

Roughness and Cracking Level Boundaries

The existing roughness and cracking classification levels for NOS were based on the information available in the early 1980s. However, the pavement performance data show that these levels are no longer appropriate. For example, Crack Level 2 represents 11 to 30 percent crack in the pavement and is currently used in the NOS as the medium crack level. However, pavements at a crack level of more than 10 percent are not in an acceptable condition state. In addition, a Maysmeter value of 90 is too rough to be considered in the good category, as is the case with the existing NOS system. And the existing classification puts a pavement at 10 percent crack and Maysmeter number of 80 into the best condition state, which no longer can be viewed as a good pavement by today's engineering practice.

Therefore, two sets of pavement condition state criteria are needed for Interstates and non-Interstates respectively. The definition of the new classifications should be based on the current pavement condition. In addition, on the basis of the ADOT pavement design practice, pavements with serviceability indexes (SIs) of less than 3.0 for Interstates and 2.5 for non-Interstates are considered to be in the poor condition. Therefore, pavements with SIs of less than 3.0 and 2.5 were classified to be in the high roughness category for Interstates and non-Interstates, respectively. It is generally assumed that an Interstate pavement with an SI higher than 3.5 is in good condition. Therefore, Interstate pavements with SIs higher than 3.5 were classified to be in the low roughness level. For the same reason, non-Interstate pavements with SIs higher than 3.0 were classified to be in the low roughness level. Equation 3 shows the correlation between the SI and Maysmeter numbers:

$$SI = 0.3488 + 4.6836 \cdot 0.9970^{(R-4.255)/0.54} \qquad (3)$$

where R is the calibrated Maysmeter value.

Ride quality consistently dominates the highway preservation program, so the importance of determining cracking levels for Interstates and non-Interstates is secondary. There-

TABLE 5 New Condition State Numbering System

R_o	C_o	INDEX TO FIRST CRACK, I_c				
		1	2	3	4	5
1	1	1	10	19	28	37
1	2	2	11	20	29	38
1	3	3	12	21	30	39
2	1	4	13	22	31	140
2	2	5	14	23	32	41
2	3	6	15	24	33	42
3	1	7	16	25	34	43
3	2	8	17	26	35	44
3	3	9	18	27	36	45

TABLE 6 New Action Groups of Rehabilitation Actions

ACTION GROUP	ACTIONS	AVE. COST($/SY) INTERSTATE	AVE. COST($/SY) NON-INTERSTATE
1	ROUTINE MAIN.	.05	.05
2	SEAL COAT, ACFC, ACSC	1.20 - 2.6	1.25 - 2.7
3	ACFC+AR,ARAC	5.00 - 8.90	5.10 - 9.00
4	2"AC+AR,3"AC+FC	9.20 - 11.00	9.30 - 11.20
5	4.5"AC+FC & OTHER HEAVIER ACTIONS	12.00 +	12.00 +

The rehabilitation costs were derived based on 1990 data.

fore, the classification of cracking levels is grouped into the same ranges for both Interstates and non-Interstates. From the information given, it is determined that the following new classification levels are appropriate:

Function	*Factor*	*Levels*	*Unit*
Interstate	Roughness	<76, 76–104, >104	Maysmeter output (in./mi)
	Cracking	0–8, 6–15, >15	Percentage of area
Non-Interstate	Roughness	<94, 94–142, >142	Maysmeter output (in./mi)
	Cracking	0–8, 9–15, >15	Percentage of area

Development of New TPMs

Ideally, the transition probabilities are obtained by observing the performance of a large number of pavements under different rehabilitation actions over a long period. More than 10 years of pavement performance data are available now. Therefore, the proportion of roads moving from states i to j in 1 year, following kth rehabilitation action, can be determined directly from the performance data base. The following equation is applied to calculate the transition probability from state i to state j for each road category on the basis of the new pavement condition state structure:

$$p_{ij}(a_k) = \frac{m_j(a_k)}{m_i(a_k)} \qquad \text{for } i,j = 1, \ldots, 45,\ k = 1, \ldots, 6 \tag{4}$$

where

$p_{ij}(a_k)$ = transition probability from states i to j after action k is taken;
$m_j(a_k)$ = total number of miles where condition states before and after action k are i and j, respectively; and
$m_i(a_k)$ = total number of miles where condition state before action k is i.

In addition, the following probability property must be observed by adjusting the biggest value among $p_{ij}(a_k)$, for $j = 1, \ldots, 45$, and for each i and k:

$$\sum_{j=1}^{120} p_{ij}(a_k) = 1 \qquad \text{for } i = 1, \ldots, 45,\ k = 1, \ldots, 6 \tag{5}$$

The matrices have been generated for both Interstate and non-Interstate highways on the basis of the pavement performance data from 1979 to 1991. Transition probabilities predict pavement condition states on the basis of a finite-state Markov Chain process (*2,6*). Thus, the TPMs consist of one-step probabilities and can only be directly used to predict the change in condition state from one year to the next.

The Chapman-Kolmogorov equation (*7*) provides a method for computing the n-step TPM from a single-step TPM. The matrix for n-step transition probabilities can be obtained by multiplying matrices of one-step transition probabilities:

$$P^{(n)} = P \cdot P \cdots P = P^n \tag{6}$$

Therefore, the transition probabilities of pavement condition for n years can be obtained from the existing one-step transition probabilities. As a result, long-term pavement probabilistic behavior can be revealed. Figure 4 shows typical pavement probabilistic behavior curves. The upper curve shows the probability of pavements' starting in the best condition state and remaining in the best condition state over time. The lower curve shows the probability of pavements' starting in the best condition state and transitioning to the worst condition state over time.

One set of TPMs was generated from the pavement performance data base based on the new roughness and cracking levels. The new transition probabilities for remaining in the best condition under routine maintenance are shown in Table 7 for the 15 road categories. The table also presents the number of observations used to determine the probabilities. The probabilities with small sample sizes in the tables should not be used. It should be noted that the probabilities based on the new levels are smaller than those based on the original levels. This indicates that if the current pavement perfor-

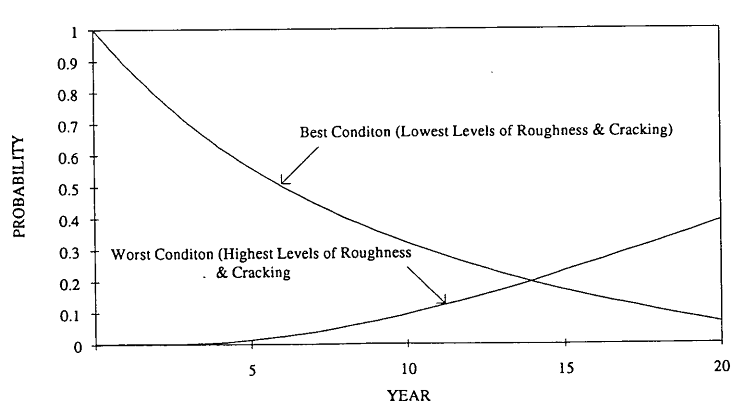

FIGURE 4 Pavement probabilistic behavior starting from best condition state under routine maintenance.

TABLE 7 Transition Probability Comparison Based on New Classification of Roughness and Cracking Levels

TRAFFIC LEVEL		LOW			MEDIUM			HIGH		
REGION		DESERT	MEDIUM	HIGH	DESERT	MEDIUM	HIGH	DESERT	MEDIUM	HIGH
REHABILITATION ACTIONS / ROAD CATEGORY										
INTERSTATE	1	N/A	N/A	N/A	0.91/1300	0.909/263	N/A	0.837/1890	0.855/1475	0.833/42
	2	N/A	N/A	N/A	0.919/478	0.905/21	N/A	0.82/423	0.889/458	0.763/118
	3	N/A	N/A	N/A	1.0/16	N/A	N/A	0.692/13	N/A	0.5/2
	4	N/A	N/A	N/A	0.862/435	1/7	N/A	0.847/785	0.83/783	0.939/98
	5	N/A	N/A	N/A	0.902/325	1/3	N/A	0.823/1172	0.893/693	0.928/499
NON-INTERSTATE	1	0.333/24	0.771/201	0.75/32	0.857/356	0.836/317	0.718/209	0.703/121	0.869/206	0.773/88
	2	0.836/311	0.794/656	0.806/366	0.79/854	0.838/1022	0.793/834	0.741/197	0.7/50	0.707/92
	3	N/A	0.962/26	N/A	0.958/118	0.88/25	0.766/47	0.881/59	N/A	N/A
	4	0.581/31	0.704/287	0.514/35	0.869/465	0.809/236	0.923/39	0.923/13	0.7/20	0.143/7
	5	N/A	N/A	N/A	0.833/102	N/A	1.0/9	N/A	N/A	N/A

NOTE:
The first number in the cell is the probability to stay in the best condition under routine maintenance for each rehabilitation action,
The second number in the cell indicates the sample size used to compute the probability,
N/A = Sample data are not available.

mance standards are used, the pavement preservation needs will be increased because of the more stringent roughness and cracking classifications.

In some instances the transitions do not exist in the pavement performance data files or the probability based on this transition is not representative of the real-world situation because of the small sample size. Therefore, to fulfill model requirements, the regression-based transition probabilities from the original NOS, or manually generated probabilities based on engineering judgment, can be used in the recommended model. This will not affect the output of the model substantially because these transitions are rarely if ever used in the optimization process.

Accessibility Rules

Condition state j is termed to be accessible from state i if $p_{ij}(a_k) > 0$ (7). No accessibility rules for routine maintenance were established in setting up the original TPMs. As a result, an illogic situation can occur when performance predictions are made by using a TPM for a pavement section in poor condition, such as State 24, high roughness and cracking. For example, Figure 5 shows that 10 percent of pavements in the worst condition will transition to the best condition state over 20 years under routine maintenance. However, in reality pavements in poor condition will not significantly improve over time under routine maintenance.

It is recommended that the data showing pavement performance improvement under routine maintenance be discarded and accessible condition states for routine maintenance be established for the development of new matrices. The pavement performance data base demonstrates that the pavement condition will not deteriorate two levels in 1 year. Accessibility rules, which prevent an improvement in pavement condition and deterioration of two levels in 1 year, were implemented by setting the probability of an illogical transition to 0. Table 8 gives the accessible transitions based on the rules, from Condition States 1 to 9. The same rules apply to Condition States 10 to 45 on the basis of roughness and cracking levels.

Figure 6 shows the effect of the accessibility rules for Interstates with medium traffic in the desert region. The accessibility rules result in a more rapid reduction in the percentage of pavements in the best condition state, and pavements

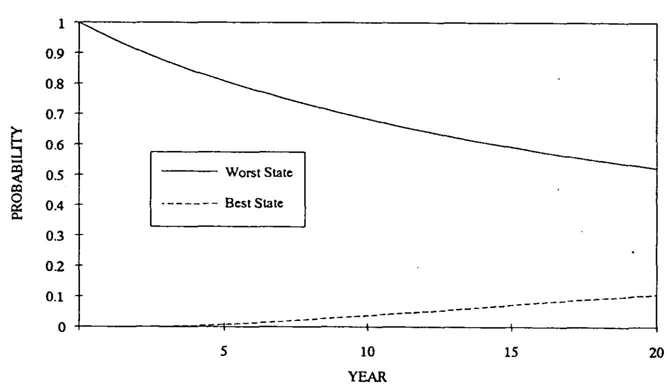

FIGURE 5 Pavement probabilistic behavior starting from worst condition state under routine maintenance on basis of original TPMs.

TABLE 8 Accessibility Table for Condition States 1 to 9 Under Routine Maintenance

FROM	TO
1	1, 2; 4, 5
2	2, 3; 5, 6
3	3; 6
4	4, 5; 7, 8
5	5, 6; 8, 9
6	6, 9
7	7, 8
8	8, 9
9	9

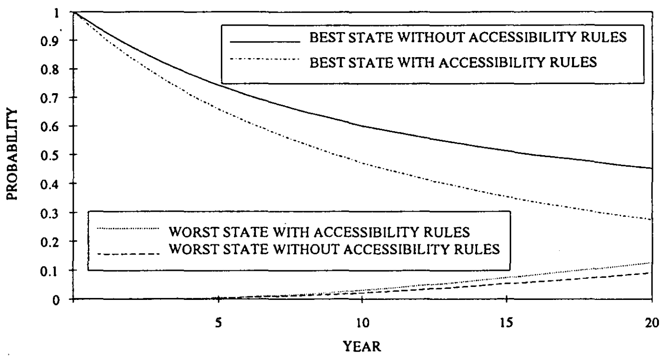

FIGURE 6 Pavement probabilistic behavior starting from best condition state under routine maintenance, with and without accessibility rules.

have a higher probability of transitioning to the worst condition state over time. It is clear that the probabilistic behavior curves presented in Figure 6 are more realistic than those in Figure 4.

CONCLUSION

The concept of the Markov chain has been used to predict pavement performance for more than a decade. The time-independent property of the Markov process is suitable for the transition equations shown by Equations 2 and 3 in the NOS linear formulation. A new study conducted by ADOT determined that the fit of Markovian predictions with actual pavement behavior was satisfactory (*9*).

A new structure of pavement condition states was set up in this paper for the optimization model used by ADOT. New TPMs were established for both the Interstates and non-Interstates on the basis of the 13-year pavement performance data base. The TPMs were modified with accessibility rules to improve the prediction of pavement performance. New analysis tools were revealed to analyze the long-term probabilistic behavior of the pavement. The revised model has been successfully implemented to an advanced 32-bit operating system environment in a high-end 486 microcomputer (*9*). ADOT is using the new NOS to generate the next 5-year Highway Preservation Program with an annual expenditure approaching $100 million. It is believed that these enhancements to the PMS will improve the reliability and accessibility of the system for ADOT.

ACKNOWLEDGMENTS

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Network Condition Analysis for Pavement Program Development: A Case Study

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The ability to describe roadway network condition properly is an essential requirement for pavement management. The methodology employed to develop and use condition measures for pavement network characterization as part of a broader effort to develop and implement a pavement management system for the New York State Thruway Authority is presented. The methodology uses a distress index as the criterion for evaluating pavement network condition. Specific intervals of the index scale are mapped to qualitative condition classes through a series of interactions with field engineers. The resulting condition classes are used in a process that relates network condition to needs and facilitates development of the annual and multiyear highway capital improvement and maintenance programs. A distinction between pavement and highway needs is necessary to accommodate Thruway-specific program development and budgeting practices. Two types of network-level analysis are implemented. In the first, the network is characterized using uniform sections based solely on pavement condition, and analysis is limited to short-term pavement needs. In the second, the network is characterized using planning sections, and analysis includes the broader long-term highway needs. It is concluded that pavement condition characterization provides an objective basis for program development but that it must be supplemented by additional highway information. This is because treatment needs for specific projects cannot always be determined solely on the basis of characterized pavement condition.

Once a novel concept, pavement management systems (PMSs) are now an established tool for the preservation and improvement of existing pavements (*1*,*2*). The New York State Thruway Authority (NYSTA) and Rensselaer Polytechnic Institute (RPI) have been cooperating since 1988 to develop a PMS for the authority. The PMS is based on experience with local conditions, materials, and pavement performance. It uses modern decision-making procedures and state-of-the-art technology to record, store, and analyze information. Technical details are documented elsewhere (*3–8*).

This paper provides a case study of the process through which a specific component of the developed PMS is being integrated into the operations of the NYSTA. The study outlines the use of a distress index as a criterion for network condition evaluation and pavement program development. The distress index is also used in other development methodologies—such as economic analysis, prioritization, and optimization—that constitute distinct components of the authority's PMS.

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Integration of the distress index with project programming activities was advanced through a series of interactions with the field engineers and headquarters personnel responsible for managing the highway maintenance and rehabilitation program. Three specific objectives were accomplished during this study: (a) evaluation of the condition of NYSTA pavements using a PMS methodology, (b) initiation of a process that can convert network condition into a scope of work and establish an annual and multiyear highway program, and (c) identification of needed enhancements to developed PMS methodologies, based on feedback from experiences to date.

OVERVIEW OF NYSTA PMS

Operational Perspective

NYSTA was established in 1950 to construct, maintain, and operate a limited-access toll road spanning the state of New York. The Thruway currently consists of 1030 km (640 mi) of Interstate-type highways, administered as four divisions. The pavement was originally constructed of reinforced portland cement concrete (PCC). Typical slabs are 30.5 m (100 ft) long and 23 cm (9 in.) thick and have expansion joints with load transfer devices. The original slabs were constructed on 30.5 cm (12 in.) of granulated subbase course, with no provision for subsurface drainage.

As the original pavements deteriorated over the years, approximately 90 percent of the entire network was overlaid with asphalt concrete. Underdrain has been installed in conjunction with many rehabilitation projects. Shoulders were originally constructed of chloride-treated granulated material or sod. All have since received at least a thin asphalt overlay, and some have been fully reconstructed with asphalt concrete.

In 1986, with 10 years remaining under the initial organizing legislation and with increasing problems brought about by a chronic shortage of funds, a $1.7 billion infrastructure improvement program was developed for the period 1988 to 1996. The highway component ($500 million) of this long-term program was based on generalized pavement surface condition ratings obtained by a windshield survey. The PMS research and development effort was initiated in 1988 in response to the need to improve the information basis and decision methodologies used for highway program development and monitoring.

Analytical Methodologies

A methodology has been developed for visually evaluating surface distress on asphalt overlaid and PCC pavements in an objective and reliable manner (*3*). Individual distress magnitudes are assessed using linguistic scales that consider distress type, severity, and extent along nominal lengths of pavement. The survey that collects this information has been applied annually to the driving lanes and shoulders of all Thruway pavements since 1989. A 3-week rater training and testing program has been held annually to ensure the quality of the collected data.

A set of project- and network-level methodologies has been developed as part of NYSTA's PMS (*6–8*). It includes condition analysis, project definition, categorization, treatment selection, life-cycle cost analysis, project ranking, optimal project scheduling, and optimal program development. The methodologies were initially developed using a series of spreadsheets and stand-alone computer programs. They are being enhanced and integrated under a prototype windows-based program manager, and linkages are being developed to a centralized relational data base.

CONDITION ASSESSMENT

The distress data collection activity generates eight distress ratings for each 160.9-m (0.10-mi) nominal segment of road surveyed (*3*). In essence, these data represent localized assessments of pavement condition at discrete points in time and space. Detailed information of this type is essential for refined project-level analysis, but pavement condition must also be expressed in a more aggregate manner to support network-level activities. This synthesis is accomplished through a methodology that combines distress data from individual segments into indexes that represent the aggregate condition of each pavement project. Thus, distresses reflecting the condition of a specific pavement component, such as slab, joint, shoulder, or the entire pavement surface, are combined into indexes descriptive of the specific component (*5*).

The indexes are produced by a calculation method that accounts for the relative significance of each individual distress through the use of appropriate weighting factors. Index values are scaled proportionately to the maximum possible value and are reported on a 100-point scale. The scaled value denotes the calculated cumulative distress condition relative to the maximum value that a given distress index may receive. Thus, distress indexes can range from 0 to 100, with 100 representing a condition of no surface distress. Details of the calculation method are provided by Grivas et al. (*5*).

Consideration of the decision support potential offered by each of the developed indexes led to the designation of the lane distress index (LDI) as the condition measure to be used for pavement network characterization. It is anticipated that LDI will also be used to monitor surface condition over time. This is illustrated in Table 1, which presents the change of the pavement network condition from 1990 to 1991, exclusive of those sections undergoing rehabilitation or reconstruction during that period. The aggregate change in condition is summarized for all overlaid (OVL) and concrete (PCC) pavement meeting previous criteria. The reported values represent net condition changes, including effects of both deterioration and maintenance work. It can be seen that on average, overlaid pavements declined by 0.4 points, and concrete pavement declined by 6.5 points over the 1-year period. As more data become available, a capability will be developed to predict trends on the basis of past performance.

TABLE 1 Aggregate Change in Thruway Condition (1990–1991)

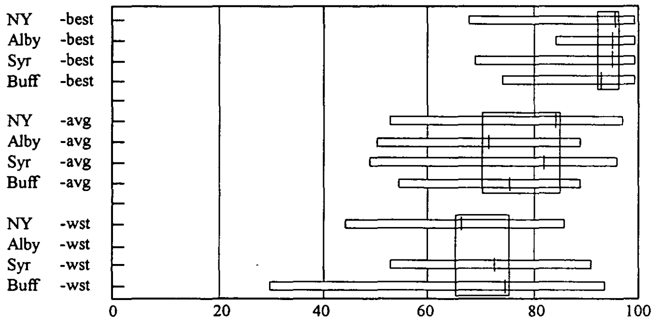
	Pavement Type	
Summary	OVL	PCC
Total Number of Rated Segments in Both Years	8,546	693
Number of Rated Segments with an LDI Change of <5 Points	2,758 (32.3%)	84 (12.1%)
Number with LDI Increase	1,989 (23.3%)	155 (22.4%)
Number with LDI Decrease	6,557 (76.7%)	538 (77.6%)
Net (increase - decrease)	- 4,568 (53.4%)	- 383 (55.2%)
Approximate Mean Increase	12.2	16.7
Approximate Mean Decrease	- 12.6	- 23.2

NETWORK CONDITION EVALUATION

NYSTA field engineers have accepted the LDI for a wide range of uses, including comparison of projects on the basis of exhibited surface distresses and characterization of system condition. Soon after acceptance of the LDI, the need was identified to use the quantitative values of LDI as the criterion for evaluating network condition—that is, to establish what constitutes excellent, good, fair, and poor pavement. This was achieved by a series of interactions with field engineers and a sensitivity analysis of the boundary values for the resulting condition classes.

First Iteration

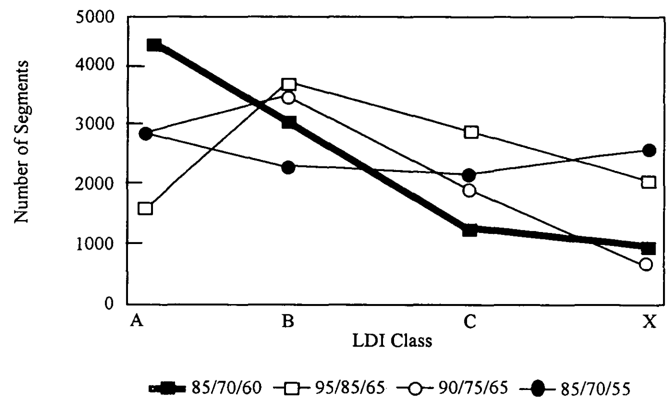
In the first iteration of this study, engineers from each of the four divisions were asked to subjectively identify sections of pavement that exhibited the best, average, and worst condition. The responses characterized 22 sections, totaling 234.6 km (145.8 mi) of pavement. The LDI for each 160.9-m (0.10-mi) interval of these sections was calculated, and the mean and range of the LDI for each category of pavement in each division were compared. The results for overlaid pavement are summarized in Figure 1, where the bars indicate the range and the tick marks on the bars indicate the mean value of LDI determined for each category. This study suggested that



Engineers' Assessment	Division	LDI Values Min	Mean	Max
Best	New York	68.2	95.7	99.4
	Albany	84.3	95.3	99.4
	Syracuse	69.6	95.2	99.4
	Buffalo	75.3	93.5	99.4
Average	New York	53.5	83.7	97.3
	Albany	50.7	71.0	88.9
	Syracuse	50.0	81.7	96.0
	Buffalo	55.0	75.0	88.9
Worst	New York	44.4	64.8	84.3
	Albany	*	*	*
	Syracuse	53.6	71.8	89.3
	Buffalo	30.6	74.6	92.8

* not reported

FIGURE 1 Ranges of LDI for overlaid pavement sections identified as representing best, average, and worst condition in each division.



LDI Class	No. of Segments	Rel. Freq.	Rel. Cum. Freq.
A > 90 - 100	2859	0.29	0.29
B > 80 - 90	2344	0.24	0.53
C > 70 - 80	2168	0.22	0.75
X ≤ 70	2452	0.25	1.00
A > 95 - 100	1629	0.17	0.17
B > 80 - 95	3574	0.36	0.53
C > 65 - 80	2685	0.27	0.80
X ≤ 65	1935	0.20	1.00
A > 90 - 100	2859	0.29	0.29
B > 75 - 90	3465	0.35	0.64
C > 65 - 75	1564	0.16	0.80
X ≤ 65	1935	0.20	1.00
A > 85 - 100	4488	0.46	0.46
B > 70 - 85	2886	0.29	0.75
C > 60 - 70	1339	0.14	0.89
X ≤ 60	1113	0.11	1.00
A > 85 - 100	4485	0.46	0.46
B > 70 - 85	2886	0.29	0.75
C > 55 - 70	1642	0.17	0.92
X ≤ 55	810	0.08	1.00

FIGURE 2 Sensitivity of overlaid pavement network characterization to selection of LDI intervals.

LDI was a meaningful basis for developing qualitative condition classes. It also provided an early indication of the need to resolve differences between individuals for such subjective assessments.

Second Iteration

From the results of the first iteration, four preliminary condition classes, referred to as A (excellent), B (good), C (fair), and X (poor), were proposed. Field engineers were asked to assign each section under their jurisdiction to one of the qualitative classes. The mean values and ranges of LDI of sections in each class were compared as before. The obtained results confirmed the findings of the first iteration, namely, that each individual showed a consistent trend of assigning sections with lower LDI to poorer condition classes, and that subjective assessments by different individuals were not always consistent.

Sensitivity Analysis

The demonstrated lack of consistency between individuals emphasized the need to relate qualitative characterizations to a relatively objective measure such as LDI. This was accomplished by defining intervals of the 100-point LDI scale to correspond to each of the four condition classes. Figure 2 illustrates the sensitivity of the characterization of system condition to the boundaries of the LDI intervals for each class. For example, it can be seen from Figure 2 that if Class A (excellent) is defined by $95 < LDI \leq 100$, then 17 percent of the overlaid pavement in the system is characterized as excellent. However, if Class A is defined by $85 < LDI \leq 100$, then 46 percent of the overlaid pavement is considered excellent. The heavy line corresponds to the limiting values between LDI classes that were eventually selected.

Results

The resulting network characterization based on the selected LDI intervals is presented in Tables 2 and 3. In Table 2 it can be seen that the percentages of pavement considered excellent and good have increased for overlaid pavement, and decreased for concrete pavement, between 1990 and 1991. Table 3 provides a summary of 1991 pavement condition for the four Thruway divisions. Constraints imposed by variations in geographic and demographic characterization across the state, in combination with historically different approaches to pavement maintenance, result in distinctly different characterizations between divisions. It can be seen, for example, that the Syracuse division is composed entirely of asphalt overlaid pavement and has the highest percentage of excellent pavement and the lowest percentage of poor pavement among all divisions. Interpretation of such observations must of course be tempered by consideration of past maintenance practices and other factors such as climate, soil, and traffic that affect pavement performance across the state.

PRELIMINARY PROGRAM DEVELOPMENT

The developed PMS uses a staged approach to program development, one in which defined projects are screened to identify feasible scopes of work before proceeding with an economic analysis of alternatives, scheduling, and optimization (*6,8*). The relationship between LDI, which is used for network characterization, and a project scope of work, which is determined during preliminary program development, provides an important link between project- and system-level methodologies.

TABLE 2 Characterization of Thruway Network Condition

Condition Class (LDI)	Overlaid 1990	Overlaid 1991	Concrete 1990	Concrete 1991
A ("Excellent") LDI > 85 - 100	42.4% 630.0 km	45.7% 722.3 km	1.1% 1.9 km	1.2% 3.4 km
B ("Very Good") LDI > 70 - 85	25.6% 380.4 km	29.4% 464.4 km	5.2% 9.3 km	2.6% 7.4 km
C ("Good") LDI > 60 - 70	18.4% 273.4 km	13.6% 215.5 km	4.8% 8.7 km	3.7% 10.5 km
X ("Poor") LDI 0 - 60	13.5% 200.5 km	11.3% 179.1 km	88.9% 159.8 km	92.6% 264.7 km
Length Surveyed (km)	1484.3	1581.3	179.7	286.0

1 km=0.6 mi

TABLE 3 Comparison of Division Network Characterizations

Division	Pvt Type	LDI Condition Class (%) A	B	C	X
New York (492.3 km)	Overlaid	30.1	10.5	9.1	4.3
	Concrete	0.3	1.0	2.0	42.7
Albany (440.5 km)	Overlaid	40.5	20.3	14.1	14.4
	Concrete	0.1	0.1	0.1	10.5
Syracuse (450.6 km)	Overlaid	50.6	30.3	13.0	6.1
	Concrete	0.0	0.0	0.0	0.0
Buffalo (483.9 km)	Overlaid	34.7	38.6	10.4	13.8
	Concrete	0.4	0.4	0.1	1.7

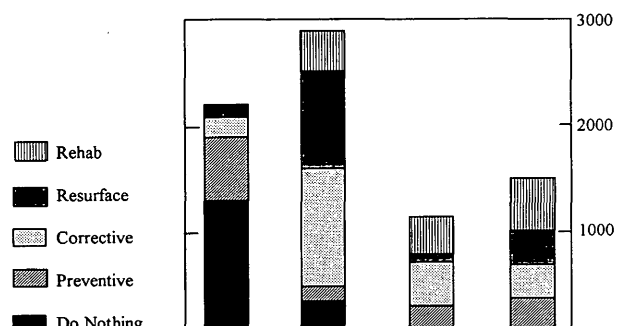
1 km = 0.6 mi

Uniform Sections

The developed PMS defines uniform sections on the basis of LDI values and construction and maintenance history. When defined in this manner, each payment section has needs distinct from those surrounding it. This facilitates economic programming, as treatments can be tailored to each project, thus minimizing occurrences of inappropriate treatment. Figure 3 shows, for the case of three divisions, how the scope of work applied to each uniform section in 1991 compares with the condition class characterization. It can be seen that the more involved scopes of work tend to be applied to sections in worse condition. It is also observed that about a fifth of the pavements in poor condition are receiving preventive maintenance. This apparent inconsistency is due to the attempt to maintain pavement sections in serviceable condition until programmed rehabilitation or reconstruction is performed. Similarly, sections in excellent condition may be resurfaced if, for example, roughness is outside acceptable limits. Clearly, in some cases the condition characterization must be supplemented with other engineering parameters to provide a comprehensive evaluation.

Planning Sections

Early in the process of integrating the PMS with existing project programming procedures, it became clear that a distinction had to be made between condition-generated "pavement" projects, which are the focus of the developed PMS, and "highway" projects, which include items such as guiderails, lighting, interchanges, ramps, toll plazas, signs, and slopes, in addition to pavement lane and shoulder improvements. The distinction between pavements and highways was introduced because (a) the condition and requirements of nonpavement items can significantly affect the scope and cost of work re-



	A	B	C	X
Rehab	0	374	260	505
Resurface	104	854	64	286
Corrective	196	991	462	327
Preventive	652	202	298	312
Do Nothing	1,279	432	42	38

FIGURE 3 Comparison of implemented scope of work with condition class for 1991 uniform sections (New York, Albany, and Buffalo divisions).

quired at a given location, and (b) for multiyear planning, it is prudent to coordinate work on adjacent highway sections, bridges, and bridge approaches to achieve economies of scale and scope and minimize inconvenience to users.

Uniform sections are accepted as useful for planning and assessing annual maintenance activities. However, because they are defined without consideration of highway characteristics, they are considered to be of limited use for long-term capital programming. Therefore, the concept of planning sections was introduced to improve the process of program development and monitoring. Planning sections are used primarily for long-term capital programming and may incorporate several uniform sections in addition to bridges, ramps, toll plazas, slopes, and so on. Figure 4 illustrates the difference between uniform and planning sections. Whereas the boundaries of uniform sections may change from year to year, the boundaries of planning sections are structured to be fixed for long periods to facilitate coordination between related projects in different years.

Table 4 provides an example of the annual (1992) and multiyear (1993–2000) programs for one division (Buffalo) that corresponds to the condition classes determined on the basis of the 1991 condition survey data. The number of miles and the percentage of the division network that are programmed for each treatment type are also provided.

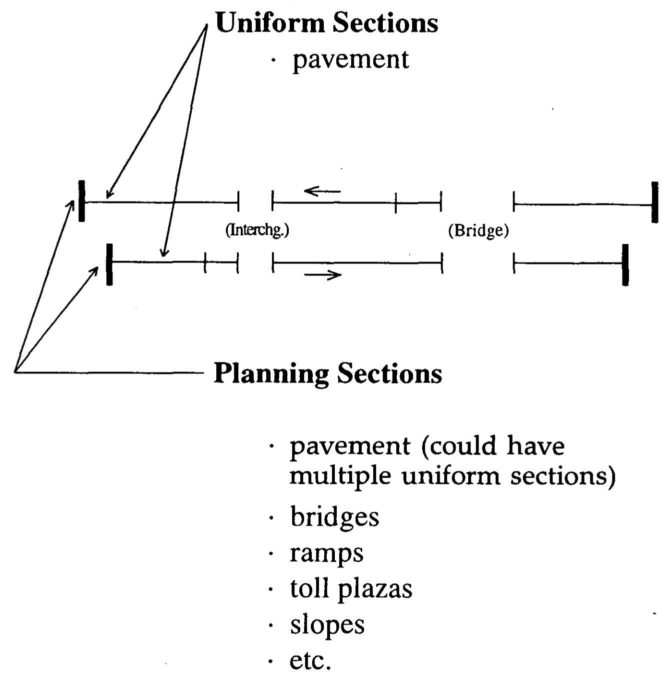


FIGURE 4 Relationship between uniform sections and planning sections.

TABLE 4 Annual and Multiyear Capital Program for a Division

	1992			Future M&R		
Condition Class (1991)	Scheduled Work	Length (in km)[a]	Percent of Class	Next Major Work	Length (in km)	Yr.
	Do nothing	34.0	28%	Mill and pave	8.9	'96
	Preventive mtce.	48.3	40%	Mill and pave	25.3	'98
A	Prevent. & correct. mtce.	4.2	4%	Paving (scope unknown)	4.2	'97
(119.4 km)	Pave (selected portions)	16.3	14%		38.3	
	Mill and pave (DL only)	15.1	13%			
	Mill and pave	1.6	1%			
	Do nothing	21.6	8%	Mill and pave (DL only)	10.5	'93
	Preventive mtce.	5.8	2%	Mill and pave (DL only)	34.3	'94
B	Prevent. & correct. mtce.	88.7	32%	Mill and pave (DL only)	7.4	'95
(273.4 km)	Corrective mtce.	55.2	20%	Mill and pave (recycle)	49.7	'93
	Mill and pave (DL only)	72.7	27%	Mill and pave	5.8	'96
	Mill and pave	29.5	11%	Mill and pave	1.9	'98
				Mill and pave	20.0	2000
				Paving (scope unknown)	4.7	'97
				Rehabilitation	20.3	2000
				Rehab/reconstruct	8.5	'94
				Rehab/recon.-add lane	3.2	'94
					166.2	
C	Preventive mtce.	6.9	15%	Mill, rubblize, and pave	6.9	'93
(46.5 km)	Prevent. & correct. mtce.	29.3	63%	Rehabilitation	27.7	'93
	Mill and pave (DL only)	10.3	22%		34.6	
X	Preventive mtce.	42.8	64%	Rehabilitation	42.8	'92
(67.3 km)	Prevent. & correct. mtce.	5.5	8%	Reconstruction	10.9	'93/'94
	Interim pave	19.0	28%	Reconstruction	13.5	'95/'96
					67.3	

1 km = 0.6 mi

FUTURE DEVELOPMENTS

Enhancements to the developed PMS are being pursued jointly by the NYSTA and RPI. Activities focus on

- Expanding pavement condition assessment procedures to provide additional quantitative measures such as roughness, rutting, and transverse profile;
- Automating pavement image analysis;
- Refining the decision methodologies to reflect improvements suggested from operational use of methods and products; and
- Integrating the component stand-alone application programs and the centralized relational data base into a networked computer system.

SUMMARY AND CONCLUSIONS

This paper presented the process through which a distress index was integrated into the NYSTA's project programming operations. The index has been examined and used to provide classification structures for network characterization and to support preliminary program development. Field assessments of pavement quality were associated with distress index values to provide a basis for network characterization. These characterizations were compared with the defined scope of work for programmed projects using 1991 data.

On the basis of the achievements and findings presented in this study, the following conclusions may be drawn:

- The participation of PMS users in the development of criteria for network characterization is essential for acceptance and integration into the project programming processes.
- Though network characterization provides a basis for objective pavement programming, it alone does not represent a comprehensive view of roadway condition. The needs of nonpavement components of the roadway (ramps, bridges, rock slopes, etc.) must also be accommodated during program development.
- The relationship between the characterization of pavement condition and the treatments applied to specific projects can be a very complex one. Treatments cannot always be determined solely on the basis of characterized pavement condition.

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Views and opinions expressed herein do not necessarily reflect those of the NYSTA.

Integrated Pavement and Bridge Management Optimization

WILLIAM V. HARPER AND KAMRAN MAJIDZADEH

An integrated pavement and bridge management system that allows cost minimization or benefit maximization is presented. It integrates the pavement and bridge systems so that management may allocate scarce resources optimally across the combined systems. Fuzzy set theory is used in these optimizations to better address the desirability or undesirability of the condition states used to categorize the pavement and bridge segments modeled. Both steady-state and multiyear models are presented.

A highway maintenance management system (HMMS) has been developed. This system integrates a pavement management system (PMS), a bridges and structures management system (B&SMS), and a nonpavement management system. A relational data base (Oracle) is used to perform the needed data storage and retrieval functions. This paper focuses on the integration of the PMS and the B&SMS. The full integration with the nonpavement management system may be found elsewhere (*1–3*).

The HMMS is a flexible modular system that can be easily adapted to meet various needs. The particular adaptation presented here is for a given client, but it can be modified easily for other applications. The integrated system allows the optimal allocation of the budget across the various subsystems (e.g., across the PMS and B&SMS in this paper). Thus, it is not necessary to make an arbitrary division of the budget into the subsystem; instead, an optimal division will be determined by the HMMS.

The PMS and B&SMS steady-state and multiyear results may be optimized using either cost minimization or benefit maximization. The PMS is divided into nine strata based on three levels of climate and three functional classes. The condition state variables for the PMS are rutting (three levels), cracking (three levels), delta cracking—1 year change in cracking (three levels), roughness (three levels), and index to first crack (four levels). These variables result in 324 condition states. There are 17 possible maintenance actions with a feasible subset for each condition state. In the cost-minimization models (*2,3*), management specifies desired performance levels and the optimization finds the lowest-cost plan that will meet the performance goals. In the benefit maximization models, benefits based on fuzzy set memberships and importance weights are maximized subject to budgetary controls.

The B&SMS is divided into 43 strata: 36 for bridges, 6 for culverts, and 1 for tunnels. The 36 bridge strata result from 3 climates, 6 major bridge types, and 2 functional classes. Culverts are not subdivided by type in the optimizations and thus have only three climates and two functional classes, resulting in six strata. The condition state variables depend on the stratum. For example, steel bridges have deck (four levels), superstructure (four levels), substructure (four levels), superstructure age (three levels), and substructure age (three levels), for a total of 576 condition states. For this bridge type there are 40 maintenance scopes (e.g., deck repair) with a selected subset feasible for each condition state. Harper et al. describe this in more detail elsewhere (*4,5*).

The PMS and B&SMS are modular systems with prediction, cost, optimization, packaging, and comparator modules. The prediction modules determine the transition probabilities that estimate the degradation rates for the PMS or B&SMS segments. In the PMS a segment is a 1-km single lane of road. In the B&SMS, the definition of a segment depends on the stratum. For steel bridges it is a superstructure span with a substructure pier or abutment. The survey results are converted to condition states as described by Harper et al. (*4*) and are used in Bayesian updating algorithms to adapt the transition probabilities to the actual environmental conditions encountered. The cost module determines the action/scope optimization costs. This paper focuses on the optimization module. The packaging module takes the selected optimal stratum solutions and makes assignments to the actual segments. The optimization selections are made more specific, and detailed cost estimates are created in different formats to satisfy management needs. The comparator module provides feedback on the system performance and implementation.

FUZZY SET THEORY ADAPTATIONS

In classical set theory, either each "object" (e.g., condition state) is a member of a set or it is not. As fractals have stretched the boundaries of many disciplines to consider noninteger dimensions to supplement the integer dimensions found in classical science, fuzzy set theory expands the concept of the membership of an object in a set to be any value on the continuum [0,1.0] with larger values representing a higher or stronger degree of membership in the set. Classical sets are special cases of fuzzy sets in which the membership is restricted to values of 0 (object is not a member of the set) and 1 (object is a member of the set).

Early versions of the cost minimization models (*5*) categorized each condition state into one of the following mutually exclusive categories:

- Desirable,
- Undesirable, or
- Neither desirable nor undesirable.

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Previously the B&SMS categorized as undesirable any condition state that had at least one element (e.g., for bridges, deck, superstructure, or substructure) in critical condition (good, fair, poor, and critical are the possible levels). Though one would surely agree that a bridge segment with deck, superstructure, and substructure all at the critical level is an undesirable condition state, it is not so clear-cut with another segment when the deck is in critical condition and the other two elements are in good condition. It is apparent that the former segment is more undesirable than the latter, which has only one critical element. The previous performance constraints (*5–7*) do not directly account for such distinctions.

Fuzziness is a natural result of the lack of well-defined boundaries. An example would be the set of "rich" people. The transition between nonmembership and membership for this set is gradual and lacks an obvious boundary. Clearly some individuals are rich and would have a membership in this set equal to 1, but for many others it is not so obvious. Zadeh in 1965 published the initial work in this area (*8*). He set the groundwork for a fertile field that is seeing many applications including consumer products.

Confusion about fuzzy set theory often occurs because fuzzy sets are assumed to be related to probabilistic random variables or some form of uncertainty. Instead, fuzziness is a result of the absence of sharply defined criteria of class membership. The fuzziness ensues from the vagueness or imprecision that results from the inability to classify adequately objects using conventional sets. Thus fuzzy sets are essential to address the true situation properly. Zadeh has argued the following: "Indeed, fuzziness is more than a facet of reality; it is one of its most pervasive characteristics—a characteristic rooted in the bounded capacity of the human mind to process and store information" (*9*).

Categorizing a condition state into one of the three categories was a difficult task. These are not black-and-white situations that are readily apparent. Each condition state within one of these three groupings was treated as having equal weight within that category—that is, each condition state had a membership of 1 in the set it was placed and a membership of 0 in the other two sets.

In the optimization models presented here the condition states need not be treated as a member of only one set. Instead, each condition state has a membership in both the desirable and undesirable fuzzy sets. This membership—$\Phi_{id}(s)$ (desirable), $\Phi_{iu}(s)$ (undesirable)—may take any value on the range [0,1.0], that is, $\Phi_{id}(s)$, $\Phi_{iu}(s) \in [0,1.0]$. An extremely desirable B&SMS condition state with all elements good has $\Phi_{id}(s) = 1.0$ and $\Phi_{iu}(s) = 0$. Similarly, an extremely undesirable B&SMS condition state with all elements critical has $\Phi_{id}(s) = 0.0$ and $\Phi_{iu}(s) = 1.0$. A similar situation holds for the PMS. Many condition states will have nonzero memberships in both the desirable and undesirable fuzzy sets. Additional details on the fuzzy set memberships may be found elsewhere (*1*).

STEADY-STATE BENEFIT MAXIMIZATION

The steady-state models in the PMS and B&SMS are solved in order to set 5-year goals for the multiyear planning models. The model is given in the following. The summations over i cover the entire set of condition states for each stratum. Each s representing a stratum is unique. The PMS/B&SMS steady-state model uses the following variables:

Cost minimization and benefit maximization:

$w_{ia}(s)$ = proportion of units in stratum s that are in condition state i and receive action/scope a. These are the decision variables.

$w^*_{ia}(s)$ = optimal output $w_{ia}(s)$.

$P_{iaj}(s)$ = probability of a segment transitioning in 1 year from condition state i to condition state j when action/scope a is applied in stratum s.

$C_{ia}(s)$ = cost of action/scope a for a segment in stratum s in state i.

$C^*(s)$ = optimal steady-state average segment cost for stratum s:

$$C^*(s) = \sum_{i \in I(s)} \sum_{a \in M_i(s)} w^*_{ia}(s) C_{ia}(s)$$

$N(s)$ = number of segments in stratum s.

$I(s)$ = index set of conditions states i for stratum s.

$M_i(s)$ = set of feasible actions/scopes for condition state i in stratum s.

$p_k(s)$ = performance goal upper- or lower-bound for generalized performance constraint k of stratum s.

$\Phi_{ik}(s)$ = generalized performance constraint parameter for condition state i; may be either fuzzy set memberships, $\Phi_{iu}(s)$ or $\Phi_{id}(s)$, or set to other values depending on the form of the generalized constraint k for stratum s.

$\$_k(s)$ = stratum budget limits; they may be used to bound expenditures (upper or lower bound) in stratum s where $\$_k(s)$ is a specified budget limit.

S_{P} = index set of PMS strata.

S_{BS} = index set of B&SMS strata.

$S_{\mathrm{P+BS}}$ = index set of PMS and B&SMS strata.

B_{BS} = total annual budget for bridges and structures.

B_{P} = total annual budget for pavement.

$B_{\mathrm{P+BS}}$ = total annual budget for pavement, bridges, and structures.

Benefit maximization objective function:

α = Lagrange multiplier used to move budget constraint into objective function; this allows separation of the budget integrated optimization into individual stratum problems. The units of α for benefit maximization are (units of benefit)/(units of cost). It is unitless for multiyear cost minimization. This is an output of the optimization process.

$$\alpha \in [0.0,\infty)$$

$N_n(s)$ = normalized number of segments in stratum s; this is the proportion of segments in stratum s relative to the entire subsystem (either PMS or B&SMS).

$W_d(s)$ = importance weight for being in desirable levels in stratum s.

$W_u(s)$ = importance weight for not being in undesirable levels in stratum s.

$\Phi_{id}(s)$ = desirable fuzzy set membership for condition state i in stratum s.

$\Phi_{iu}(s)$ = undesirable fuzzy set membership for condition state i in stratum s.

$\pi_i(s)$ = net worth of condition state i in stratum s that combines the individual desirable/not in undesirable importance weights, $W_d(s)$ and $W_u(s)$, with $\Phi_{id}(s)$ and $\Phi_{iu}(s)$ as follows:

$$\pi_i(s) = W_d(s)\,\Phi_{id}(s) - W_u(s)\,\Phi_{iu}(s)$$

ϕ_{sys} = relative weight of subsystem:

$$\phi_{sys} = \begin{cases} \phi_{B\&SMS} & \text{for a B\&SMS stratum} \\ \phi_{PMS} & \text{for a PMS stratum} \\ \phi_{NPMS} & \text{for an NPMS stratum} \end{cases}$$

The PMS and B&SMS steady-state models are

For the benefit maximization objective function,

maximize

$$N_n(s)\left[\phi_{sys} \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)\pi_i(s)\right] - \alpha\, N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)C_{ia}(s) \quad (1)$$

For the cost minimization objective function,

minimize

$$N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)C_{ia}(s) \quad (2)$$

subject to (same constraints for benefit maximization or cost minimization)

$$w_{ia}(s) \geq 0 \qquad \text{for all } i, a, s \quad (3)$$

$$\sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s) = 1 \qquad \text{for all } s \quad (4)$$

$$\sum_{a\in M_j(s)} w_{ja}(s) - \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)P_{iaj}(s) = 0 \qquad \text{for all } j, s \quad (5)$$

$$\sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)\Phi_{ik}(s)\ (\geq \text{ or } \leq)\ p_k(s) \qquad k = 1, \ldots, K(s) \text{ for all } s \quad (6)$$

$$N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s)C_{ia}(s)\ (\geq \text{ or } \leq)\ \$_k(s) \qquad k = 1, \ldots, K_{BL}(s) \quad (7)$$

The benefit maximization objective function (Equation 1) maximizes a weighted sum reflecting benefits. The coefficient of $w_{ia}(s)$ is the product of several factors: normalized number of segments $N_n(s)$, $\Phi_{id}(s)$ and $\Phi_{iu}(s)$ that measure the degree of desirable or undesirable membership, importance weights $W_d(s)$ and $W_u(s)$, and the relative subsystem weight ϕ_{sys}. The weights $W_d(s)$ and $W_u(s)$ indicate the relative importance of the difference between proportions of strata in desirable conditions and the proportion not in undesirable conditions, the difference between functional classes, climatic differences, and bridge type (for bridge strata). For steady-state budget integration Equation 1 is summed over all strata, as shown for B&SMS in the following, to incorporate the budget constraint.

$$\sum_{s\in S_{BS}} N_n(s)\left[\phi_{sys} \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)\pi_i(s)\right] - \alpha \sum_{s\in S_{BS}} N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)C_{ia}(s) \quad (8)$$

The second (Lagrange) term of the benefit maximization objective function enforces Constraint 9, thus ensuring that the budget (B_{P+BS}, B_P, or B_{BS}) is met. Lagrange relaxation is used since it permits the separation of the problem into an equivalent set of individual stratum models without having to actually specify the budget constraint. Each value of α corresponds to a given total budget level. This is a monotonic decreasing function that decrements at discrete levels of α.

$$\sum_{s\in S_{BS}} N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}(s)C_{ia}(s) \leq (B_{BS}, B_P, \text{ or } B_{P+BS}) \quad (9)$$

(S_{BS} is replaced by S_P or S_{P+BS} as appropriate.)

The cost minimization objective function (Equation 2) minimizes the cost in stratum s. Constraints 3 and 4 ensure that solutions satisfy probability axioms. The variables $w_{ia}(s)$ are elements of a discrete joint probability distribution. Constraint 3 ensures the nonnegativity (implicit in LP) of each individual element in this joint probability distribution, and Constraint 4 forces the sum over the feasible sample space (in a statistical sense) to equal 1. Constraint 5 includes the steady-state equations for a Markov process (force the proportion of the network in condition state i to remain fixed, i.e., at steady state).

Constraint 6 includes generalized performance constraints for each stratum (optional in benefit maximization but almost always necessary in cost minimization). These performance goal constraints allow considerable flexibility and bestow significant management control. Management may make detailed specific goals of relevance to them using these generalized performance constraints. Potential examples of the generalized performance constraints include constraints using fuzzy set goals or the older designations of desirable/undesirable goals. Another option is to set element goals, for example, percentage of decks wanted in at least fair condition (or similar goals on distresses in PMS).

Equation 7 allows the optional inclusion of an upper or lower budget bound for an individual stratum. This is not normally used—usually the Lagrange term is used instead to control the entire network budget.

IMPORTANCE WEIGHTS

This section briefly covers the importance weights that are fully described elsewhere (*1–3*). These are multiplicative weights that are used to derive the $w_d(s)$ and $w_u(s)$ used in the PMS and B&SMS. They are developed within each subsystem (PMS, B&SMS), and then the weights across the subsystems are incorporated as well as weighing desirable versus undesirable.

Within the B&SMS the strata factors depend on whether the stratum is a bridge, culvert, or tunnel stratum. For bridges the stratum factors are bridge type, climate, and functional class. For culverts, only climate and functional class are necessary. Tunnels have only one stratum and thus do not require any further breakdown.

Selected internal B&SMS ranking weights are given below. The ranking weights used by Highway Maintenance Associates (*2,3*) have to be inverted to show importance.

- Functional class
  - –Primary [2]
  - –Secondary [6]
- Climate
  - –Desert [1]
  - –Mountain [1]
  - –Coastal [.75]
- Bridge type
  - –Concrete slab, simple [6]
  - –Concrete slab, continuous [6]
  - –Concrete girders (or R.C. Box) [6]
  - –Steel composite [8]
  - –Prestressed girder [4]
  - –Prestressed box [4]
- Structure type
  - –Bridge [3]
  - –Tunnel [3]
  - –Culvert [8]

Following is an example calculation of how the preceding ranking weights are converted to importance weights used in the optimization. This example deals only with the climatic aspect.

$$\text{desert} = \text{coastal} = 1/[1 + 1 + 1/0.75] = 0.3$$

$$\text{mountain} = (1/0.75)/[1 + 1 + 1/0.75] = 0.4$$

The PMS strata are based on climate (three levels) and functional class (three levels). The same climate weights used for the B&SMS are also used for the PMS. For functional class, the ranking weights established were primary [2], secondary [4], and feeder [8].

Tables 1 and 2 contain selected intermediate importance weights that result from the previous material. They are incorporated with additional weights (e.g., PMS versus B&SMS, and desirable versus undesirable) when the optimizations are run. The results are used in both the steady-state and multiyear optimizations. The following list gives the six bridge types referred to in Table 1:

1. Reinforced concrete slab bridge, simple span;
2. Reinforced concrete bridges, continuous span;
3. Prestressed girder (I, T, etc.) bridges (or reinforced concrete box girder bridges);
4. Steel composite bridges;
5. Reinforced concrete T-girder bridges; and
6. Prestressed box girder bridges.

TABLE 1 Intermediate Importance Weights, Bridges

Func. Class	Climate	Bridge Type	Weight
Primary	Desert	1	1.31
Primary	Desert	4	.99
Primary	Desert	5	1.97
Primary	Mountain	1	1.75
Primary	Mountain	4	1.31
Primary	Mountain	5	2.63
Secondary	Desert	1	.44
Secondary	Desert	4	.33
Secondary	Desert	5	.66

TABLE 2 Intermediate Importance Weights, PMS

Func. Class	Climate	Weight
Primary	Desert	1.54
Primary	Mountain	2.06
Secondary	Desert	.77
Secondary	Mountain	1.03
Feeder	Desert	.39
Feeder	Mountain	.51

MULTIYEAR PMS/B&SMS OPTIMIZATION MODEL

Multiyear budget integration is a complex problem. This section develops a budget allocation such that the first-year budget is met while at the same time providing "smoothing" of the multiyear stratum budgets over the planning horizon leading to the desired steady-state goals. The first-year budget can be achieved if sufficient relaxation of both the performance goals and budget targets is allowed.

The following variables are used (in addition to those defined beforehand under steady-state) in the PMS/B&SMS multiyear optimization:

r = discount rate for computing net present value in cost minimization objective function.

$M_i^1(s)$ = index set of feasible PMS maintenance actions a for pavement in condition state i in stratum s that fix medium raveling, poor friction coefficient, or both.

$M_i^2(s)$ = index set of feasible PMS maintenance actions a for pavement in condition state i in stratum s that fix high raveling.

$\hat{w}_{ia}^1(s)$ = lower bound on the proportion of segments in stratum s that is in condition state i and should receive mandatory maintenance action/scope a in Year 1.

$q_i(s)$ = proportion of segments in stratum s in condition state i at beginning of Year 1.

$q_i^1(s)$ = proportion of pavement that is in PMS stratum s in condition state i at the beginning of Year 1 and that has either medium raveling or poor friction coefficient requiring action in set $M_i^1(s)$.

$q_i^2(s)$ = proportion of pavement that is in PMS stratum s in condition state i at beginning of Year 1 and that has high raveling requiring action in set $M_i^2(s)$.

$w_{ia}^*(s)$ = optimal proportions from the steady-state model for stratum s.
$C^*(s)$ = optimal average segment steady-state cost for stratum s.
$g(s)$ = sixth-year tolerance on steady-state optimal $w_{ia}^*(s)$ for stratum s.
$h(s)$ = sixth-year tolerance on steady-state optimal average segment cost $C^*(s)$ for stratum s.
$n_L^{t,t+1}(s)$ = parameter setting lower bound in budget balancing constraints for stratum s between years t and $t+1$.
$n_U^{t,t+1}(s)$ = parameter setting upper bound in budget balancing constraints for stratum s between years t and $t+1$.
$i_{\text{core}}(s)$ = B&SMS core condition state with same element condition levels as condition state i but does not include element-age parameters.
$I_{\text{core}}(s)$ = set of all core condition states for B&SMS stratum s (maximum of 64 bridges with a separate deck).
$i_{\text{EA}}(s)$ = set of B&SMS full condition states with core condition state $i_{\text{core}}(s)$, with all possible element ages for stratum s; there is a maximum of nine condition states in each set.
$w_{ia}^t(s)$ = proportion of segments in stratum s that is in condition state i and should receive maintenance action/scope a in year t; these are the output decision variables.
$E^t(s)$ = expected expenditures in year t in stratum s; this equals $N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s)\, C_{ia}(s)$.
$K_{\text{GP}}(s)$ = number of user-defined generalized performance constraints in stratum s.
$K_{\text{BF}}(s)$ = number of user-defined budget fluctuation constraints in stratum s.

The PMS/B&SMS optimization model is as follows. The constraints are shown only for an individual stratum s; however, they apply to all strata. This model divides into separable problems using the Lagrange multiplier α. Each problem is an individual stratum linear program. They are tied together externally through the Lagrange multiplier.

Parametric programming on the Lagrange multiplier α allows efficient solution of this problem. It takes only a fraction of the individual stratum solution time to get all solutions over the desired α range with parametric programming once the $\alpha = 0$ solution is found. This controls the total network budget ensuring that the optimal allocation across all strata meets the desired budget. Using this approach a series of optimal solutions versus budget is created. Management can easily see the advantages of different budget levels.

The PMS/B&SMS multiyear models are as follows:

For the benefit maximization objective function,

maximize

$$\phi_{\text{sys}} \sum_{t=1}^{T} N_n(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s)\pi_i(s)$$

$$- \alpha N(s) \sum_{t=t_L}^{t_u} \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s) C_{ia}(s)/(t_U - t_L + 1) \quad (10)$$

For the cost minimization objective function,

minimize

$$\sum_{t=1}^{T-1} N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} (1+r)^{1-t} w_{ia}^t(s) C_{ia}(s)$$

$$+ \alpha N(s) \sum_{t=t_L}^{t_U} \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s) C_{ia}(s)/(t_U - t_L + 1) \quad (11)$$

subject to (same constraints for cost minimization and benefit maximization)

$$w_{ia}^1(s) \ge \hat{w}_{ia}^1(s) \quad \text{for all } i \text{ in } I(s) \text{ and } a \text{ in } M_i(s), \text{ for Year 1 with mandatory projects in stratum } s \quad (12)$$

$$w_{ia}^t(s) \ge 0 \quad \text{for all } i \text{ in } I(s),\ a \text{ in } M_i(s), \text{ and } 1 \le t \le T \quad (13)$$

{Implicit in LP}

$$\sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s) = 1 \qquad 1 \le t \le T \quad (14)$$

$$\sum_{a\in M_i(s)} w_{ia}^1(s) = q_i(s) \qquad \text{for all } i \text{ in } I(s) \quad (15)$$

$$\sum_{a\in M_i^1(s)} w_{ia}^1(s) \ge q_i^1(s) + q_i^2(s) \qquad \text{for all } i \text{ in } I(s) \text{ (PMS only)} \quad (16)$$

$$\sum_{a\in M_i^2(s)} w_{ia}^1(s) \ge q_i^2(s) \qquad \text{for all } i \text{ in } I(s) \text{ (PMS only)} \quad (17)$$

$$\sum_{a\in M_j(s)} w_{ja}^t(s) - \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^{t-1}(s) P_{iaj}(s) = 0 \qquad \text{for all } j \text{ in } I(s) \text{ and } 2 \le t \le T \quad (18)$$

$$\sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s)\Phi_{ik}(s)\ (\ge \text{ or } \le)\ p_k(s) \qquad k = 1, \ldots, K_{\text{GP}}(s) \quad (19)$$

$$E^t(s) - \sum_{t=t_L}^{t_U} [E^t(s)/(t_U - t_L + 1)][1 + \delta_k^t(s)]\ (\le \text{ or } \ge)\ 0 \qquad k = 1, \ldots, K_{\text{BF}}(s) \quad (20)$$

$$N(s) \sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^t(s) C_{ia}(s)\ (\ge \text{ or } \le)\ \$_k(s) \qquad k = 1, \ldots, K_{\text{BL}}(s) \quad (21)$$

$$\sum_{i\in I(s)} \sum_{a\in M_i(s)} w_{ia}^T(s) C_{ia}(s) \le [1 + h(s)] C^*(s) \quad (22)$$

$$\sum_{a\in M_i(s)} w_{ia}^T(s) \ge \sum_{a\in M_i(s)} [1 - g(s)] w_{ia}^*(s) \qquad \text{for all } i \text{ in } I(s) \quad (23)$$

$$\sum_{a \in M_i(s)} w_{ia}^T(s) \leq \sum_{a \in M_i(s)} [1 + g(s)] w_{ia}^*(s) \quad \text{for all } i \text{ in } I(s) \quad (24)$$

$$\sum_{i \in [i_{EA}(s)]} \sum_{a \in M_i(s)} w_{ia}^T(s) \geq \sum_{i \in [i_{EA}(s)]} \sum_{a \in M_i(s)} [1 - g(s)] w_{ia}^*(s) \quad \text{for all } i_{EA}(s) \text{ in } I(s) \quad (25)$$

$$\sum_{i \in [i_{EA}(s)]} \sum_{a \in M_i(s)} w_{ia}^T(s) \leq \sum_{i \in [i_{EA}(s)]} \sum_{a \in M_i(s)} [1 + g(s)] w_{ia}^*(s) \quad \text{for all } i_{EA}(s) \text{ in } I(s) \quad (26)$$

The first term of the benefit maximization objective function in Equation 10 maximizes benefits, and the second Lagrange term enforces the first-year budget constraint (though it can be used to control the average budget expenditures also). The first term of the cost minimization objective function (Equation 11) minimizes the average present value cost per segment of maintenance over the time horizon of interest, and the Lagrange second term serves the same purpose as in the benefit maximization objective function.

Constraint 12 handles the mandatory projects. Equation 13 (implicit in linear programming) ensures that the decision variables are nonnegative. Equation 14 forces the sum of the proportions in each year to equal 1, and Equation 15 ensures that the first-year boundary conditions are satisfied. PMS Equations 16 and 17 account for the necessary action upgrades to handle friction and raveling problems. Equation 18 is the probabilistic mass balance (ensures the proper transfer from one year to the next) equation from one year to the next. Equation 19 is the generalized performance constraints that allow considerable flexibility in goal setting by the decision makers. Equation 20 bounds the variability allowed from year to year in the optimal budget. Equation 21 allows the optional inclusion of stratum budget bounds on any given year: these may be upper or lower bounds. Equation 22 enforces the steady-state budget constraint. Equations 23 and 24 are the PMS-only steady-state performance constraints, and Equations 25 and 26 are the same for the B&SMS.

FURTHER DISCUSSION

The HMMS allows both cost minimization and benefit maximization. In cost minimization there is no need for the many parameters introduced that in essence weight some aspect of pavement versus bridges. The coefficients used in the benefit maximization model presented here represent the specific values of one realization of this system: the Kingdom of Saudi Arabia. These values represent the combined interactive efforts of a multinational task force overseen by the World Bank and its consultants. Although such values are not always easy to obtain and agree upon, they do represent rationale tradeoffs for estimating the significance of pavement versus bridges.

In the cost minimization mode, one can minimize cost with or without user cost (*10*). Thus the HMMS allows the minimization of agency cost or user cost in addition to the maximization of benefits as defined in this paper.

Each agency should evaluate its own set of parameters so that the weights are reflective of its values. The sensitivity of the results relative to the parameter values may be readily tested since the key parameters are used in the objective function. As an example, efficient parametric programming may easily determine the impact of changes in ϕ_{sys}.

The benefit maximization run shown in the next section was done for Kingdom of Saudi Arabia. Many additional runs may be found (*1–3*). The run shown in the next section had to meet specified performance goals. Subject to meeting those goals it is clear that the benefit maximization wanted to allocate proportionally more additional funds to the bridge system when more money was available. In this example this is primarily due to the bridges' being weighted more heavily. It can be shown theoretically that the benefit maximization first-year results asymptote as the Lagrange multiplier increases to a cost minimization (with the same performance goals). Thus the higher weighting of bridges versus pavement tends to shift supplemental funding (above the minimum needed to achieve the performance goals) to bridges in this case.

Traffic is introduced into the optimization in two ways. First, the functional class acts as a surrogate for traffic. Second, the condition prediction models in the pavement system (*2*) directly use traffic in distress estimation that results in the transition probabilities.

In the bridge system the secondary functional class includes bridges on secondary and feeder roads. Thus, the secondary bridge functional class weight is between the secondary and feeder functional class weights for pavement.

EXAMPLE RUN

Figure 1 graphs the total PMS and B&SMS network (all strata) budget as a function of the Lagrange multiplier α. The budget is a monotonically decreasing function of the Lagrange multiplier. As the budget is reduced the optimal mix across all bridge and pavement strata is determined. This ensures that the best use is made of the scarce resources available.

In this example the total budget decreases 71 percent over the range of the Lagrange multiplier shown. Most of this comes from a corresponding 76 percent reduction in the B&SMS budget, whereas the PMS budget was reduced only 35 percent. These runs are based on multiyear benefit maximization. In all cases shown the performance goals specified for each stratum were met; however, since this was a benefit maximization run, it attempted to achieve the most benefit possible. For benefit maximization when the Lagrange multiplier $\alpha = 0$, this corresponds to an unconstrained cost situation. So it is not surprising that the budget can be significantly reduced and still meet the performance goals. There is no significant drop in the total budget for values of the Lagrange multiplier larger than shown in Figure 1.

SUMMARY

Optimization models have been presented for steady-state and multiyear pavement and bridge management systems. These optimization models integrate the pavement and bridge management systems so that management can optimally allocate resources across the combined system. The use of importance weights and fuzzy set memberships was discussed.

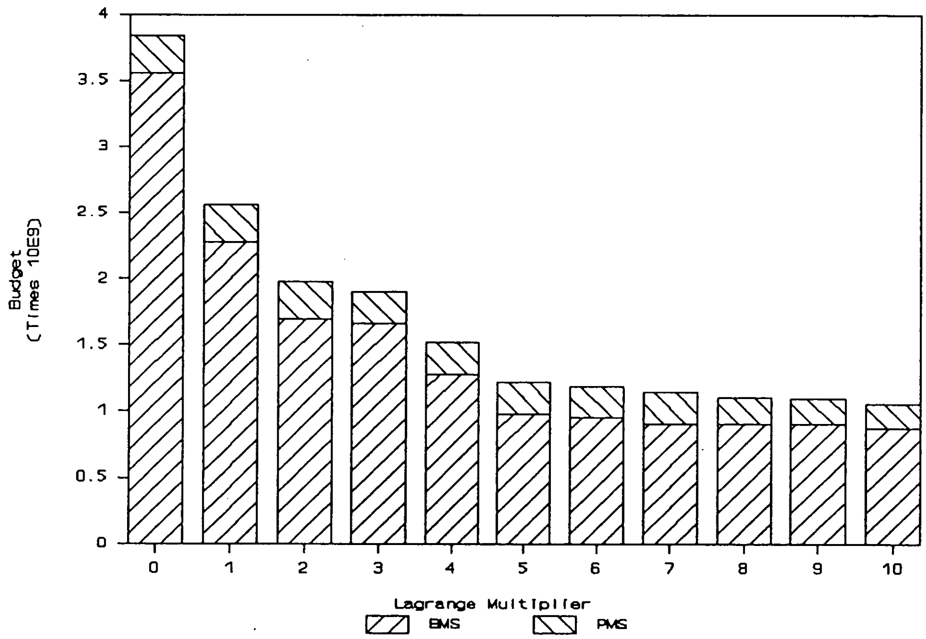

FIGURE 1 Total budget as a function of Lagrange multiplier.

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Service Lives and Costs of Local Highway Maintenance and Rehabilitation Treatments

JOHN COLLURA, GARY SPRING, AND KENNETH B. BLACK

Reliable estimates of the service life and cost of typical maintenance and rehabilitation (M&R) treatments are very important in the implementation of pavement management systems. The primary objective of this project was twofold: (a) to develop and test a survey questionnaire that may be used to obtain reliable estimates of service lives and costs of maintenance and rehabilitation treatments commonly used on local roads in Massachusetts and other parts of New England, and (b) to use the survey questionnaire to estimate the service lives and costs of such treatments in Massachusetts. Sixty-eight cities and towns in Massachusetts were surveyed. The data were analyzed to estimate the service life and cost of thin overlays, chip seals, and sand seals; they were also used as a basis for developing performance curves.

Capital available for expenditures on local highway improvement projects has steadily decreased over the past decade, as the highway infrastructure continues to age (*1*). Consequently, emphasis has been placed on maintaining that infrastructure. Yet more than 40 percent of U.S. highways may be classified as being in fair to poor condition (*2*). This indicates a need to allocate limited resources for maintenance and rehabilitation (M&R) more efficiently, especially for small cities and towns that constitute a significant proportion of the total paved road mileage in the United States.

High-quality maintenance is an important determinant of pavement performance; it can slow the rate of pavement deterioration due to loads. Many small city and town agencies take a "worst first" approach to their maintenance activities, which often is not cost-effective. Deferred maintenance allows the severity of defects to worsen. Continued deferral of M&R actions can shorten the time between construction and reconstruction and increase the cost of reconstruction by as much as four to five times, thus significantly increasing the life-cycle costs of a pavement (*3*).

Maintenance plans consist of determining not only when an improvement should be made but also what type should be used. More effective decisions about when and which treatment should be applied (a variety of alternative treatments may be used for different types and levels of pavement distress) require good estimates of pavement service lives and costs. These estimates may be used for various activities (*4–7*):

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- Estimating and allocating available funds,
- Identifying cost-effective solutions,
- Anticipating when necessary expenditures will recur, and
- Justifying work plans to elected officials.

State and local pavement maintenance records are not typically well kept (*8*), thus, expected life and cost information is not generally readily available. The best life expectancy information appears to be in the heads (and archaic records) of experienced highway superintendents who have seen many cycles of maintenance activities (*4,9*). This unrecorded information is, however, being lost as these individuals retire. These data would be an invaluable aid to many local highway superintendents in devising maintenance work plans. With regard to the pavement management needs of small cities and towns, FHWA's Rural Technical Assistance Program over the past few years has focused on training and pavement design (*10*). Little if any effort has been made to examine the life-cycle costs of maintenance options typically used by small cities and towns, such as thin overlays, seal coats, slurry seals, and surface treatments.

Several studies have been conducted in recent years to ascertain some usable values that could be used to make better decisions about low-cost pavement maintenance activities. An Ontario survey examined average service lives of maintenance treatments that included crack seals, chip seals, and thin overlays (*11*). An Indiana survey of 33 superintendents and highway foremen examined minimum, average, and maximum service lives for routine maintenance activities on roadways in poor, fair, and good condition (*4*).

Many other factors, however, affect pavement life (*12*). Neither of the studies in Indiana or Ontario included in its analysis many of these important factors. A New Hampshire survey was designed to include present pavement condition, daily truck volumes, drainage, and pavement structures as variables affecting pavement life (*13*). The survey was never carried out, presumably because the questionnaire was too long. Estimates of the lives and costs of maintenance treatments in the New England region considering these other variables would be very useful.

PROJECT OBJECTIVE

The objective of this project was twofold: (a) to develop and test a survey questionnaire to collect service life and cost data about local pavement M&R practices in Massachusetts and other parts of New England, and (b) to use the questionnaire to estimate the service lives and costs of such practices in Massachusetts.

SURVEY QUESTIONNAIRE

The general design of the questionnaire reflects what was learned from studying previous efforts just described. The questionnaire is laid out in tabular-matrix format. To reduce the length of time required to gather this type of information, the questionnaire is divided by type of maintenance treatment. Types chosen for inclusion are the ones most often used in New England (*9*) and are described in Table 1.

Several major factors affect the performance of a maintenance treatment. Accordingly, the instrument should be adequate to capture the factors that are most important in determining the life of a particular treatment, including severity and extent of loadings to which the pavement structure will be subjected (truck volumes), general condition of pavement structure, and pavement condition before treatment (*4,11–13*).

Truck Volumes and Pavement Structure

The effects of truck volume levels on pavement condition are well documented (*14–16*). Since surface treatments provide no structural capacity to the pavement, load levels and the condition of its substructure (see Table 1 for definitions used in this study) have a fundamental relationship with the structure's overall performance and life. For this study, truck volumes were placed into three load levels: light (less than 20 per day), medium (between 21 and 125 per day), and heavy (greater than 125 per day). A truck is defined as any vehicle with a gross vehicle weight greater than 10,000 lb. This was considered adequate, given the low degree of sensitivity that pavement performance exhibits with changes in volume—the functional form of the AASHTO equations is logarithmic,

TABLE 1 Definition of Survey Questionnaire Variables

Maintenance Treatments (Seal Types)

Sand Seal	Chip Seal	Overlay (1.905 cm)	Overlay (2.54 - 3.81 cm)
• Application of low viscosity or moderately diluted asphalt covered with fine (sand gradation) aggregate. Low viscosity and sand combination is designed to fill many small cracks on the existing surface.	• Single application of liquefied asphalt followed by single layer of uniform size aggregate.	• Thin layer of hot mix asphalt concrete.	• Thicker layer of hot mix asphalt.

Present Pavement Conditions

Fair	Poor
• Moderate to severe ravelling. • Longitudinal and transverse cracks (up to 1.27 cm) show first signs of slight ravelling and secondary cracks. First signs of longitudinal cracks near pavement edge. • Block cracking up to 50% of surface. • Extensive to severe bleeding or polishing. • Some patching or edge wedging in good condition.	• Closely spaced longitudinal and transverse cracks often show ravelling and crack erosion. • Severe block cracking. • Some alligator cracking (<25% of surface). • Patches in fair to poor condition. • Moderate rutting or distortion (2.54 - 5.08 cm deep). • Occasional potholes.

Pavement Structure Conditions

Good	Fair	Poor
• Suitable capacity for anticipated truck volumes. Has good drainage conditions.	• Marginally suited for anticipated truck volumes and/or has fair drainage.	• Inappropriate for anticipated truck volumes and/or has poor drainage.

Drainage Conditions

Good	Fair	Poor
• Ditches, culverts, and inlets are clean. Road shoulders slope away from roadway.	• Ditches, culverts and inlets are fairly clean. Road shoulders slope down and away from roadway.	• Ditches neither function nor exist. Culverts and inlets, if present, are clogged. Road shoulders are often higher than roadway. Extensive frost heaving.

thus requiring an order of magnitude shift in loadings to affect significantly the pavement's structural capacity and therefore its life.

Pavement Condition Before Treatment

The positive correlation between maintenance level and maintenance cost is also a well-documented phenomenon (*16*). The condition of the pavement at the time of treatment certainly influences the type of treatment appropriate as well as its expected life (i.e., life-cycle costs). As the benefit derived [i.e., Δ PCI (pavement condition index)] from making an improvement increases, so does its cost (*3*). Quantitative and qualitative estimates of condition were as defined in Table 1.

Improvement Costs

Highway superintendents are comfortable using unit costs of various maintenance options, both in terms of manpower and materials, and are familiar with variations in costs due to changes in road or climatic conditions. Included on the questionnaire were questions about unit capital costs for each treatment type.

CONDUCT OF SURVEY

The survey questionnaire was used to interview 68 local highway superintendents in Massachusetts. The commonwealth has two somewhat different geographic areas with regard to both climate and engineering characteristics of pavement substructures. The eastern part of the commonwealth is generally low-lying flatlands with sandy soils, whereas the western part is characterized by rolling hills at higher elevations with gravelly soils. Figure 1 depicts the spatial distribution of cities and towns surveyed, and Tables 2 and 3 provide a list of towns and the treatments used. In choosing superintendents to be interviewed, an effort was made to find a person (or persons) in each agency with sufficient experience regarding the level of truck volumes on local roads, characteristics of pavement substructure material, and other factors examined in the questionnaire.

SURVEY RESULTS AND ANALYSIS

Data were obtained with the survey questionnaire to estimate the service life and cost of major treatments used in each region, and in selected instances these data were used to develop performance curves.

Service Life and Cost

The data were tabulated and analyzed using a Microsoft Excel spreadsheet program. For each cell in the questionnaire, all responses were used to estimate the mean service life, in years, and the standard deviation. After this, responses more than two standard deviations from the mean were identified as outliers and removed from the data set. In several cases, the outliers were found to be responses from young and less-experienced personnel.

Tables 4 and 5 present the estimated service lives of 3.81-cm overlays and chip seals in each region for certain conditions. Table 6 gives the estimated service lives of sand seals and 1.91-cm overlays on pavements in fair condition for the combined regions. Because survey response rates were low for these two alternatives, the data from the eastern and western regions were combined. Table 7 summarizes the costs of all four treatments.

Performance Curves

A variety of curve shapes have been proposed to model pavement performance (*17*). Because it has not yet been shown that the more complicated mathematical forms yield notably better results than the simple mathematical function, it was decided that an exponential function would be used for this study.

The general form of the exponential curve

$$\mathrm{PCI} = ae^{bt} + k$$

contains three unknowns: a, b, and k. At least three ordered pairs (t and PCI) are necessary to calibrate this general form to our specific case. Two points are relatively easily and directly obtained. They are

$t_1 = 0$, $\mathrm{PCI}_1 = 100$ when pavement treatment is new, and

t_2 = mean age, $\mathrm{PCI}_2 = 50$ when pavement is in fair condition.

The value t is the average service life determined from the survey questionnaire. The third ordered pair must be estimated. Point t_2, PCI_3, represents the time and PCI when the pavement has deteriorated to poor condition. As the interview process progressed and preliminary modeling was being done, it became evident that the location of this third point was necessary in order to estimate performance curves. An additional question was asked of all the later interviewees: "If no maintenance is done, how long will it take for the pavement to deteriorate from fair to poor?" The answer to this question, added to the value of t_2, yields t_3. The value of PCI (poor condition) was set at 30. With these three data points, calibrating the model and estimating a, b, and k was a straightforward procedure.

Once the model was calibrated, it was a simple task to generate curves with data from the survey. Pavement structure and pavement condition were held constant (i.e., good and fair), and sets of graphs were prepared for different levels of truck traffic.

SUMMARY AND CONCLUSIONS

Local pavement management efforts continue to be carried out by cities and towns, so there is a need for good, reliable estimates of service life, cost, and other measures of perfor-

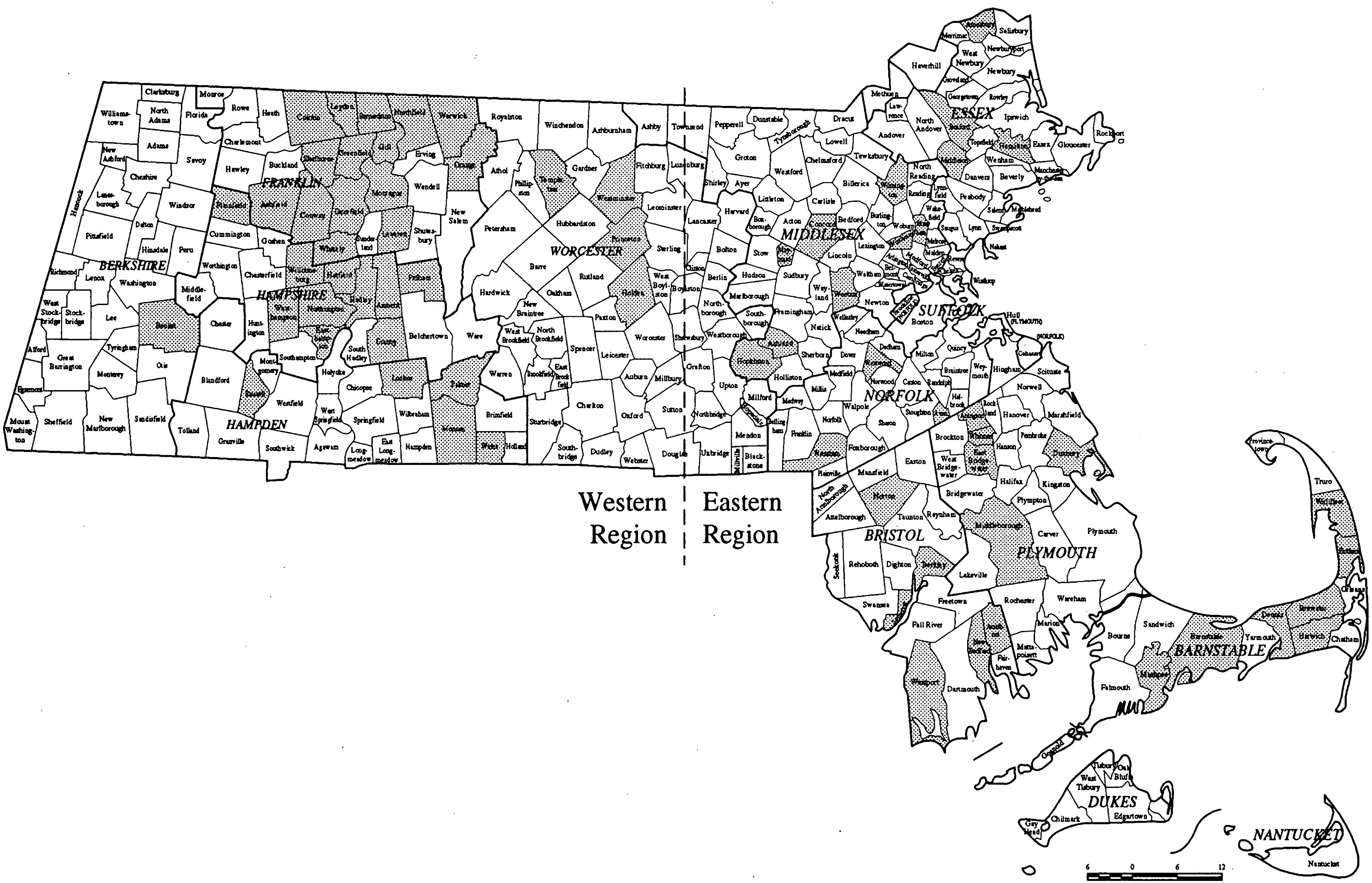

FIGURE 1 **Massachusetts survey sites.**

TABLE 2 Towns and Treatments, Eastern Region

TOWN	population	sand seal	chip seal	overlay 1.905 cm	overlay 3.81 cm
Abington, MA	13,810		x	x	
Acushnet, MA	8,970	x			x
Amesbury, MA	14,056				x
Ashland, MA	12,008				x
Avon, MA	4,770	x	x		x
Barnstable, MA	36,431		x		x
Bedford, MA	12,660		x		x
Berkley, MA	3,920	x		x	
Boxford, MA	6,449	x	x		x
Brewster, MA	7,876				x
Concord, MA	15,424		x		x
Dennis, MA	13,173	x	x		x
Duxbury, MA	14,168		x		x
East Bridgewater, MA	10,640		x		x
Eastham, MA	4,600	x			
Hamilton, MA	7,190				x
Harwich, MA	10,055	x	x		x
Hopkinton, MA	9,150	x			x
Mashpee, MA	9,543	x	x		x
Maynard, MA	10,357	x	x		x
Middleborough, MA	17,838		x		x
Middletown, MA	5,082				x
Norton, MA	14,344	x	x		x
Seekonk, MA	13,040		x		x
Somerset, MA	17,690		x		x
Stoneham, MA	22,590			x	x
Wellfleet, MA	2,610	x			x
Weston, MA	10,600				x
Westport, MA	13,241	x			x
Westwood, MA	12,600				x
Whitman, MA	13,137	x			x
Wilmington, MA	18,070			x	x
Winchester, MA	20,858			x	
Wrentham, MA	8,868				x

TABLE 3 Towns and Treatments, Western Region

TOWN	population	sand seal	chip seal	overlay 1.905 cm	overlay 3.81 cm
Amherst, MA	31,740		x		x
Ashfield, MA	1,620		x		x
Becket, MA	1,700		x		x
Bernardston, MA	1,820		x		x
Colrain, MA	1,690		x		x
Conway, MA	1,515		x		
Deerfield, MA	4,830		x		x
Easthampton, MA	16,160		x		x
Gill, MA	1,452		x		x
Granby, MA	5,710		x		x
Greenfield, MA	17,950			x	
Hadley, MA	4,300		x		x
Hatfield, MA	3,110		x		x
Holden, MA	14,767	x		x	x
Leverett, MA	1,660		x		x
Ludlow, MA	18,146				x
Monson, MA	8,000		x		x
Montague, MA	8,994		x		x
Northampton, MA	30,384				x
Northfield, MA	2,600		x		x
Orange, MA	7,346		x		
Palmer, MA	12,120	x			x
Pelham, MA	1,452		x		x
Plainfield, MA	480		x		x
Princeton, MA	3,200	x	x	x	x
Russell, MA	1,475		x		x
Shelburne, MA	2,000		x		
Templeton, MA	6,408	x		x	x
Wales, MA	4,700	x			x
Warwick, MA	600		x		x
Westhampton, MA	1,403		x		x
Westminster, MA	5,870	x	x	x	x
Whately, MA	1,390		x		x
Williamsburg, MA	2,600		x		x

TABLE 4 Pavement Life: 2.54- to 3.81-cm Overlay, Eastern and Western Regions

EAST

Present Pavement Condition	Daily Truck Traffic	Pavement Structure Good	Fair	Poor
	Low	14.9	11	6
F	0 - 20	n=26 s=2.4	n=26 s=3.3	n=1
A	Moderate	11.8	7.9	5
I	21-125	n=26 s=3.6	n=24 s=3.1	n=1
R	High	9.4	6.2	3.5
	>125	n=20 s=3.4	n=16 s=2.2	n=1
	Low	10.3	6	6
P	0-20	n=3 s=2.3	n=13 s=2.4	n=1
O	Moderate	8.8	5.1	4.5
O	21-125	n=4 s=1.1	n=12 s=1.8	n=1
R	High	6.5	4.1	2.5
	>125	n=4 s=1.8	n=9 s=1.6	n=1

WEST

Present Pavement Condition	Daily Truck Traffic	Pavement Structure Good	Fair	Poor
	Low	13.2	9.4	7.2
F	0 - 20	n=20 s=3.7	n=16 s=2.8	n=9 s=2.0
A	Moderate	12	8.4	6.1
I	21-125	n=19 s=5.1	n=18 s=3.8	n=9 s=2.6
R	High	7.7	5.9	4.7
	>125	n=11 s=2.6	n=19 s=3.2	n=5 s=3.0
	Low	11.2	9.8	4.6
P	0-20	n=10 s=3.6	n=9 s=5.1	n=3 s=1.3
O	Moderate	8.5	6.6	2
O	21-125	n=11 s=3.3	n=7 s=2.9	n=2 s=1.0
R	High	6.4	3	4.3
	>125	n=8 s=2.3	n=3 s=1.1	n=3 s=2.4

TABLE 5 Pavement Life: Chip Seals, Eastern and Western Regions

EAST

Present Pavement Condition	Daily Truck Traffic	Pavement Structure Good	Fair	Poor
F	Low	8.7	6.3	4.6
A	0 - 20	n=17 s=1.5	n=15 s=1.9	n=8 s=2.0
I	Moderate	8	4.5	3.2
R	21-125	n=16 s=2.3	n=13 s=1.8	n=6 s=2.2
P	Low	7.4	4	2.1
O	0-20	n=7 s=2.2	n=8 s=1.6	n=5 s=1.2
O	Moderate	6.1	2	1.2
R	21-125	n=7 s=2.1	n=6.0 s=0.7	n=5 s=0.2

WEST

Present Pavement Condition	Daily Truck Traffic	Pavement Structure Good	Fair	Poor
F	Low	6.75	4.7	3.9
A	0 - 20	n=20 s=1.4	n=15 s=1.1	n=7 s=1.4
I	Moderate	5.2	3.8	3
R	21-125	n=15 s=1.2	n=6 s=1.1	n=43 s=1.4
P	Low	5.7	9.8	3.1
O	0-20	n=13 s=1.7	n=9 s=5.1	n=7 s=1.7
O	Moderate	4.8	6.6	2.3
R	21-125	n=10 s=1.5	n=7 s=2.9	n=3 s=1.3

TABLE 6 Pavement Life: Fair Condition, Combined Regions

Daily Truck Traffic	Pavement Structure Good	Fair
Sand seals		
Low	5.9	3.5
0–20	$n = 20$ $s = 2.7$	$n = 17$ $s = 1.5$
1.905-cm overlay		
Low	11.7	6.1
0–20	$n = 11$ $s = 5.4$	$n = 9$ $s = 2.0$
Moderate	7.4	4.3
21–125	$n = 12$ $s = 2.4$	$n = 9$ $s = 2.0$

TABLE 7 Costs of Treatments, Combined Regions

Treatment	Number of Observations	Cost	Standard Deviation
Sand seal	8	\$ 0.43/yd²	\$0.22
Chip seal	24	\$ 0.80/yd²	\$0.32
Bituminous overlay[a]	47	\$30.36/ton	\$3.88

[a]1.905- and 2.54- to 3.81-cm overlays combined.

mance. The survey questionnaire developed and employed in this project serves as a tool to obtain the data required to make these estimates. Such estimates of performance will facilitate the use of personal computer–based pavement management systems and, in turn, provide a decision aid for more efficient and effective allocation of limited pavement maintenance resources.

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Comprehensive Ranking Index for Flexible Pavement Using Fuzzy Sets Model

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Pavement management at the network level usually requires an index to select candidate projects and rank them for scheduling maintenance and rehabilitation activities. Such an index should consider all the factors that affect pavement performance. One of the problems in pavement prioritization is that there is no absolute attribute value at which a pavement has failed. Instead, it is a matter of acceptability. The acceptability of a pavement's condition involves largely the subjective judgment of the pavement engineers and the pavement users. A methodology to develop an index model called the overall acceptability index for flexible pavements using the fuzzy sets concept is presented. The methodology can capture the subjective judgment of the pavement engineer and the pavement user and combine the most important pavement attributes such as roughness, distress, structural capacity, and skid resistance into one index. A case study to apply the methodology is included, and the results are discussed.

The development of systematic procedures for scheduling maintenance and rehabilitation (M&R) activities is one of the major concerns of state and federal highway agencies. Over the years, funding for M&R activities has not been able to keep pace with the needs, resulting in a backlog of projects for many of the agencies. Such problems demand good management of road networks and have led to increased interest in the implementation of pavement management systems (PMSs). FHWA also requires that each state's department of transportation (DOT) have a PMS in use by 1993.

A PMS normally operates at two levels: project and network. Activities at the project level are concerned with specific technical and management decisions for each individual project; activities at the network level are mainly the responsibility of administrators who are primarily concerned with making decisions covering groups of projects or highway links up to an entire highway network (*1,2*). Detailed technical data are not of major concern at this level. At the network level, pavement evaluation measures are used to assess the relative adequacy of each pavement link or section. From this, decisions are made on what projects to include in upcoming M&R work programs. The selection of candidate highway links for M&R work is done through an optimization analysis using condition data. Scores are generally calculated for each evaluation measure per pavement section using a procedure established within the particular agency involved. The scores obtained can then be combined into a single index to establish priorities for M&R work.

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One of the problems in pavement prioritization or optimization is that there is no absolute attribute value at which the pavement has failed. Instead, it is a matter of acceptability. The acceptability of the pavement condition involves largely the subjective judgment of the person or persons using the highway or making the decisions. To develop a rational index for the selection and prioritization of the candidate sections for M&R, such subjective effects must be considered in the index formulation.

Many factors affect the performance of a pavement. Flexible pavements can usually be evaluated by four attributes or evaluation measures:

1. Roughness,
2. Surface distress,
3. Structural capacity, and
4. Skid resistance between the tire and pavement surface.

Each of the four attributes, however, evaluates only one aspect of pavement performance. Therefore, it is necessary to develop an index that considers all four attributes together to give an overall performance evaluation.

The development of a combined performance index for pavements is also a necessary requirement on the system output function for the pavement management process. Such a combined index should take into the consideration both the subjective judgment of decision makers and the most important attributes of pavement.

This paper documents the use of fuzzy set theory to model the subjective decision-making process involved in selecting candidate pavement sections for M&R. The specific application discussed is the formulation of a prioritization index for flexible pavements. The approach adopted is expected to lead to a more realistic and rational way of evaluating candidate sections for priority programming at the PMS network level. A review and evaluation of several approaches to formulating such an index is made to provide background information on existing practices.

The development of a model called the overall acceptability index (OAI), which is based on fuzzy set theory, is discussed. Fuzzy set theory is briefly discussed, and the OAI model is presented. The data for formulating the model are based on a survey of persons who have knowledge and experience in the field of pavement engineering. Data for the model presented in this paper were obtained by surveying faculty and students in the pavement study area at the University of Texas. The four pavement attributes listed earlier were considered.

Regression analysis was conducted on the data from the survey to obtain the membership functions. The results from the regression analysis are discussed, and the conclusions of the study are presented with recommendations for future research activities.

REVIEW OF EXISTING ALTERNATIVE APPROACHES TO FORMULATING A COMBINED INDEX

As discussed earlier, an important phase of M&R programming is the selection of candidate highway links. A combined rating or index is used to express the overall condition of the pavement section or highway link in terms of a combination of selected attributes and the subjective judgment of the decision makers. There are different approaches to develop such a combined index in pavement area; a brief review of several of them follows.

Unique Sums Approach

The unique sums approach is characteristic of a rating system used in Sweden (*3*), in which road sections are classified with respect to the variables pavement wear, deformation (roughness and cracking), and amount of treatment in routine maintenance. For each variable, levels are identified that indicate the extent of distress (none obvious, considerable, serious); for each level, a class number and a rating are assigned.

Each road section is therefore characterized by the three ratings, which are then added to give a composite rating. The rating numbers were chosen in such a way that the sum of numerical values for every combination of variable levels is unique, that is, each sum is different from the other sums.

Utility Theory

Texas DOT is using the utility theory to develop a measure of overall pavement performance (*4*). Basically, the procedure involves the establishment of utility functions that express a decision maker's preference over different levels of selected attributes. These functions are developed by acquiring expert opinion through interviews. A utility curve will be constructed for each pavement attribute selected. A composite measure of pavement performance can then be obtained by combining the utility curves into a single equation. The procedure assumes mutual preferential independence between attributes. The intuitive meaning of this condition is that there is no interaction of preference between attributes. Priority can then be established by comparing the relative values obtained from the combined multiattribute utility function.

Delphi Method

The Delphi technique is another method that can be used to formulate a prioritization index. This method has been used in Texas and Maine to evaluate pavement condition (*5,6*). In it, an attempt to achieve a consensus among a group of experts is made through cycles of intensive questioning interspersed with controlled opinion feedback. The technique avoids the direct confrontation of experts with one another, which is the traditional method of pooling individual opinions. In this way, some of the serious difficulties inherent in face-to-face interaction are circumvented. The final output of the process is a set of importance ratings reflecting the group consensus that may be used for establishing priorities.

Factorial Rating Method

The factorial rating method was proposed by Fernando (*7*). Essentially, the formulation of an index using this method is based on a factorial design consisting of the following factors:

1. Degree of pavement distress,
2. Present serviceability index (PSI),
3. Traffic, and
4. Environmental condition of rainfall and freeze-thaw cycles.

The application of the method involved the participation of many highway engineers, who were asked to give their opinions on the establishment of rehabilitation priorities. The responses obtained were then evaluated with the hope of gaining a better understanding of the ways in which pavement engineers establish priorities in actual practice. The prioritization procedure was based on the results of the survey.

BASIC CONCEPTS OF FUZZY SET THEORY

The concepts of fuzzy set theory were introduced by Zadeh in 1965 (*8*). It is especially useful for the representation of imprecise knowledge of the type that is prevalent in human concept formulation and reasoning. For example, the linguistic terminology of old and young, good and bad, acceptable and unacceptable, and so on are all imprecise concepts.

Concept of Classical Set and Fuzzy Set

A fuzzy set, in its basic sense, is a set in which objects have a gradual rather than an abrupt transition from membership to nonmembership. In conventional (classical) set theory, either an object belongs to a set U or it does not; the characteristic (membership) function f_u can be represented as:

$$f_u = \begin{cases} 1 & u \text{ belongs to } U \\ 0 & u \text{ does not belong to } U \end{cases}$$

The concept of fuzzy sets extends the range of membership value for f_u and allows graded membership transition, usually defined on an interval [0,1]. Consequently, an object may belong to a set with a certain degree of membership. Figure 1 illustrates this concept.

Methods for Determination of Membership Function

The membership function can be determined in actual applications with several methods; a few of them are briefly described in the following.

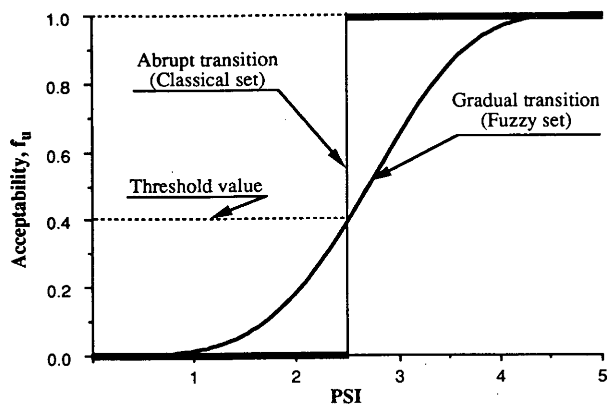

FIGURE 1 Concept of classical set and fuzzy set.

Normative Method

In the normative method, the membership function is defined or selected directly by the users according to the nature of the problem and the user's experience in the field. Membership function tables are also available in some of the fuzzy set references (*9*).

Binary Direct Rating

For the binary direct rating, a group of persons is asked to answer yes or no according to whether the linguistic term describes the element or not. Regression analysis is then used to obtain the membership functions.

Continuous Direct Rating

In the continuous direct rating, a group of persons rates elements on a predesigned continuous scale from "definitely in the concept" to "definitely not in the concept." Regression analysis is then used to obtain the membership functions.

OAI MODEL

Each of the four pavement attributes roughness, distress, structural capacity, and skid resistance has different categories of severity. However, there is not a distinct transition point between the various categories but a gradual transition. For example, in the AASHO road test (*10*) the 50th percentile for acceptability occurs when the present serviceability rating (PSR) is approximately 2.9. This means that the pavement is acceptable, with respect to roughness, when the PSR is above the threshold value of 2.9. This is therefore an ideal opportunity to apply fuzzy set theory to ascertain category membership.

Thus, the next step is to apply the fuzzy set theory to the four pavement attributes that are recognized as the major factors affecting pavement performance. For each of the attributes, the description of the categories will be either "acceptable" or "unacceptable." To consider the relative importance of the attributes, a weighting value will be given to each of them. These weighting values will also be obtained from an opinion survey. With these considerations in mind, the factor set, description set, and weight set will be as follows:

- Factor set = {roughness, distress, structural capacity, skid resistance},
- Description set = {acceptable, unacceptable}, and
- Weight set = $\{w_1, w_2, w_3, w_4\}$ where $\Sigma w_i = 1$.

For each of the factors in the factor set, a membership value A_i can be obtained from the membership curve, and the OAI can be expressed as

$$\text{OAI} = \left(w_1/\sum w_i\right)A_1 + \left(w_2/\sum w_i\right)A_2 + \left(w_3/\sum w_i\right)A_3 + \left(w_4/\sum w_i\right)A_4$$

One of the important advantages of this model is that it will always ensure that the weighting values $(w_i/\Sigma w_i)$ sum to 1, even when one of the attributes is deleted from the model. For example, in a city PMS pavement structural capacity data may not be collected. In this case the index will still be valid. Because of the linear combinations of the individual acceptabilities, the model is easy to understand and operate.

The following section addresses the construction of the practical membership functions for each of the four attributes in the factor set and the corresponding weighting values.

SUBJECTIVE OPINION SURVEY AND DATA PROCESSING

To construct the membership functions for the OAI model, it was decided to conduct a subjective opinion survey about the level of acceptance for the selected pavement attributes and their relative importance.

Roughness is measured by the existing PSI. PSI is primarily a function of pavement roughness and is measured on a scale of 0 to 5. A PSI value of 0.0 indicates an extremely rough pavement and therefore totally unacceptable, and a value of 5.0 corresponds to the roughness of a well-constructed new pavement (*10*).

On asphalt pavements there are many types of distress: fatigue cracking, temperature/moisture cracking, rutting, and so on. However, for network-level purposes it is necessary to perform not detail analysis but analysis suited for overall planning. Therefore, the measures for various distresses are aggregated into a single measure. In this study the aggregated measure is defined as the percentage of distressed area. This means, for example, that if 20 percent or more of the survey section suffers from any type of distress, the entire section will be judged as acceptable or unacceptable.

Structural adequacy is essential for a pavement to serve traffic. It is usually measured using a falling weight deflectometer or a Benkelman beam. In this study, structural capacity of a pavement is measured as a percentage of its capacity when newly constructed or relative to the capacity of some other new pavement having a similar structure.

Skid resistance is an indirect measure of safety. The coefficient of friction determines the skid resistance of a pavement. Theoretically, the maximum coefficient of friction is 1.0 and the minimum is 0.0. In practice, however, the maximum usually attained on a newly constructed, dry pavement is about 0.8. On an old, wet pavement, which represents the worst condition, the coefficient of friction is approximately 0.2.

To characterize the degree of acceptability of the four attributes, it was necessary to obtain subjective opinions of persons having knowledge of pavement design and pavement performance. Therefore, ideal persons to be surveyed should include district engineers from highway agencies such as Texas DOT. However, because of constraints in this study, the persons selected included faculty and students in the pavement study area (pavement design and pavement management systems) at the University of Texas at Austin. Twenty persons were surveyed.

Each person was required to complete rating forms that were specially designed for this study. The forms consists of the four attributes as identified for Interstate and secondary roads. Associated with each attribute is a scale on which the rater can mark the level of acceptability. Also included in the survey is a weight factor to capture the relative importance of the attribute with respect to pavement performance.

The rater marked the level considered to be the minimum (or maximum) level of acceptance for each of the four attributes. For example, if the rater thought that the maximum percent of distress tolerable on an Interstate pavement was 20 percent, the rater marked 20 percent. The raters also entered a weight for each of the four attributes to indicate his or her opinion about their relative importance.

From the survey the frequency at each acceptability level for each attribute was determined. The cumulative sum of the number of ratings over the entire rating scale was calculated for each attribute. By dividing cumulative frequency at each acceptability level by the total number of responses per attribute, the degree of acceptability on a 0-to-1 scale was determined. The degree of acceptability was plotted against the attribute scale for each attribute. Nonlinear regression analyses were performed on each of the four attribute plots, and the best-fit function (highest R-squared value) was chosen as the membership function.

RESULTS ANALYSIS

There are two membership functions for each of the four pavement attributes: one for Interstates and the other for secondary roads. The eight membership curves for roughness, distress, structural capacity, and safety and their equations are shown in Figures 2 through 9.

The curves take the general form

$$A = e^{-ax^b}$$

or

$$A = 1 - e^{-ax^b}$$

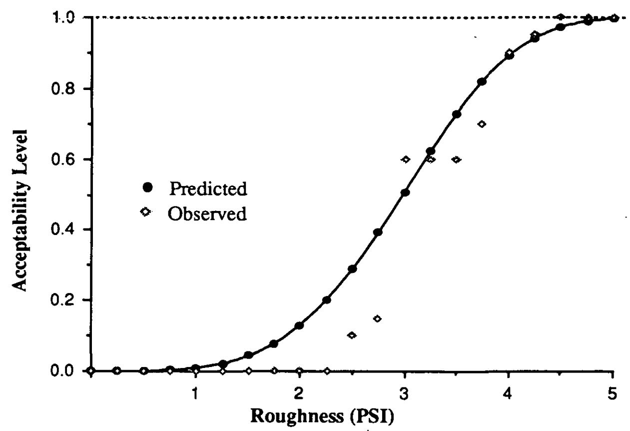

FIGURE 2 Membership curve for pavement roughness, interstate roads.

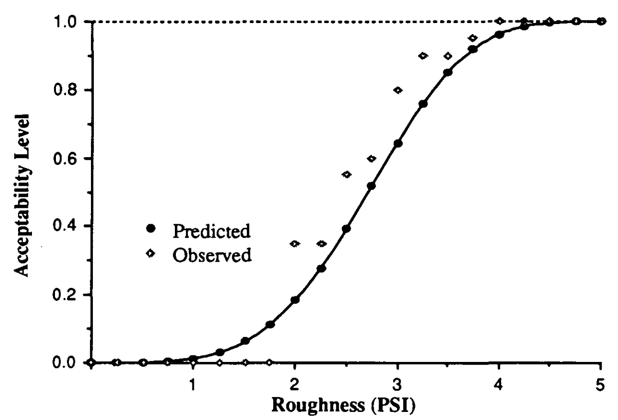

FIGURE 3 Membership curve for pavement roughness, secondary roads.

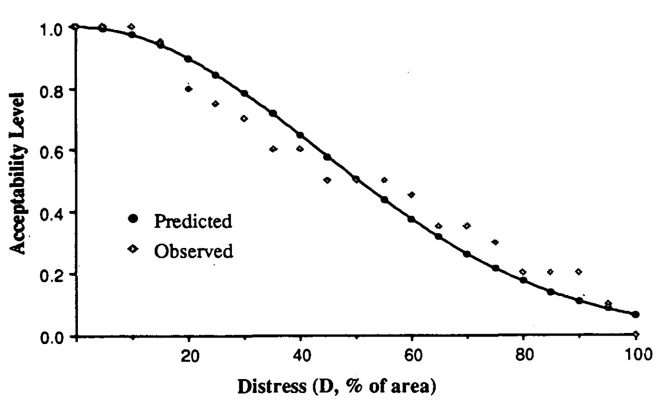

FIGURE 4 Membership curve for distress, interstate roads.

High regression coefficients (R-square) ranging from .955 to .979 were obtained.

Though the curves for roughness, distress, and structural capacity appear to be S-shaped, the curve for skid resistance demonstrates a linear membership transition for both Interstate and secondary roads. For the same attribute value, the degree of acceptance for Interstate roads is normally lower

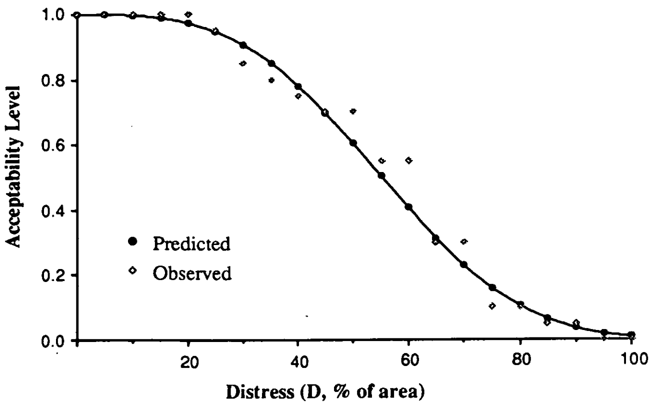

FIGURE 5 Membership curve for distress, secondary roads.

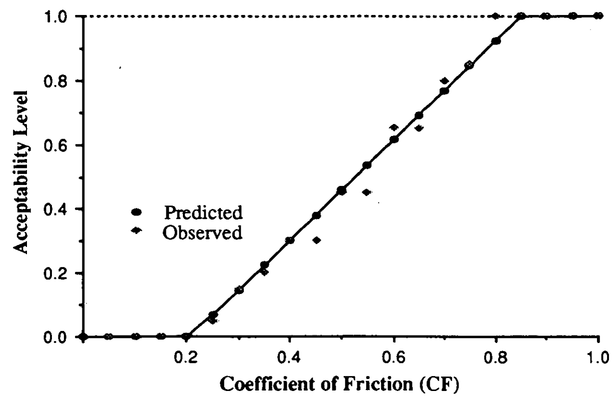

FIGURE 8 Membership curve for skid resistance, Interstate roads.

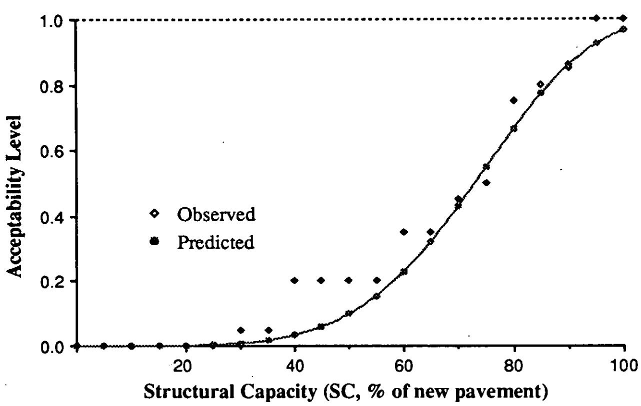

FIGURE 6 Membership curve for structural capacity, Interstate roads.

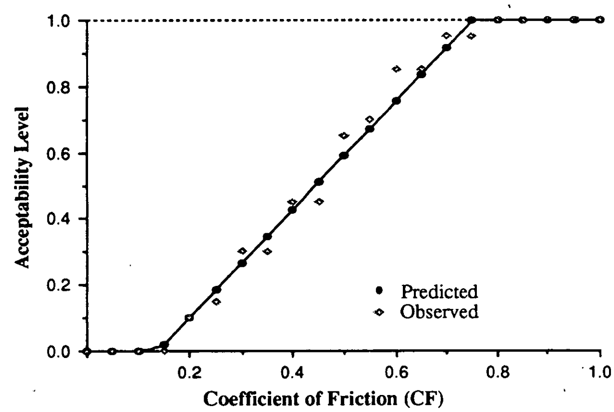

FIGURE 9 Membership curve for skid resistance, secondary roads.

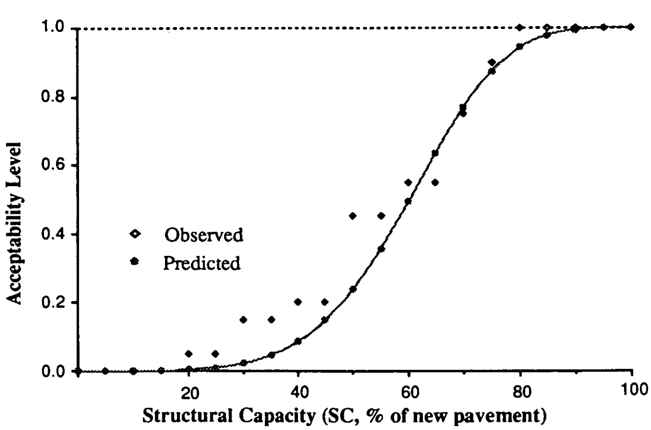

FIGURE 7 Membership curve for structural capacity, secondary roads.

than that for secondary roads. This means that the performance requirements for Interstate highways are higher than those for secondary roads, which is consistent with the real situation.

Figures 10 through 13 show the comparisons of the acceptability for roughness (PSI), distress, structural capacity, and skid resistance between Interstate and secondary roads. The average acceptability is 0.5, which means that 50 percent of the pavement engineers accept the pavement condition at this attribute level. Taking roughness as an example, the PSI value corresponding to an acceptability of 0.5 is 3.0 for Interstate and 2.7 for secondary, as shown in Figure 10. The 50 percent acceptance values for all four attributes for Interstate and secondary roads are presented in Table 1. It can be seen that the expected performance for Interstate is generally higher than that for secondary. Summaries of the membership functions and weights are given in Tables 2 and 3.

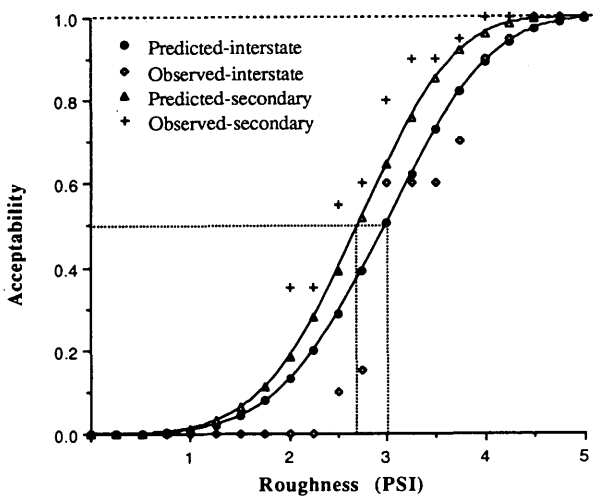

FIGURE 10 Comparison of Interstate and secondary roads for PSI.

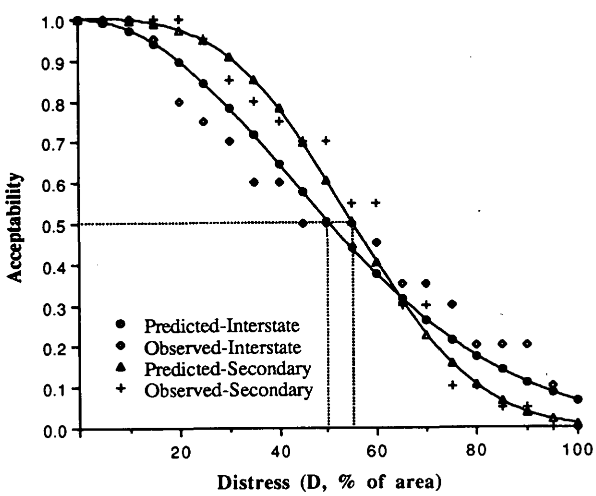

FIGURE 11 Comparison of Interstate and secondary roads for distress.

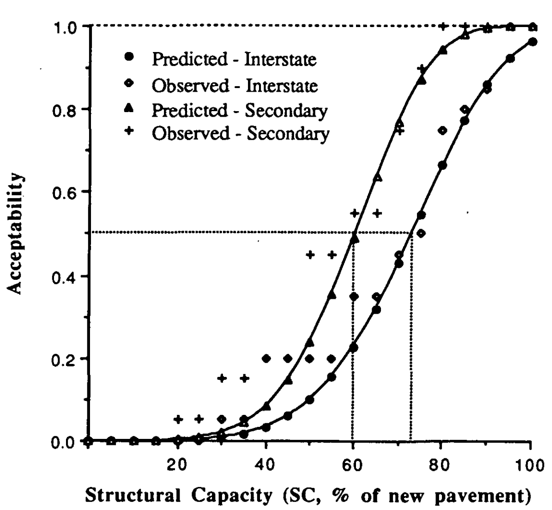

FIGURE 12 Comparison of Interstate and secondary roads for structural capacity.

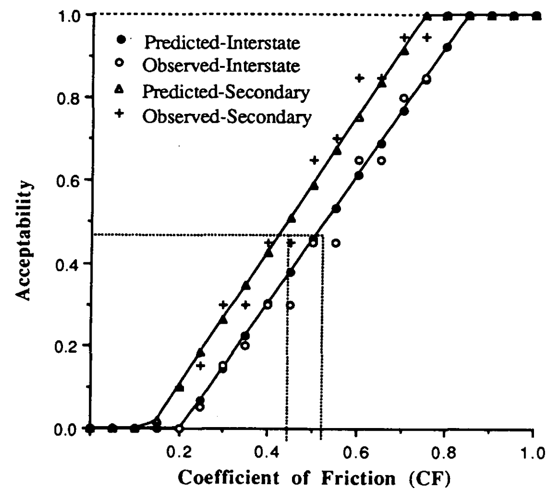

FIGURE 13 Comparison of Interstate and secondary roads for skid resistance.

TABLE 1 Attribute Values for Acceptability of 0.5

Attribute	Interstate	Secondary
Roughness (PSI)	3.0	2.7
Distress (% of area)	50	55
Structural Capacity (% of new)	73	60
Skid Resistance (Coefficient of friction)	0.51	0.45

TABLE 2 Membership Functions and Weights for Interstate Roads

Attribute	Membership Function	R-square	Weight
Roughness	$A = 1\text{-}\exp(-0.008688*PSI^4)$	0.995	0.344
Distress	$A = \exp(-0.000002729*D^2)$	0.965	0.203
Structural Capacity	$A = 1\text{-}\exp(-0.104*(SC/50)^5)$	0.972	0.222
Skid Resistance	$A = -0.32231+1.5582*CF$	0.977	0.236

TABLE 3 Membership Functions and Weights for Secondary Roads

Attribute	Membership Function	R-square	Weight
Roughness	$A = 1\text{-}\exp(-0.01274*PSI^4)$	0.970	0.306
Distress	$A = \exp(-0.00000185*D^2)$	0.971	0.244
Structural Capacity	$A = 1\text{-}\exp(-0.207*(SC/50)^5)$	0.960	0.225
Skid Resistance	$A = -0.2246+1.6308*CF$	0.979	0.231

CONCLUSIONS

Pavement management is an area in which imprecise concepts and subjective knowledge exist. In an attempt to model this knowledge and concepts, fuzzy set theory can be used.

The OAI model using fuzzy set theory combined pavement roughness, distress, structural capacity, and safety as well as their relative importance into a single index that gives a comprehensive evaluation of a pavement. The concept is simple and practical to use.

The membership functions are the basis of the OAI model. The methodology for establishing the membership functions is presented with examples. The procedures can be applied to any other similar problem.

The OAI model is independent of the number of pavement attributes included because the sum of the weighting values is always 1.

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URMS: A Graphical Urban Roadway Management System at Network Level

XIN CHEN, JOSE WEISSMANN, TERRY DOSSEY, AND W. RONALD HUDSON

A graphical urban roadway management system (URMS) is described. The objective of the system is to assist in scheduling maintenance and rehabilitation (M&R) projects at the network level. URMS works in graphics mode and is characterized by simplicity, flexibility, and user-friendliness. In URMS, management sections can be composed of one or more street blocks. Pavement condition index, which is derived from seven types of distress, is the main calculation variable used in the system. Other evaluation indexes include pavement age, mixed average daily traffic, and truck average daily traffic. The assignment of M&R strategy to each section is performed by means of a decision tree. A methodology combining two matrices and an equation is used for project prioritization. Users can change distress types, M&R strategies, and parameters of all the models. The entire system, including the data base and all models and graphics, is written in Turbo Pascal with the Borland Graphics Interface. The system was tested and its functionality demonstrated with the use of data from the city of Austin, Texas.

Pavement management systems (PMSs) have gained popularity in the transportation industry as tools to help managers and engineers make decisions for managing pavements (*1*). Considerable effort is now under way at state and local government levels for developing and implementing PMSs (*2–6*). It has been shown that implementing properly designed and developed PMSs improves not only the efficiency but also the effectiveness of decision making involved in managing pavements (*7,8*).

The successful implementation of a PMS depends mainly on three factors: reliable data, realistic models for processing the data, and user-friendly software for organizing the inputs and presenting the outputs. In general, the more relevant information on pavement condition collected, the better PMS performance will be. Much of the information needed for supporting a complex PMS is costly to collect, particularly for cities in which expensive equipment such as devices for measuring pavement deflection, roughness, and friction are not available. Adopting simple and consistent PMS practices in the initial phase of PMS implementation is recommended for medium-size urban pavement networks where a complex system is not justified (*4*). Unlike pavement thickness design programs, which are based on proven algorithms and scientific facts, a PMS for selecting cost-effective maintenance and rehabilitation (M&R) projects at the network level is very much dependent on local policy and engineering judgment.

Since the development of PMS software is time-consuming and expensive, it is desirable that the resulting software be flexible in such a way that it can be easily tailored to local policies of the agency that will finally use it. Flexible PMS computer programs that allow users to change some of the data items and parameters of models or to select user-defined models are desirable (*8*) and may significantly reduce the cost of developing and implementing PMSs by extending the applicability of the product to many agencies. User-friendly PMS software is also important in the implementation phase. Good PMS programs should be easy to use and easy to learn. The application of graphical user interface technology greatly improves the user-friendliness of PMS software (*6,7*).

Geographic information system (GIS) technology has also been applied to pavement management (*7*); However, because of the high costs and the time and effort to implement it for pavement management (*6*), its applicability is restricted to medium and large cities.

Under the auspices of the Energy Research Application Program sponsored by the state of Texas, research toward the development of a comprehensive urban roadway management system (URMS) is under way. The main objective of the URMS project is to develop a comprehensive PMS for managing urban pavements effectively; the focus is to save energy in terms of roadway user operating costs and pavement M&R costs. The complete system covers M&R planning at the network level and pavement design, construction, and maintenance at the project level.

Described in this paper is the pilot program, the first part of the URMS: M&R scheduling at the network level. The objective of this initial part of the system is to schedule cost-effective M&R projects at the network level. The system is designed to work in graphics mode on any IBM personal computer (or compatible) with a VGA monitor. Figure 1 shows the overall structure of the system. Basically, it is com-

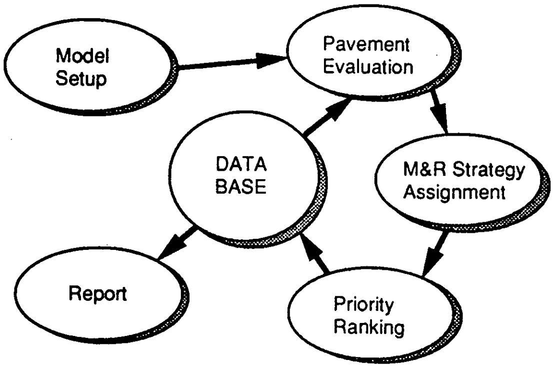


FIGURE 1 Data flow chart.

Center for Transportation Research, ECJ 6.10, University of Texas, Austin, Tex. 78712.

posed of a data base module, a pavement evaluation module, a M&R selection module, and a reporting module. In the URMS, management sections are identified by one or more blocks. Pavement condition index (PCI), which is derived from seven distress types for either flexible or rigid pavements is the main condition variable used in the program. Other condition variables include pavement age (AGE), mixed average daily traffic (MADT), and truck average daily traffic (TADT). A decision tree that takes PCI, AGE, and TADT into account is used for assigning M&R strategy for each section. Two priority ranking matrices and a priority rating equation are combined for M&R project prioritization. The data base and all models and graphics are combined into an integrated program. The system was tested with sample data from the city of Austin, Texas.

DATA BASE MODULE

Thirty-nine data items are used in the subsystem. Some data items can be shared by the design, construction, and maintenance subsystems. Data can be classified into

- Basic data: the minimum required data for running the program,
- Street map data: street map x-y coordinate data, and
- Distress data: percentage of distress in terms of distress type and severity.

Basic data covers section code, street name, location from, location to, pavement type (flexible or rigid pavement), section length, number of traffic lanes, pavement width (total width of traffic lanes), construction year, last major rehabilitation year (medium overlay, thick overlay, or reconstruction), average daily traffic (ADT), traffic growth rate, percentage of trucks, and PCI.

Street map data are optional and distress data can also be optional if PCI is available from an external computer file that has been calculated using some other model defined by the user. The street map data cover pavement section "location from" and "location to" x-y coordinates. The seven default distress types used in the PCI calculations are

- For flexible pavements: alligator cracking, block cracking, longitudinal and transverse cracking, rutting, bleeding/polishing, raveling/pothole, and patching.
- For rigid pavements: linear cracking, D-cracking, polishing, faulting, spalling, corner break, and patching.

Again, these distress types can be changed by users, if desired.

In the URMS computer program, management sections can be one block to several blocks long. The section code consists of a letter and six digits. The letter can be "A" for arterial street, "C" for collect, or "L" for local. The rest of the code consists of street and block sequence numbers that can be defined by the user.

All the information for one management section can be displayed on one screen as shown in Figure 2. The section is

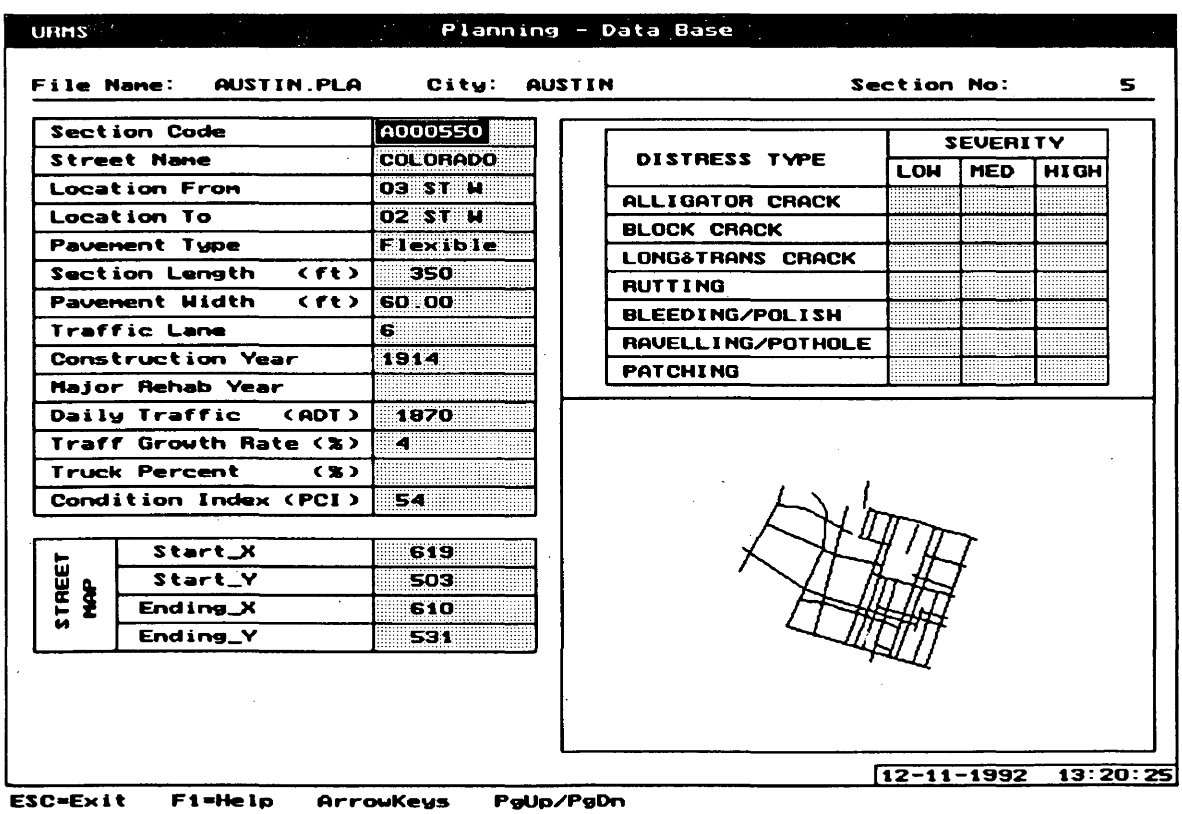


FIGURE 2 Data input and edit screen.

highlighted in the street map in the lower right box. Figure 3 shows 20 sections (records) on one screen (PT = pavement type, LEN = section length in feet, W = pavement width in feet, YEAR = construction year, r = traffic growth rate, $\%T$ = percentage of truck). The bottom box shows PCI and ADT in scale, the numbers being the last two digits of the first column that are used to find the records. The data base handling capabilities integrated into the URMS include many functions such as editing, sorting, and searching.

EVALUATION MODULE

Three types of evaluation index—PCI, pavement age index, and traffic index—are included in the URMS. PCI is a function of pavement distress type, severity, and density. The following equation is used to compute PCI:

$$PCI = 100 - \sum_i \sum_j W_{ij} \times D_{ij} \tag{1}$$

where W_{ij} is the weight of distress type i and severity type j, and D_{ij} is the percentage of area of distress type i and severity type j. Distress weights (range from 0 to 1) reflect the relative contribution of the distress type and severity to PCI. In general, they are determined by engineering judgment. The default values are set for the first use of the system; users can change both the distress types and weights to suit local conditions.

Pavement age is defined as the time from the year of new construction to the year of the distress survey. Because the total service lives of flexible and rigid pavements are quite different, pavement ages for the two types of pavement are calculated separately. All the evaluation indexes are divided into five classes, as shown in Figure 4. The limiting values for all the evaluations shown are default values (MADT and TADT in vehicles per lane per day), which can also be changed by the user.

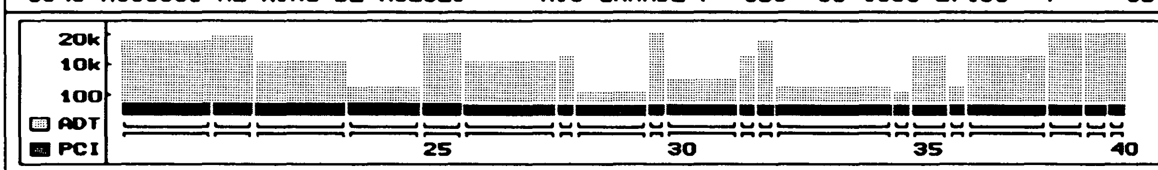
Figure 5 shows the main screen of the output for the evaluation module. The left box presents the section results one by one. Detailed information of each section can also be presented at the same time using a function key. In Figure 5, the two boxes to the right present the summary evaluation results for the whole network in terms of PCI, AGE, MADT, and TADT. The lower right bar chart shows the PCI distribution. A street map that shows the distribution of pavement or traffic condition can also be drawn at this point.

M&R PROGRAM MODULE

In this pilot program, two simple models—M&R strategy assignment and priority ranking model—are combined for selecting M&R projects. First, each section is assigned an M&R strategy by the decision tree model based on the evaluation results. There are two decision trees in the URMS: one for flexible pavements and another for rigid pavements. Figure 6 shows the decision tree for flexible pavements. If

URMS Planning - Browse Data

NO	S. CODE	ST.NAME	FROM	TO	PT	LEN	W	NCY	ADT	r	%T	PCI
0021	A000630	CONGRESS	04 ST	09 ST	F	1750	60	1986	18280	4		72
0022	A000130	05 ST W	LAMAR	WEST AV	F	800	60	1983	19660	4		71
0023	C000281	15 ST E	I 35	SAN JACINT	F	1800	30	1985	11730	4		70
0024	C000890	NUECES ST	15 ST W	ML KING BL	F	1400	38	1980	4400	4		70
0025	A000830	ML KING BL	TRINITY	RED RIVER	F	800	60	1982	30660	4		70
0026	C000280	15 ST E	SAN JACINT	I 35	F	1800	30	1985	11730	4		69
0027	C000200	11 ST E	SAN JACINT	TRINITY	F	350	41	1982	13020	4		69
0028	C000930	RIO GRANDE	15 ST W	ML KING BL	F	1400	40	1980	2660	4		69
0029	A000050	01 ST W	LAVACA	COLORADO	F	350	60	1984	20020	4		69
0030	A000650	CONGRESS	15 ST	ML KING BL	F	1400	42	1980	6500	4		68
0031	C000180	11 ST E	CONGRESS	BRAZOS	F	350	60	1982	13020	4		67
0032	A000110	05 ST W	COLORADO	CONGRESS	F	350	57	1983	18370	4		66
0033	C000250	12 ST W	LAVACA	SHOAL CREE	F	2250	22	1978	4625	4		66
0034	A000570	COLORADO	05 ST W	04 ST W	F	350	60	1978	3470	4		65
0035	C000210	11 ST E	TRINITY	RED RIVER	F	700	44	1983	13420	4		65
0036	A000580	COLORADO	06 ST W	05 ST W	F	350	60	1976	4830	4		65
0037	C000291	15 ST W	RIO GRANDE	LAVACA	F	1540	30	1985	13975	4		65
0038	A000850	ML KING BL	CONGRESS	LAVACA	F	700	60	1982	27130	4		65
0039	A000870	ML KING BL	GUADALUPE	NUECES	F	450	60	1982	27130	4		65
0040	A000880	ML KING BL	NUECES	RIO GRANDE	F	350	60	1980	27130	4		65



ESC=Menu F1=Help Arrow_Key=Move F3=DataSheet F5=Sort F8=Search PgUp/PgDn

FIGURE 3 Browse data screen.

URMS — Planning - Model Setup

Page 7-3 Evaluation Criteria

PROGRAM YEAR 1992

PAVEMENT CONDITION INDEX (PCI)

	1 Bad	2 Poor	3 Fair	4 Good	5 Excell
PCI	< 30	30 - 50	50 - 70	70 - 90	>= 90

PAVEMENT AGE (AGE)

	1 V. Old	2 Old	3 Average	4 New	5 V.New
Flexible	> 20	20 - 15	15 - 10	10 - 5	<= 5
Rigid	> 40	40 - 30	30 - 20	20 - 10	<= 10

TRAFFIC CLASSIFICATION

ADT(x1000)	1 V.Heavy	2 Heavy	3 Medium	4 Light	5 V.Light
1. Mixed	> 4.0	4.0 - 3.0	3.0 - 2.0	2.0 - 1.0	<= 1.0
2. Truck	> 0.4	0.4 - 0.3	0.3 - 0.2	0.2 - 0.1	<= 0.1

ESC=Exit F2=Save PgUp/PgDn

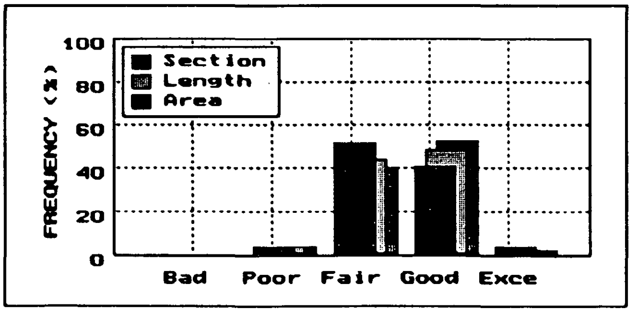
FIGURE 4 Evaluation criteria screen.

URMS — Planning - Evaluation

NO	S. CODE	PT	PCI	AGE	MT	TT
0026	C000150	F	3	3	3	3
0027	C000190	F	3	3	3	3
0028	C000180	F	3	3	3	3
0029	C000230	F	3	3	3	3
0030	A000040	F	3	4	2	2
0031	A000050	F	3	4	2	2
0032	A000100	F	4	3	3	3
0033	A000270	F	4	3	3	3
0034	A000780	F	4	3	3	3
0035	C000280	F	3	4	2	2
0036	A000650	F	3	3	4	4
0037	A000030	F	4	4	1	1
0038	C000240	F	3	3	4	4
0039	A000800	F	3	4	3	3
0040	A000770	F	4	3	4	4
0041	A000010	F	4	4	2	2
0042	C000890	F	4	3	4	4
0043	A000630	F	4	4	2	2
0044	A000020	F	4	4	2	2
0045	A000060	F	4	4	2	2
0046	A000070	F	4	4	2	2
0047	C000930	F	3	3	5	5
0048	C000281	F	4	4	2	2
0049	A000810	F	3	4	4	4
0050	A000790	F	4	4	3	3

NETWORK CONDITION DISTRIBUTION

CLASS	SECTION	LENGTH (mile)	AREA (k-SY)
Bad	0	0.0	0.0
Poor	2	0.4	10.9
Fair	29	4.3	118.0
Good	23	4.9	154.4
Exce	2	0.3	9.3
Total	56	9.9	292.6



PCI AGE MADT TADT

ESC=Menu F1=Help F3=DataSheet F5=Sort F8=Search F10=Map PgUp/PgDn

FIGURE 5 Evaluation main screen.

the total required M&R cost is greater than the available budget, prioritization is performed. In the decision tree, PCI, AGE, and TADT are taken into account. Up to 18 types of M&R strategies can be defined by the user. The default types for flexible pavements are

- Do nothing,
- Routine maintenance,
- Thin overlay,
- Medium overlay,
- Thick overlay, and
- Reconstruction.

For the sake of simplicity, the five classes of each variable are further combined into two or three groups. Users can group them by changing the variable codes, as illustrated in Figure 6.

The prioritization procedure can be conducted using one or more variables. Basically, there are two ways to construct a priority ranking model if multiple variables are to be considered: the matrix method and the equation method. A more flexible way, which combines two matrices and an equation for computing the priority index (PIX), is presented in Figure 7. As shown in the figure, PIX is a function of the PCI, pavement age, mixed traffic, and street class. Any of the four variables can be ignored by setting one or more of the parameters to 0. For example, street class and traffic variables can be eliminated by changing the weight of 30 to 0 in the equation. Street class will also not be taken into account if each row number is the same in the right matrix. By analyzing the information in Figure 7, it can be implied that the smaller the PIX, the higher the priority for that section.

The URMS currently determines an annual M&R program. It can be improved to determine multiyear M&R program with the inclusion of pavement deterioration models. It currently can approximate M&R programs for up to 5 years on the basis of the PIX approach as discussed. The basic idea of the approximation is that sections of higher priority will be scheduled for M&R earlier than those of lower priority. If some noncontiguous short sections are selected by the program, these sections can be combined manually.

The main output screen for the M&R module is shown in the background of Figure 8. In Figure 8 the last four columns present the basic outputs M&R strategy (*S*), PIX, recommended action year (RAY), and M&R cost in thousands of dollars, for each section. Figure 8 also presents the summary information of the recommended M&R program that covers the total M&R needs, including the recommended number of M&R sections and required M&R budgets. The M&R information for each section can also be summarized in bar charts and presented in a street map with different colors.

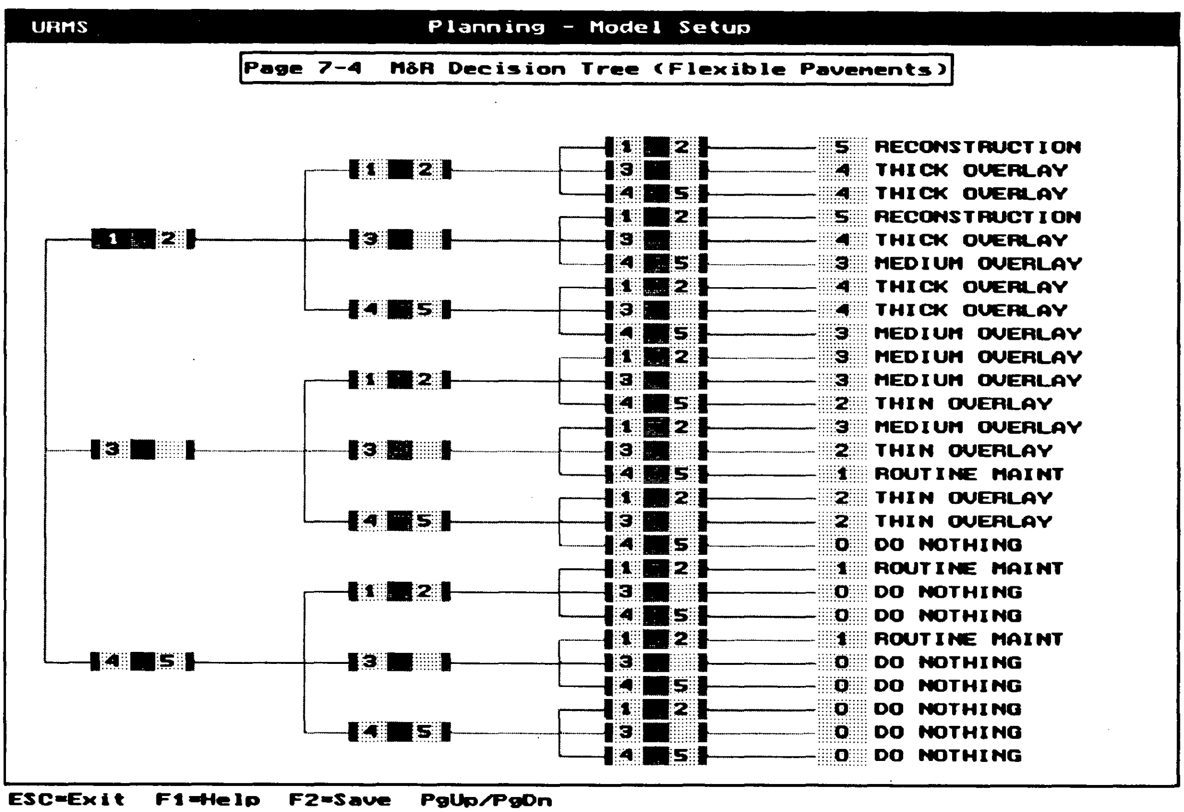


FIGURE 6 M&R strategy assignment decision tree screen.

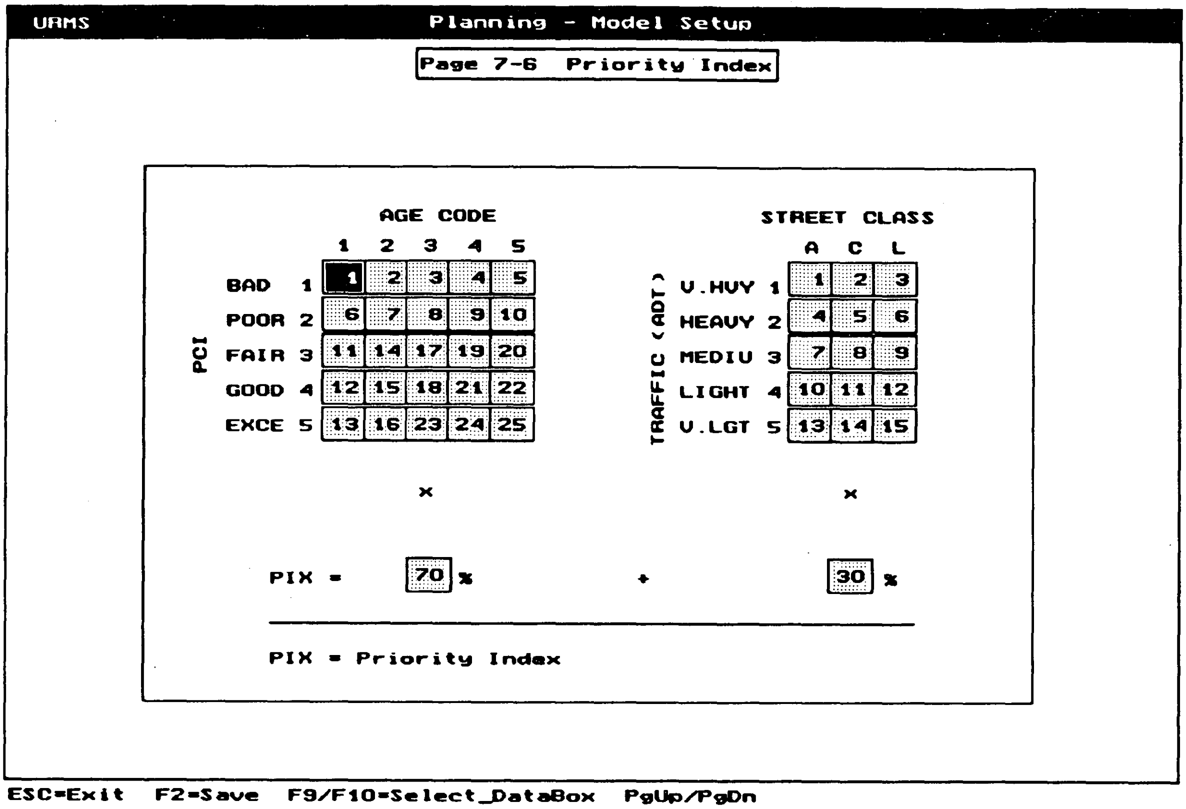


FIGURE 7 PIX screen.

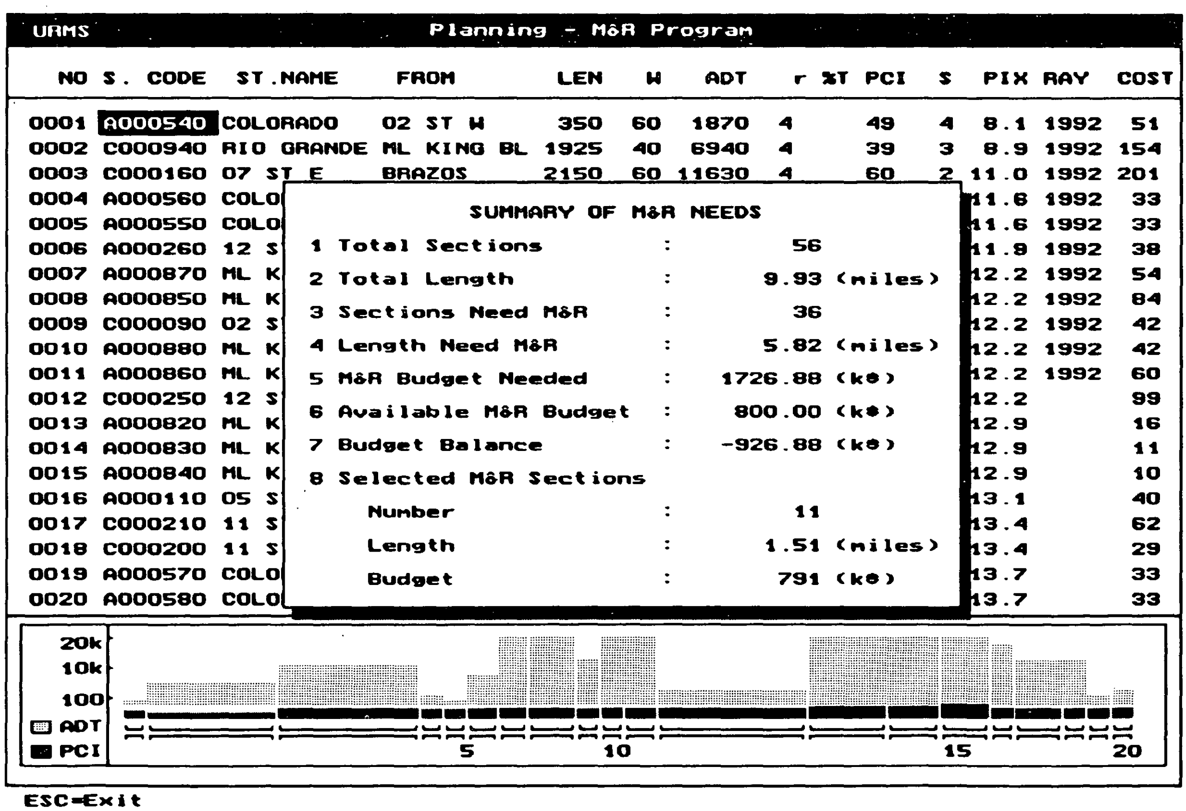


FIGURE 8 M&R program screen.

REPORT MODULE

URMS can generate seven types of report: four types are listings, and three types are summaries. Listing reports include

1. Basic input and output information,
2. Recommended M&R projects,
3. Pavement distress data, and
4. Street map *x-y* coordinates.

Summary reports include

1. Street functional classes and pavement types,
2. Pavement condition and traffic evaluation, and
3. M&R needs and recommended M&R projects.

Figures 9 through 11 present three sample reports. Basic input and output information for 35 sections are listed in Figure 9. Figure 10 presents the summary evaluation infor-

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LISTING OF BASIC INPUT AND OUTPUT INFORMATION

Input File: AUSTIN.PLA Report Date: 12-11-1992 Page: 1

NO	SECTION CODE	STREET NAME	LOCATION FORM	LOCATION TO	TYPE	LENGTH (ft)	WIDTH ft	TRAF LN	CONSTRU YEAR	LAST REHAB YR	TRAFFIC	ADT	RATE %	TRUC %	PCI	M&R	P INDEX	ACTION YEAR	M&R COST K$
00001	A000540	COLORADO	02 ST W	01 ST W	F	350	60	6	1914		1870		4		49	4	8.1	1992	51.3
00002	C000940	RIO GRANDE	ML KING BL	24 ST W	F	1925	40	4	1980		6940		4		39	3	8.9	1992	154.0
00003	C000160	07 ST E	BRAZOS	I 35	F	2150	60	6	1928		11630		4		60	2	11.0	1992	200.7
00004	A000560	COLORADO	04 ST W	03 ST W	F	350	60	6	1914		3470		4		58	2	11.6	1992	32.7
00005	A000550	COLORADO	03 ST W	02 ST W	F	350	60	6	1914		1870		4		54	2	11.6	1992	32.7
00006	A000260	12 ST W	SHOAL CREE	LAMAR	F	475	40	4	1978		9250		4		63	3	11.9	1992	38.0
00007	A000850	ML KING BL	CONGRESS	LAVACA	F	700	60	6	1982		27130		4		65	3	12.2	1992	84.0
00008	A000880	ML KING BL	NUECES	RIO GRANDE	F	350	60	6	1980		27130		4		65	3	12.2	1992	42.0
00009	C000090	02 ST E	BRAZOS	CONGRESS	F	350	60	6	1978		13860		4		50	3	12.2	1992	42.0
00010	A000860	ML KING BL	LAVACA	GUADALUPE	F	500	60	6	1982		27130		4		60	3	12.2	1992	60.0
00011	A000870	ML KING BL	GUADALUPE	NUECES	F	450	60	6	1982		27130		4		65	3	12.2	1992	54.0
00012	C000250	12 ST W	LAVACA	SHOAL CREE	F	2250	22	2	1978		4625		4		66	3	12.2		99.0
00013	A000820	ML KING BL	CONGRESS	TRINITY	F	1200	60	6	1982		30660		4		74	1	12.9		16.0
00014	A000840	ML KING BL	RED RIVER	I 35	F	750	60	6	1982		30730		4		88	1	12.9		10.0
00015	A000830	ML KING BL	TRINITY	RED RIVER	F	800	60	6	1982		30660		4		70	1	12.9		10.7
00016	A000110	05 ST W	COLORADO	CONGRESS	F	350	57	6	1983		18370		4		66	3	13.1		39.9
00017	C000210	11 ST E	TRINITY	RED RIVER	F	700	44	4	1983		13420		4		65	3	13.4		61.6
00018	C000200	11 ST E	SAN JACINT	TRINITY	F	350	41	4	1982		13020		4		69	3	13.4		28.7
00019	A000580	COLORADO	06 ST W	05 ST W	F	350	60	6	1976		4830		4		65	2	13.7		32.7
00020	A000570	COLORADO	05 ST W	04 ST W	F	350	60	6	1978		3470		4		65	2	13.7		32.7
00021	A000130	05 ST W	LAMAR	WEST AV	F	800	60	6	1983		19660		4		71	1	13.8		10.7
00022	A000120	05 ST W	WEST AV	COLORADO	F	2650	56	6	1983		18370		4		75	1	13.8		33.0
00023	C000290	15 ST W	LAVACA	RIO GRANDE	F	1540	30	3	1985		13975		4		63	2	13.9		71.9
00024	C000291	15 ST W	RIO GRANDE	LAVACA	F	1540	30	3	1985		13975		4		65	2	13.9		71.9
00025	C000220	11 ST E	RED RIVER	I 35	F	700	44	4	1982		13420		4		77	1	14.1		6.8
00026	C000190	11 ST E	BRAZOS	SAN JACINT	F	350	60	6	1982		13020		4		60	2	14.3		32.7
00027	C000230	11 ST W	CONGRESS	COLORADO	F	350	60	6	1982		12030		4		61	2	14.3		32.7
00028	C000150	07 ST E	CONGRESS	BRAZOS	F	350	55	5	1983		11220		4		61	2	14.3		29.9
00029	C000180	11 ST E	CONGRESS	BRAZOS	F	350	60	6	1982		13020		4		67	2	14.3		32.7
00030	A000040	01 ST W	COLORADO	CONGRESS	F	350	60	6	1984		19800		4		64	2	14.5		32.7
00031	A000050	01 ST W	LAVACA	COLORADO	F	350	60	6	1984		20020		4		69	2	14.5		32.7
00032	A000780	LAVACA ST	04 ST W	11 ST W	F	2450	60	6	1980		16790		4		77	0	14.7		
00033	A000270	12 ST W	LAMAR	WEST LYNN	F	2750	44	4	1980		9250		4		75	0	14.7		
00034	A000100	05 ST E	SAN JACINT	CONGRESS	F	700	57	6	1983		14180		4		80	0	14.7		
00035	C000280	15 ST E	SAN JACINT	I 35	F	1800	30	3	1985		11730		4		69	2	14.8		84.0

Pavement Type: F = Flexible Pavement R = Rigid Pavement
Flexible Pavement M&R Strategy
0=DO NOTHING 1=ROUTINE MAINT 2=THIN OVERLAY 3=MEDIUM OVERLAY 4=THICK OVERLAY
5=RECONSTRUCTION
Rigid Pavement M&R Strategy
0=DO NOTHING 1=ROUTINE MAINT 2=THIN AC OVERLAY 3=MEDIUM AC OVERLAY 4=THICK AC OVERLAY

City: AUSTIN User: University of Texas Analyst: Chen

FIGURE.9 Sample listing printout.

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SUMMARY OF PAVEMENT CONDITION AND TRAFFIC EVALUATION (1991)

Input File: AUSTIN.PLA Report Date: 12-11-1993

CONDITION CODE	CLASS DESCRIPTION	LIMITING VALUE	SECTION NUMBER	%	LENGTH MILES	%	AREA 1000 SY	%
* PCI								
1	Bad	<= 30	0	0.0	0.0	0.0	0.0	0.0
2	Poor	30 - 50	2	3.6	0.4	4.3	10.9	3.7
3	Fair	50 - 70	29	51.8	4.3	43.8	118.0	40.3
4	Good	70 - 90	23	41.1	4.9	49.2	154.4	52.8
5	Exce	> 90	2	3.6	0.3	2.7	9.3	3.2
* AGE								
1	V.Old	> 20(40)	4	7.1	0.6	6.1	21.3	7.3
2	Old	15(30) - 20(40)	5	8.9	0.7	7.2	14.6	5.0
3	Fair	10(20) - 15(30)	26	46.4	4.8	48.6	146.4	50.0
4	New	5(10) - 10(20)	21	37.5	3.8	38.1	110.3	37.7
5	V.New	<= 5(10)	0	0.0	0.0	0.0	0.0	0.0
* MADT								
1	V.Hvy	> 4000	10	17.9	1.6	16.4	46.1	15.8
2	Heavy	3000 - 4000	17	30.4	3.1	30.7	91.0	31.1
3	Mediu	2000 - 3000	12	21.4	2.2	22.4	62.6	21.4
4	Light	1000 - 2000	9	16.1	2.1	21.1	63.3	21.6
5	V.Lgt	<= 1000	8	14.3	0.9	9.3	29.6	10.1
* TADT								
1	V.Hvy	> 400	10	17.9	1.6	16.4	46.1	15.8
2	Heavy	300 - 400	17	30.4	3.1	30.7	91.0	31.1
3	Mediu	200 - 300	12	21.4	2.2	22.4	62.6	21.4
4	Light	100 - 200	9	16.1	2.1	21.1	63.3	21.6
5	V.Lgt	<= 100	8	14.3	0.9	9.3	29.6	10.1
	TOTAL		56	100.0	9.9	100.0	292.6	100.0

City: Demonstration User: University of Texas Analyst: Chen

FIGURE 10 Pavement evaluation summary printout.

mation of pavement condition and traffic. Two types of M&R summary are given in Figure 11. One presents the summary of M&R needs and another shows the recommended M&R sections for the analysis period. In this example 36 flexible pavement sections require maintenance or rehabilitation at a cost of $1.73 million; but only $0.8 million is available. Because of the shortage of funds, only 11 pavement sections are selected for maintenance or rehabilitation out of the 36 candidate sections.

CONCLUSIONS

A graphical URMS was described in this paper. The system was written in Turbo Pascal and is designed for scheduling cost-effective M&R projects at the network level. The functionality of the system was tested with sample data from Austin, Texas. The system is characterized by

- Simplicity: the system uses reduced pavement data, all basic data can be collected easily. It includes simple models that can be easily understood and used.
- Flexibility: users can change some of the data items and all the model parameters.
- User-friendliness: all the input and output are conveniently organized through the use of a graphical interface. Online help is provided and the system is easy to learn and use.

ACKNOWLEDGMENTS

This work was sponsored by the state of Texas. The authors are grateful to the city of Austin for supplying actual data to test the program. Appreciation is also extended to all members of the project advisory committee for their valuable suggestions and recommendations.

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SUMMARY OF MAINTENANCE & REHABILITATION PROGRAM
Flexible Pavements

1. Maintenance & Rehabilitation Needs

Input File: AUSTIN.PLA Report Date: 12-11-1992

M&R STRATEGY Code	Description	UNIT COST ($/SY)	SECTION Number	%	LENGTH (mile)	%	BUDGET $1000	%
0	DO NOTHING	0.00	20	35.7	4.11	41.4	0.00	0.0
1	ROUTINE MAINT	2.00	9	16.1	1.97	19.8	122.00	7.1
2	THIN OVERLAY	14.00	15	26.8	2.19	22.1	850.34	49.2
3	MEDIUM OVERLAY	18.00	11	19.6	1.59	16.0	703.20	40.7
4	THICK OVERLAY	22.00	1	1.8	0.07	0.7	51.33	3.0
5	RECONSTRUCTION	45.00	0	0.0	0.00	0.0	0.00	0.0
	TOTAL		56	100.0	9.93	100.0	1726.9	100.0

2. Recommended M & R projects for 1992

Input File: AUSTIN.PLA Report Date: 12-11-1992

M&R STRATEGY Code	Description	UNIT COST ($/SY)	SECTION Number	%	LENGTH (mile)	%	BUDGET $1000	%
0	DO NOTHING	0.00	45	80.4	8.43	84.8	0.00	0.0
1	ROUTINE MAINT	2.00	0	0.0	0.00	0.0	0.00	0.0
2	THIN OVERLAY	14.00	3	5.4	0.54	5.4	266.00	33.6
3	MEDIUM OVERLAY	18.00	7	12.5	0.90	9.1	474.00	59.9
4	THICK OVERLAY	22.00	1	1.8	0.07	0.7	51.33	6.5
5	RECONSTRUCTION	45.00	0	0.0	0.00	0.0	0.00	0.0
	TOTAL		56	100.0	9.93	100.0	791.3	100.0

City: AUSTIN User: University of Texas Analyst: Chen

FIGURE 11 M&R program summary printout.

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Condition-Based Treatment Recommendation for Project-Level Pavement Management

D. A. GRIVAS AND B. C. SCHULTZ

A methodology is presented for developing preliminary treatment recommendations for candidate pavement projects. Emphasis is placed on the efficient use of available pavement management data. The condition-based evaluation procedure is structured into two subproblems, depending on the complexity of pavement condition. Projects exhibiting simple surface distress undergo only initial screening. Projects with complex condition are identified by the initial screening process and then further evaluated in a detailed analysis. The initial screening uses a matching between distresses, treatments, and treatment classes to analyze projects. The detailed analysis explores both surface distress and nondistress characteristics such as traffic loadings and deterioration rate to generate recommendations about a scope of work. The methodology has been implemented on the New York State Thruway pavement system. The generated results and their validation are presented and discussed. It is concluded that the treatment recommendation methodology is a viable technique that will be further developed for use in project-level pavement management. Results of the analysis support future work in the areas of life-cycle cost analysis, multiyear planning, and program optimization.

One of the primary objectives of project-level pavement management is to generate a prioritized annual needs list. Such condition-based needs assessment facilitates consistent planning, programming, and resource allocation. A state-of-the-art pavement management system (PMS) uses five methodologies to achieve this goal: (a) condition assessment, (b) project determination, (c) treatment recommendation, (d) cost estimation, and (e) project priority ranking.

This paper describes a treatment recommendation methodology that is applied at a level of detail appropriate for pavement management. It is part of the PMS of the New York State Thruway Authority (NYSTA). The methodology combines matrix and decision-tree methods in a staged approach that increases analysis complexity for projects with more complicated conditions. For each project, the objectives are to (a) identify specific treatments required, (b) suggest the scope of work for implementing treatments, and (c) generate feasible alternatives for use in network-level analysis. For all projects, treatments appropriate to pavement condition are determined on the basis of previous maintenance and rehabilitation experience.

The sequence of major tasks in the treatment recommendation methodology is shown in Figure 1. The initial input for each project consists of pavement condition expressed in terms of distress ratings. Surface distress is assessed in terms of type, severity, and extent, as documented by Grivas et al. (*1*). All projects are subject to an initial screening, which matches pavement condition to appropriate treatments, treatment classes, and triggers, and generates an itemized list of treatments to address the existing distresses. A preliminary classification based on these results identifies each project in terms of its scope of work, namely, do nothing, preventive, corrective, or rehabilitation. Pavement projects with relatively simple needs are recommended for either a do-nothing or preventive scope of work. As shown in Figure 2, the initial screening process completes the analysis of scope of work for these projects.

The detailed analysis uses additional data (such as accident rates, deterioration rates, traffic characteristics, pavement age, etc.) and an enhanced decision-making process to further refine the scope of work for projects with complex condition. Corrective candidates undergo resurfacing evaluation to establish whether resurfacing should be performed in addition to the suggested corrective treatments. Rehabilitation candidates undergo rehabilitation evaluation to examine whether resurfacing, rehabilitation, or reconstruction is the appropriate scope of work.

Once the recommended scope of work is identified, alternatives can be generated. An alternative consists of itemized treatments and a scope of work. The recommended scope of work and the itemized standard treatments are designated as the preferred alternative for the unconstrained problem. A project treatment recommendation consists of all feasible alternatives and their associated cost estimates; the preferred alternative is noted for consideration by network-level analysis.

INITIAL SCREENING

The initial screening process generates a preliminary scope of work based on a tally of properties associated with the distress states (distress type–severity–extent combinations) present on a project. Each distress state is associated with its properties through rules generated with the aid of maintenance personnel, on the basis of their experience with local conditions, past maintenance practices, and treatment performance. Five properties were derived for each distress state: (a) treatment class, (b) standard treatment, (c) quick-fix treatment, (d) resurfacing trigger, and (e) drainage trigger. Figure

Civil Engineering Department, Rensselaer Polytechnic Institute, Troy, N.Y. 12180.

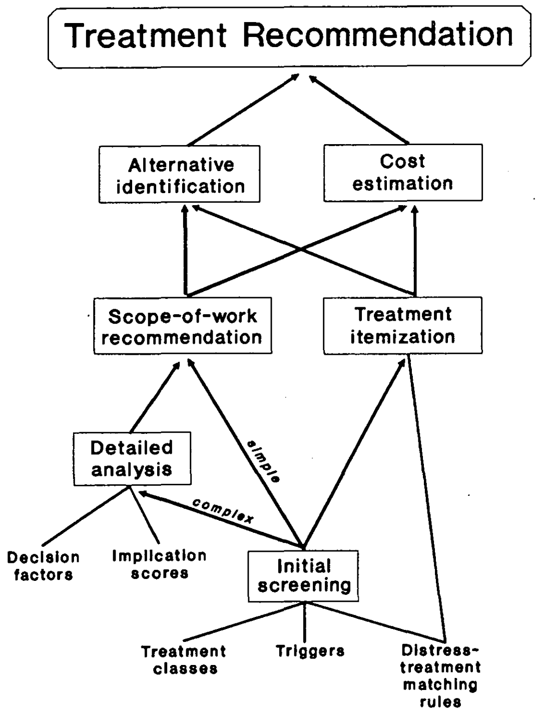

FIGURE 1 Sequence of major tasks in treatment recommendation.

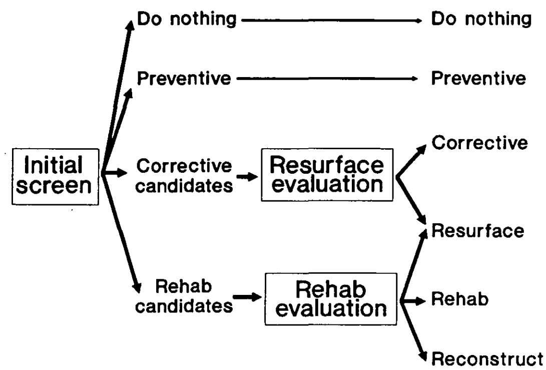

FIGURE 2 Determining recommended scope of work.

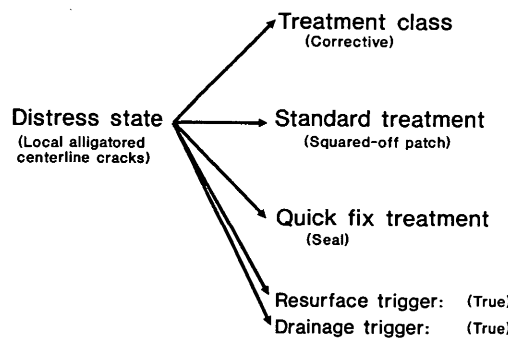

FIGURE 3 Distress-treatment matching example.

3 illustrates an example of the manner in which each distress state is associated with its properties.

Treatment classes were derived to indicate the degree of action required to address the condition associated with each distress state (the classes are do nothing, preventive, corrective, and rehabilitation evaluation). Standard treatments are those that maintenance personnel find to be most effective at mitigating further deterioration of the pavement structure. Quick-fix treatments are the recognized deferral action when the standard treatments cannot be implemented.

Triggers were developed when participants observed that certain distress states are indicative of drainage problems. Because the need for drainage work cannot be easily determined through distress-treatment matching, the drainage trigger concept was introduced to flag projects for further investigation on the need for drainage work. The resurfacing trigger was incorporated to facilitate evaluation of distress combinations that require an overlay, even when no single distress requires that treatment. Thus, each distress state is associated with two triggers: one that indicates that there may be a drainage problem, and one that indicates that the project may need to be overlaid. The triggers are Boolean in that they take only the values true and false. The value for each type of trigger for each distress state is defined by a rule generated with the aid of maintenance experts.

The matrix of all distress states and their associated properties is the basis of the initial screening process. The properties of a project are determined by first matching each distress state on a given project to its corresponding properties and then tallying the properties of all distress states at all locations on the project. The preliminary classification of scope of work is based on the percentages of all distress states in the project that require various classes of treatments and that have true resurfacing triggers. The process followed in preliminary classification was initially defined on the basis of natural breaks in the distribution of this data for 1989 projects. These decision points will be reviewed as implementation continues.

For projects with relatively simple needs, the preliminary classification completes the project scope recommendation process. The suggested scope of work is either do nothing or preventive, depending on the distribution of treatment classes determined from distress ratings. Projects with more complex conditions are classified as either corrective candidates or rehabilitation candidates and advanced to a detailed analysis.

DETAILED ANALYSIS

The resurfacing and rehabilitation evaluation procedures, which compose the detailed analysis, are similar in concept. They suggest an appropriate scope of work in a timely manner, without the requirements of lengthy analysis. Both use implication scores to refine the preliminary classification of scope of work. The scoring process aims to model the interaction between factors that affect decision making. Those projects eventually identified as rehabilitation or reconstruction become candidates for more in-depth analysis that is outside the scope of the current study (e.g., life-cycle cost analysis, pavement design characteristics).

Data Requirement

Factors such as climate, pavement design, deterioration rate, drainage, lane condition, safety, shoulder condition, and traffic may contribute to decisions for resurfacing or rehabilitation. However, only those factors that can be measured or estimated reliably should be incorporated in project-level analysis. The underlying requirements are that the decision process should be tailored to the information available and that data satisfy standards of objectivity and integrity. The primary requisites for an initial implementation are that data be available, accessible, and appropriate.

Available data may be of varying integrity depending on the information source. Project-level analysis and other pavement management activities can be effective only when the information that supports decision making is accessible in a timely manner for all projects under evaluation. This is the primary motivation for the requirement of a data base in a pavement management system. Once the requirements of availability and accessibility have been met, it is judicious to subject the remaining data sources to a review of their appropriateness for measuring the desired factor.

Factor Assessment

Table 1 summarizes decision factors that can be measured for use in the current study and indicates how data are obtained. Data are available in a variety of forms, such as alphanumeric, numeric, and Boolean. In some cases, several data items are combined into a single index.

Functional Adequacy

Functional adequacy represents the quality of the pavement surface in terms of its ability to provide a comfortable (smooth) ride to the user. For the preliminary implementation, it is derived as a function of three quantities: a subjective ride quality assessment, a ride index derived from distress ratings, and patching (expressed in terms of severity and extent). Ride quality and patching assessments are collected through questionnaires to local field personnel, who provide evaluations based on their daily experience with the candidate projects. The ride index is calculated on a scale of 0 to 100, with 100 representing the least ride disruption. The index is determined like a composite distress index, but with weighting factors adjusted to reflect ride disruption.

Because no available measure is known to be a complete and unbiased descriptor of functional adequacy, ride quality, patching, and the ride index were combined into a functional adequacy index. Each of the measures was converted to a similar scale (three point, increasing severity), and weighted averages of the scaled measures were taken.

The values of functional adequacy index range from 0.0 to 3.0. Values greater than 1.75 are defined as indicative of functional inadequacy, for the purposes of the current study. In the future, functional adequacy may be derived from direct, objective roughness measures, such as the international roughness index (*2*).

Structural Adequacy

Structural adequacy is indicated by the degree of load-related distresses that are present throughout the project. A structural

TABLE 1 Decision Factors for Rehabilitation Evaluation

Factor	Measure	Data Source	Type of Measurement
Functional adequacy	ride quality patching severity/extent ride-affecting distress	questionnaire questionnaire distress survey	alphanumeric alphanumeric numeric
Structural adequacy	load-related distress	distress survey	numeric
Deterioration rate	maintenance effort	questionnaire	alphanumeric
Traffic loads	AADT for truck classes	traffic data	numeric
Pavement safety	accident rate rutting	police rpts, traffic data distress survey	numeric numeric
Shoulder condition	shoulder distress	distress survey	numeric
Drainage problems	problem locations	questionnaire	numeric
Pavement history	surface age	questionnaire	numeric
Appurtenance safety	deficient guiderail	guiderail survey	numeric
Traffic control	restricted work hours	agency policy	boolean

adequacy index is calculated similarly to the ride disruption index but with weighting factors adjusted to reflect structural damage. For the purposes of the current study, structural adequacy index values greater than 50.0 are interpreted as structural inadequacy.

Deterioration Rate

In the initial implementation, deterioration rate is inferred directly from field personnel reports of the relative amount of maintenance effort spent on each project. A low rate is assumed if the project requires only scheduled preventive maintenance. A normal rate is assumed if the project requires occasional work in addition to scheduled preventive maintenance. A high rate is assumed if the project requires considerable maintenance work. In the future, deterioration rates may be determined from historical progressions of distress data.

Traffic Loads

Traffic load assessments are based on the average number of trucks that traverse a candidate project each day. In the current study, trucks are defined as all vehicles receiving toll tickets of a certain class; actual vehicle weights are unknown. More than 2,500 trucks a day is considered a high traffic load. At those locations where counts are not available, local personnel estimate whether truck traffic is high.

Pavement Safety

Safety is divided into two components: pavement and appurtenance. Pavement safety represents the degree of hazard due to pavement surface deficiencies. On overlaid pavements, pavement safety is determined as a function of the accident rate and average rut distress rating. Rutting is a potential hazard due to the likelihood for hydroplaning caused by water pooled in wheel track depressions. Pavement-related accident rates are considered a good indication of pavement safety deficiencies. However, they do not generally correspond to rutting as recorded by the distress survey. Several formulations were investigated to combine accident rates and rutting into a pavement safety index, which is used in rehabilitation evaluation.

Accident rates are reported as the number of accidents not related to alcohol, drugs, or animal per 100,000 vehicle-mi traveled on the project. The average rut distress rating is taken as the arithmetic mean of integer-mapped rut ratings determined by the distress survey. Each of these measures was converted to a similar scale (three point, increasing severity), and weighted averages of the scaled measures were taken.

The values of pavement safety index determined by this method range between 0.0 and 3.0. Values greater than 0.85 were defined as indicative of a high pavement safety deficit. This value corresponds approximately to an accident rate of 25 per 100,000 vehicle-mi traveled.

Shoulder Condition

Shoulder condition was assessed using a composite shoulder distress index, obtained by a weighted combination of shoulder distress ratings. Values less than or equal to 50.0 (on a scale of 100.0) were defined as indicative of inadequate shoulder condition.

Drainage Problems

Drainage problems are assessed through field personnel reports of locations with drainage problems. Drainage deficiencies are measured as percentage of 0.1-mi segments in projects with reported drainage problems. Thus, the deficiency measure considers only the extent of drainage problems. When more than 40 percent of the length of a project has reported problems, drainage deficiencies are defined high.

Pavement History

Pavement history is incorporated by considering the age of the surface layer. This is currently the only historical maintenance data that are reliably available for most projects. The information on surface age was initially collected through questionnaires to field personnel; it will eventually be available from the pavement data base. The definition of old pavement depends on the pavement type. Concrete pavements more than 15 years old and overlaid pavements more than 7 years old are considered to be old. Resurfacing evaluation also uses pavement type as a decision factor.

Appurtenance Safety

Appurtenance safety refers to items such as lighting, traffic barriers, and guiderails. The results of a guiderail condition survey have been adapted to provide an indication of appurtenance safety. Projects for which 40 percent or more of the existing guiderail is clearly substandard are defined as having high appurtenance safety deficits.

Traffic Control

Agency policy has defined locations at which work hours are restricted because of problems with traffic control and congestion. Locations at which there are year-round limitations on the roadway occupancy of maintenance crews were considered high urban with respect to traffic control.

Implication Scoring

Condition-implication (C-I) tables are used to score the appropriateness of the various scopes work evaluated for each project undergoing detailed analysis. The C-I tables for rehabilitation evaluation are given in Table 2.

TABLE 2 C-I Table for Rehabilitation Evaluation

CONDITION		IMPLICATION								
		Resurfacing			Rehabilitation with overlay			Reconstruction		
Factor	Level	Support	Ambiv.	Negate	Support	Ambiv.	Negate	Support	Ambiv.	Negate
Function	Adequate	13.1	13.1	−50.2	−6.6	29.4	−50.2	−50.2	29.4	−6.6
	Inadequate	−50.2	−6.6	29.4	24.0	7.6	−61.0	51.2	−39.3	−61.0
Structure	Adequate	30.6	−29.4	−29.4	−29.4	30.6	−29.4	−70.2	−70.2	−71.2
	Inadequate	−63.3	−63.3	64.5	13.6	19.3	−63.3	13.6	19.3	−63.3
Deterioration rate	High	−66.9	−54.0	61.5	50.7	−38.8	−60.5	61.5	−54.0	−66.9
	Medium	−28.1	7.6	7.6	50.7	−38.8	−60.5	−28.1	45.4	−60.5
	Low	50.7	−38.8	−60.5	−17.3	−17.3	18.4	−49.7	−28.1	40.0
Traffic loads	High	−51.8	−33.3	43.4	−24.0	−5.5	15.7	−5.5	15.7	−24.0
	Other	−14.8	15.7	−14.8	−14.8	15.7	−14.8	−14.8	−14.8	15.7
Pavement safety deficits	High	3.1	9.4	−20.3	−4.6	5.5	−4.6	0.0	0.0	0.0
	Other	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Shoulder condition	Adequate	21.6	−20.9	−20.9	13.6	−4.8	−20.9	64.2	−4.8	−4.8
	Inadequate	−24.0	1.6	13.6	29.7	−12.8	−44.9	29.7	−12.8	−44.9
Drainage deficits	High	−19.6	20.3	−19.6	−19.6	−19.6	20.3	−19.6	−19.6	20.3
	Other	−48.1	−36.0	42.8	35.2	−27.0	−42.1	20.3	−4.5	−34.5
Surface age	Old	−16.1	2.4	6.0	6.0	2.4	−16.1	6.0	2.4	−16.1
	Other	−5.5	11.2	−16.1	−5.5	13.0	−19.6	−9.1	−5.5	7.8
Appurtenance safety deficits	High	−16.6	−9.4	13.3	13.3	−9.4	−16.6	0.0	0.0	0.0
	Other	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Traffic control	High urban	−19.1	−15.4	17.6	−1.8	8.4	−14.2	8.4	−1.8	−14.2
	Other	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table Organization

In the C-I tables, individual cells correspond to hypotheses that incorporate (a) a decision factor, (b) a measured level of the decision factor, (c) a scope of work, and (d) an indication of the degree of support for the hypothesis. The implication score recorded in each cell represents an engineering judgment about the truth of the corresponding hypothesis. For example, the cell in the extreme upper left-hand corner of the rehabilitation C-I table (Table 2) corresponds to the hypothesis that if the pavement function is adequate, then this is supporting evidence that resurfacing is the appropriate scope of work. The implication score associated with this hypothesis is 13.1. (Derivation of implication scores is presented in the following section.) Scores may be either positive or negative, depending on whether the hypothesis is judged to be true or false, respectively. The greater the absolute value of the score, the greater the engineer's confidence in his judgment. A score value equal to 0.0 indicates that the measured-level of the given factor provides no information about whether the proposed scope of work is appropriate or not.

Implication Score Derivation

The implication scores in Table 2 are derived on the basis of engineering judgement accumulated over years of experience with Thruway pavements. First, the decision factors for initial implementation were evaluated to provide an importance score, which represents the importance of that factor in decision making, and a measurement confidence score, which represents the confidence that the factor is well measured (Table 3). Then the hypotheses in the C-I tables were evaluated to provide subjective probabilities for each hypothesis to be true for any given project. The sum of the probabilities of the three hypotheses associated with each factor, level, and project scope combination is equal to 1.00, as the three events are mutually exclusive and exhaustive. To facilitate handling of cases where the factor-level combination does not provide any information about project scope, the subjective probability values were transformed to a scale symmetric about zero.

The transformed values associated with each factor were then multiplied by a constant (determined from the measurement confidence and importance scores) to account for the greater impact of factors with higher importance and measurement confidence. Therefore, the maximum possible score varies from factor to factor. Table 3 gives the measurement confidence scores, importance scores, and maximum implication scores for each of the factors involved in the rehabilitation and resurfacing evaluations.

Project Scope Scoring

A project scope score is determined for each scope of work considered. This is a three-step process. First, each of the

TABLE 3 Factor Importance for Detailed Analysis

Factor	Importance Score	Measurement Confidence Score	Maximum Implication Score
REHABILITATION EVALUATION			
Functional adequacy	10	6	71.9
Structural adequacy	10	10	74.6
Deterioration rate	10	5	71.2
Traffic loads	8	10	61.1
Pavement safety	7	6	51.6
Shoulder condition	7	8	52.9
Drainage problems	7	3	49.5
Pavement history	3	4	23.1
Appurtenance safety	3	5	23.7
Traffic control	2	10	20.4
RESURFACING EVALUATION			
Functional adequacy	10	5	73.9
Accident rate	9	5	66.9
Rutting	7	3	51.4
Deterioration rate	5	4	38.0
Surface type	4	10	35.2
Surface age	4	9	34.5

decision factors must be evaluated for the project. Second, the arithmetic mean of the implication scores that correspond to the factor levels present is calculated for each column of the C-I table. Scores of 0.0 are omitted from the calculation, because they indicate that the corresponding factor level provides no information useful for inferring the scope of work. Third, the arithmetic means are combined into a final project scope score (PSS) by the formula

$$\text{PSS} = S^* + A + N^* \qquad (1)$$

where

$$S^* = \begin{cases} 2S & \text{for } S > 0 \\ S & \text{for } S \le 0 \end{cases}$$

S = arithmetic mean of nonzero implication scores corresponding to factor levels in supporting column;

A = arithmetic mean of nonzero implication scores corresponding to factor levels in ambivalent column;

$$N^* = \begin{cases} -2N & \text{for } N > 0 \\ N & \text{for } N \le 0 \end{cases}$$

N = arithmetic mean of nonzero implication scores corresponding to factor levels in the negating column.

The scope of work with the maximum project scope score is recommended as the most appropriate. Several formulations for calculating the final score were investigated. Scope of work recommendations are relatively robust with respect to the formula used.

TABLE 4 Summary of Scope-of-Work Recommendations, 1989

Recommended Scope–of–work	Mean Project PDI	Number of Projects	Percent* of System
Reconstruction	29.2	13	3.9
Rehabilitation	51.9	49	18.8
Resurfacing	71.2	18	15.9
Corrective	73.8	27	13.5
Preventive	88.7	56	37.9
Do–nothing	99.4	13	7.0
(unrated)	—	8	3.0

* based on centerline miles

PROJECT ALTERNATIVES

For each project, a recommended scope of work is generated from the initial screening and detailed analysis procedures. The scope of work defines alternatives to be considered in the network-level analysis by indicating that alternatives involving a greater scope of work than the recommended one need not be analyzed. There is one preferred alternative for each project, other alternatives being deferral or holding strategies. Generally, the preferred alternative entails performing the itemized standard treatments in the context of the recommended scope of work.

The preferred alternative for corrective, preventive, and do-nothing projects is to perform the standard treatments indicated by distress-treatment matching. The only other alternative is to perform the itemized quick-fix treatments. For do-nothing and preventive projects, the quick-fix treatments are often to do no work.

Resurfacing projects have four alternatives. The preferred alternative is to do the standard treatments followed by resurfacing. Different methods of resurfacing may be applicable, depending on project characteristics. Deferral alternatives are to perform standard treatments only, or quick-fix treatments only, or a disposable overlay. Choice of a deferral treatment will be made by network-level analysis based on considerations of cost, condition, and time.

Identification of alternatives for rehabilitation and reconstruction projects is a more complex procedure. In this case, the scope of work recommended by detailed analysis is only a preliminary suggestion. Alternatives for implementing rehabilitation and reconstruction projects are generated by considering methods of both rehabilitation and reconstruction and the alternatives associated with corrective and resurfacing scopes of work. Although all such projects will initially have the same alternatives, the characteristics of alternatives (cost, service life, etc.) generally vary between projects, due to local variations in performance characteristics. Evaluation of the long-term costs of each alternative will provide an indication of the preferred alternative and feasible deferral alternatives for each project. Such a life-cycle cost analysis is critical to identify the alternatives that could be most cost-effective over time. The implications of the large budgetary outlays associated with these types of projects warrant a detailed financial analysis that is beyond the scope of the current study.

COST ESTIMATION

The procedure for estimating the cost of an alternative varies depending on the scope of work. The cost of alternatives with corrective, preventive, or do-nothing scope of work is estimated from the unit costs of performing the indicated individual treatments. Alternatives with a resurfacing scope of work are cost-estimated by adding the costs of performing individual (preparatory) treatments to the cost of resurfacing. The cost of alternatives with a rehabilitation or reconstruction scope of work is a preliminary estimate based on average costs (on a lane-mile basis) of similar projects. Refined cost estimates can be performed using the NYSTA's "engineer's estimate" system, after details of nonpavement work are determined.

Alternatives with rehabilitation or reconstruction scope of work are typically associated with significant amounts of nonpavement work (e.g., rock slope remediation, bridge work). Moreover, implementation constraints (e.g., mobilization, user delay) generally result in the combination of several adjacent projects into a single job. Thus, costs of these projects cannot be easily estimated. Currently, preliminary lump-sum cost estimates are obtained on the basis of cost per lane mile from projects of similar nature implemented in recent years. The cost of resurfacing is estimated similarly, but as mentioned, the cost of individual (preparatory) treatments is calculated separately and added to the resurfacing cost to obtain the total cost of alternatives with resurfacing scope of work.

IMPLEMENTATION

The first implementation of the described treatment recommendation methodology performed in 1990, based on pavement condition in 1989. The initial screening and detailed analysis were developed using a series of spreadsheet macros. Implementation of cost-estimation spreadsheets is currently under way. Refinement of treatment completion rates is pending. Alternatives have not been explicitly listed, as the data required for their cost estimation are not yet available. Table 4 presents the results of the scope-of-work recommendations.

A preliminary comparison of NYSTA's current empirically derived paving program and the more systematic treatment recommendation results indicates relatively good agreement between the two. Key findings of the preliminary validation study are summarized in the following:

- The treatment recommendation methodology identified 62 candidates for rehabilitation evaluation. Of these, half were scheduled for paving in 1990 or 1991. Most of the remainder (24) have paving scheduled before 1996.
- The treatment recommendation methodology identified 45 candidates for corrective treatment. Of these, 18 were suggested for resurfacing. The paving program designated 14 of the 45 corrective candidates for paving 1990 or 1991; 8 of the 14 were those suggested for resurfacing by the treatment recommendation methodology. Eight other corrective candidates are scheduled for paving before 1995.
- Only 35.4 mi of pavement identified by treatment recommendation as having the scopes of work do nothing or preventive are scheduled for paving before 1993.

The treatment recommendation methodology was implemented for 1989 projects in early 1991. This time lag between distress assessment and treatment recommendation is an artifact of the research and development process. It is not expected to persist after the system becomes operational. When developing sequential methodologies, outputs of prior procedures must be obtained before development of subsequent procedures can be initiated.

DISCUSSION OF RESULTS

The goal of project-level analysis is to recommend and rank procedures for the remediation of each pavement segment. While organizing tasks needed to achieve this goal, it became apparent that it is efficient to structure the problem into two subproblems based on the complexity of project condition. Such a formulation facilitates the efficient use of resources for data collection and analysis. Because it is expensive to collect, store, and analyze data, it is judicious to tailor data requirements and analysis complexity to the needs of the decision process. Just as superfluous data need not be considered, those that contribute to decision making must not be excluded. Available resources are used most effectively by increasing data requirements and analysis complexity only for those projects with relatively complicated conditions. This concept is the basis of the staged problem-solving formulation.

The applied structure of the problem of project-level analysis leads to a cost-effective strategy for pavement management in which focus is placed on complex projects, with due consideration for preventive maintenance. It takes advantage of the fact that conditions requiring preventive maintenance are quickly and easily identified. An important finding of this study is that more than 40 percent of Thruway pavements currently exhibit simple condition. Early identification and rapid evaluation of these simple projects allows resources and effort to focus on those with complex condition. Complex projects account for the majority of annual funding requirements.

Analysis of complex projects incorporates a series of increasingly refined classifications. For example, a project may be initially characterized as complex, then as a rehabilitation candidate, and finally recommended for reconstruction. This classification accommodates the customization of analytical procedures for achieving specific tasks. As an illustration, rehabilitation and resurfacing evaluation routines incorporate only the decision factors relevant to the types of projects being analyzed. By customizing the analyses, more specific recommendations can be made. Note however, that specificity is only possible when the detailed data necessary to support it are available. The types and amount of data required for pavement management decision making are a function of system size and analysis detail.

The use of data at any level of decision making is constrained by its availability, accessibility, and appropriateness for measuring a given characteristic. These practical limitations can significantly affect the validity of an analysis. In the NYSTA detailed analysis procedure, the problems associated with factor measurement are mitigated by the assessment of factor levels. Factors are appraised in binary or tertiary levels such as adequate versus inadequate or high versus normal versus low. This facilitates use of data of varied types and degrees of accuracy. Each factor can be evaluated at the highest possible level of accuracy, whether it is subjective or objective, discrete or continuous. For the initial implementation, boundary values for defining factor levels were defined at natural breaks in the distribution of values for 1989 projects. Recall that the implication scores incorporate an adjustment that reduces the impact of those decision factors that are not well measured. The equations used to characterize factor levels were derived from the data available during the 1989 implementation. The validity of these equations cannot be proved at the current stage of development.

The implication scoring technique evolved as an alternative to using decision trees for rehabilitation decision making. Initial knowledge acquisition activities identified conditions that affect decision making but generally could not detail the impact of a given factor. The decision trees derived from these results were unsatisfactory. The large number of possible combinations of decision factor levels precluded an investigation of each such scenario.

SUMMARY AND CONCLUSIONS

Methodologies for distress assessment, project characterization, treatment recommendation, and project ranking have been developed and implemented as part of the NYSTA's pavement management system. The goal of the present study was to develop a methodology to generate condition-based project treatment recommendations for a single year. The analysis was structured into two subproblems, depending on the complexity of pavement condition. The results of the analysis support future work in the areas of life-cycle cost analysis, multiyear planning, and program optimization.

The described formulation aimed to determine systematically project requirements in a manner consistent with the authority's current practices and experience. Field personnel and management have participated extensively in system development and implementation. The treatment recommendation methodology combined matrix and decision tree methods to identify specific treatment requirements, suggest the scope of work for implementing treatments, and generate feasible alternatives for use in network-level analysis. The procedure for suggesting a scope of work followed a staged approach that increases analysis complexity as the pavement exhibits more complicated conditions. Treatment recommendation has been implemented for the 1989 projects.

The treatment recommendation methodology is currently undergoing review and adjustment and is expected to continue evolving after it becomes operational. Because of the sequential nature of the development process, some modules are currently more mature than others.

On the basis of the development and preliminary implementation presented in this study, the following conclusions were drawn:

- Decomposition based on the complexity of pavement condition enables clear communication with a wide range of experts and efficient development of decision methodologies.
- A decision process that generates treatment recommendations through a process of increasingly refined classifications of the scope of work fosters efficient use of resources for data collection and analysis.
- Acquisition and use of experience is a critical part of the development activity. Good communication between experts and developers is a fundamental requirement for creating a system compatible with agency operations.

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Views and opinions expressed herein do not necessarily reflect those of the New York State Thruway Authority.