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Transportation Research Record 1216

Contents

Foreword

This Record contains seven papers on various means of identifying and evaluating the effectiveness and efficiency of pavement management efforts. Research on this topic is ongoing worldwide: three of these papers report on efforts in the United States, two on work in Canada, and two on projects in Egypt. This record should be of interest to any professionals involved in pavement management.

The development of state pavement management systems (PMSs) from conception through change and adaptation to operation is the subject of the paper by Maze et al. The authors provide an administrative viewpoint on many of the issues that should be addressed in establishing or improving a PMS. Al-Suleiman et al. present the results of their research efforts to identify the effects of routine maintenance on pavement surface condition and service life in Indiana. They provide insights into the relationship between the level of routine maintenance expenditures and the need for pavement resurfacing for various types of pavements.

The integration of life cycle cost information and pavement performance factors in a decision support model for the evaluation of maintenance policies is the subject of the paper by Azmy et al. The authors provide a detailed description of their project to provide the Egyptian Roads and Bridges Authority (RBA) with factual information to assess alternative maintenance policies. Zaghloul et al. provide a description of their efforts to develop a simplified pavement maintenance cost model for use by the RBA. The model is based on maintenance cost as a function of pavement condition.

The relative efficiency of highway maintenance crews performing routine maintenance activities in Ontario is the subject of the paper by Kazakov et al. The model that they develop can be used to identify inefficient maintenance crews and possibly provide improvement. Jenkins et al. describe their development of test procedures to evaluate the performance characteristics of coal tar emulsion seal coats. The results were used to develop guidelines for determining the optimum quantities of additives, additional water, and sand for a given set of materials. The development of a framework for the selection of pavement preservation treatments is the subject of the paper by Hajek and Phang. Using linear programming techniques, the authors describe a means of optimizing project-specific strategies within funding constraints.

Case Studies of the Administration of Three Statewide Pavement Management Systems

T. H. Maze, Neal R. Hawkins, and James K. Cable

This paper discusses three case studies of the pavement management systems used by the state departments of transportation in Iowa, Arizona, and Pennsylvania. These case studies demonstrate how existing successful systems operate from an administrative point of view. The original intent of the research was to answer a number of practical questions raised by the managers of a state department of transportation that was considering the use of a pavement management system. Some of the questions asked included: How much will the system cost? How will a pavement management system impact current decision making? Should pavement management be controlled within the central office? Should field divisions play a major role in the system? This paper seeks to provide solutions to these questions through the examples provided by other states.

The purpose of the research described by this paper is to demonstrate how respected statewide pavement management systems operate from an administrative point of view. Much information is available on the pavement management techniques used by various agencies (such as the distress measures collected, the use of optimization programs for allocating resources, and decision rules for selecting pavement treatments). However, little is available regarding the role of the pavement management system within these agencies, the cost of planning, designing, developing, operating, and maintaining a pavement management system, and how the pavement management system helps determine the allocation of resources.

Originally, the research was conducted for a state department of transportation that was considering the development of a statewide system (*1*). At the feasibility stage, top management acknowledged a number of organizational and administrative issues, including the following practical concerns:

- How much will the system cost?
- How will a pavement management system impact current decision making?
- Should restoration, rehabilitation, and reconstruction programming, which use the pavement management system as a resource, be controlled by the central office with regional offices only reviewing the program, or should the process be initiated at the regional level?

Researchers were sent to state departments of transportation that were respected for their pavement management systems. The states visited were Iowa, Arizona, and Pennsylvania. The systems in each of these states were developed with different approaches, take different approaches to the pavement management process, and evolved at different paces.

Iowa State University, Ames, Iowa 50011.

IOWA PAVEMENT MANAGEMENT INFORMATION SYSTEM

The Iowa Pavement Management Information System (IPMIS) was, for the most part, developed in-house. The Iowa Department of Transportation (IDOT) has collected pavement conditions (such as roughness and structural capacity) since the late 1950s and maintained the information in various uncoordinated forms. In the late 1970s, IDOT decided to integrate its pavement condition measurement surveys and automate its condition data processing. The joining of these independent efforts into a systematic data collection effort became the existing IPMIS.

The current computer software for the IPMIS resides on IDOT's mainframe computer, and the individual pavement condition and pavement construction history files reside in individual flat files (not a relational/hierarchical data base file). A new data management system is being installed to merge the pavement condition and construction history data files into one relational data base system, integrate data storage and retrieval, and permit ad hoc data queries.

Pavement Condition Data Collection

The IPMIS contains data that cover five pavement condition attributes (*2*):

1. Skid resistance measured using locked wheel skid trailers,
2. Structural adequacy measured using a Road Rater,
3. Roughness measured using an electromechanical ride meter (the Iowa, Johannsen, and Kirk Ride Indicator),
4. Surface distress visually measured using a crack-and-patch survey, and
5. Remaining pavement life measured in 18-kip equivalent single-axle loads (ESALs) until terminal pavement serviceability is reached.

Pavement Section Evaluation

IDOT uses the field-generated condition data, except the skid resistance data, to evaluate pavement sections through a pave-

	Factor Value 1	2	3	4	5	6	7
Percent Remaining 18 Kips	<-19	-19	0	10	25	45	>70
P.C.C. D-Crack Occurrence Factor	> 4	4	3	2	1		0
Relative Structural Ratio	0.40	0.50	0.60	0.70	0.80	0.90	1.00
Maintenance Costs							
Rut Depth	> .50	.40	.30	.20	.10	.05	< .05
PSI Deduction	> .80	.60	.40	.25	.15	.05	< .05
Longitudinal Profile Value (I.J.K. Ride)	< 3.00	3.20	3.40	3.55	3.65	3.75	> 3.75
P.S.I. Decrease/Year 6 year basis	> .20	.20	.17	.14	.11	.08	< .05

Add factors and compute to a 7 point scale.
If PSI<2.0, the Δ_6 in PSI will reflect a factor value of 0

FIGURE 1 Iowa pavement management matrix.

ment management matrix. The matrix contains values for eight measures of pavement condition:

1. Percentage of remaining 18-kip ESAL life,
2. D-cracking occurrence,
3. Structural rating,
4. Maintenance costs,
5. Average rut depth,
6. Present Serviceability Index (PSI) (*3*),
7. Roughness, and
8. PSI decrease per year.

As shown in Figure 1, each of these eight condition measurements is divided into seven individual categories (factor scores), where 1 is poor condition pavement and 7 is good condition pavement. The matrix value for a pavement section is determined by entering the matrix for each factor and measured value and obtaining the corresponding factor value at the top of the appropriate column. For example, if the pavement has received loadings equal to its design life (0 percent remaining), then the pavement receives a factor score of 3 for the remaining pavement life. To obtain an overall measure of the pavement condition, the factor scores of all pavement condition measures are added and the sum is recomputed into a score on a scale from 1 to 7. Summary listings in decreasing matrix value, by highway district, or by matrix factor can be generated to assist administrators in developing construction and maintenance programs for the next 1 to 5 yr.

IDOT is developing a pavement condition rating (PCR) system for the condition measurement included in the pavement management matrix. The PCR would be a composite score from 0 to 100, where 0 is the poorest condition pavement and 100 is the best condition pavement. The rating system will be dependent on the pavement type, such as asphalt concrete (AC) pavement, portland cement concrete (PCC) pavement, continuously reinforced PCC pavement, and PCC pavement overlaid with AC (composite pavement). By independently factoring the condition scores to a 100-point scale for each pavement type, the composite ratings are customized for each pavement type and become comparable. Therefore, the 100-point system will permit prediction and prioritization of pavements for rehabilitation. Further, a 100-point scale PCR will be compatible with IDOT's 100-point scale sufficiency rating, which will permit the two systems to be used together to develop programs that meet pavement rehabilitation and traffic capacity needs concurrently.

Role of Pavement Management at IDOT

The IPMIS is currently managed by IDOT's Office of Materials, which is part of the Highway Division. The Office of Materials has historically been responsible for collecting pavement condition data and performing some data evaluation. When the IPMIS becomes completely operational, the Planning and Research Division will assume management responsibility for the IPMIS. The Highway Division will continue to collect and evaluate the condition data, while the Planning and Research Division conducts programming activities. This will provide a system of checks and balances to improve data quality and encourage cooperation among IDOT units.

The primary role top management foresees for the IPMIS

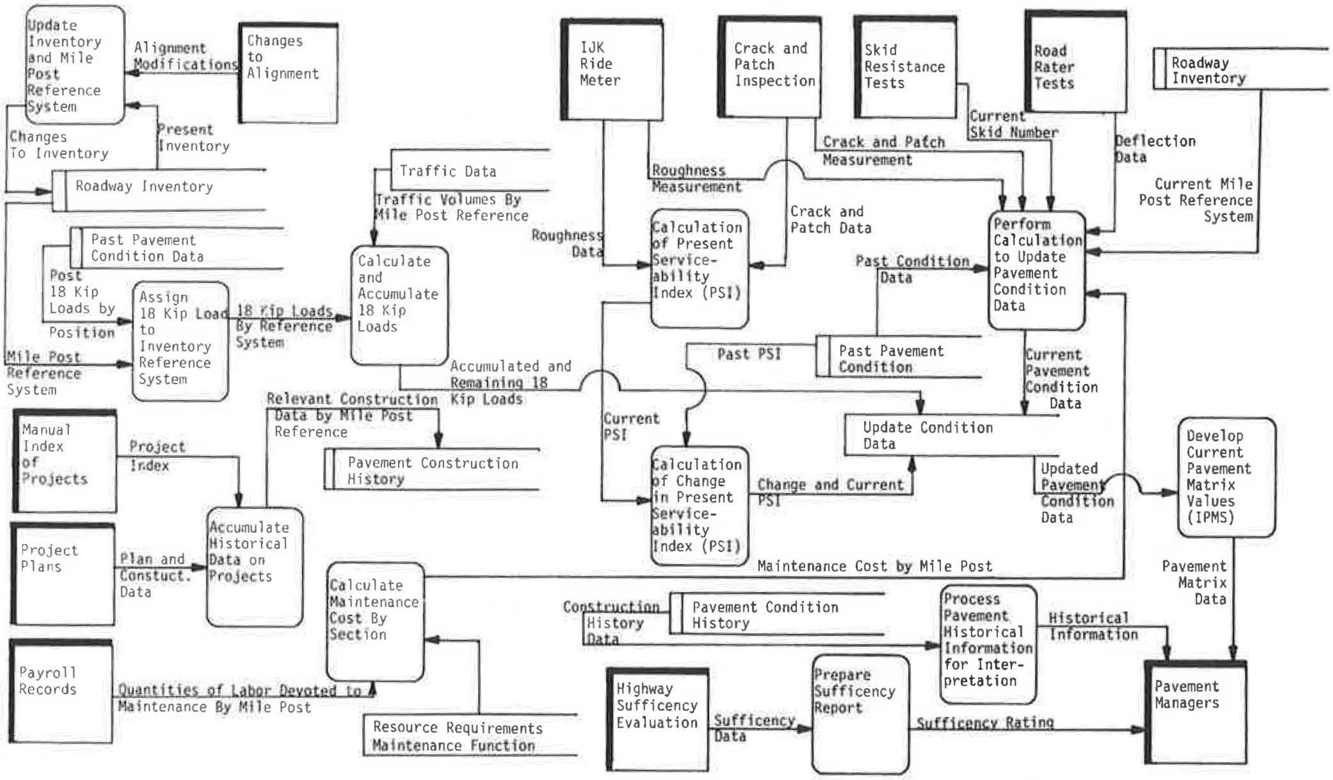

FIGURE 2 Data flow diagram of the Iowa pavement management system.

is in the programming of major pavement rehabilitation. Once the 100-point scale PCR system is operational, then the PCR will complement IDOT's sufficiency rating in the development of the highway improvement program.

The pavement management system administration has evolved from a Pavement Management Task Force consisting mostly of top management staff to the current Pavement Management Committee. Because of the top managers' demanding schedules, the task force met infrequently and the pavement management staff did not receive adequate direction. As a result, the development of the IPMIS lacked momentum. More recently, a Pavement Management Committee Task Force was formed of mid-level managers. These members meet more frequently and administer developmental activities, while the Pavement Management Committee sets policies and reviews task force activities. The development pace of Iowa's system has quickened since this task force was established.

System Inputs, Outputs, and Processes

Figure 2 is a simplified diagram outline of the data flow in the IPMIS. The flat rectangles represent data stores (data files), the double-edged boxes are external entities that begin or end data flows (pavement condition collectors and output users), the rounded rectangles are processes (compiling of data and computing), and the arrows are data flows. Some of the data stores have been drawn more than once to reduce the clutter. These data stores have a double line across their left-hand side.

The current IPMIS is a relatively simple data base system

TABLE 1 IOWA COST OF PAVEMENT CONDITION TESTS

Evaluation Test	Cost/2-Lane Mile ($)
IJK Ride Meter	9.41
Skid resistance test	15.06
Pavement deflection	34.92
Pavement texture test	86.16
Crack and patch survey	101.71

with a series of flat files. However, the development of this system took roughly 5 man-years, and an estimated 2 man-years will be required to place the IPMIS on a relational data base management system.

One of the largest difficulties in managing the data base has been the coordination of a nonstandard pavement location coordinate system. Iowa's pavement management system operates both on a physical milepost location system that originates at the west and south state lines and on an imaginary milepoint system that originates at the west or south line of each county for a particular route. Other data are referenced in other nonstandard systems. For example, limits of construction projects are based on milepoints.

Costs

IDOT's costs of performing pavement condition tests per mile are listed in Table 1. These figures include labor cost, depreciation on test equipment, and the cost of equipment maintenance and operation. It should be noted that, although the entire state highway system condition is measured, measure-

TABLE 2 ANNUAL OPERATING AND ADMINISTRATION COSTS OF IPMIS

Data Collection	Two-Lane Miles		Cost/Mile		Total
IJK Roadmeter	5,050	X	$ 9.41	=	$ 47,521
Friction (not in Matrix)	5,000	X	15.06	=	75,300
Road Rater	3,000	X	34.92	=	104,760
Crack & Patch Survey	800	X	101.71	=	81,368
Administration (2 P.E.s, 1 E.I.T., 1 Tech-4, 1 Tech Supervisor-2, and 2 Temp. Eng. Students)					50,000
Traffic, truck weight and class, 18 kip ESALs			Est.	=	50,000
Equipment Maintenance Costs			Est.	=	30,000
Computer Program Development			Est.	=	35,000
Pavement Management Task Force (5 people x 2 hours/week x 52 weeks/year x $20/hour)					10,400
Pavement Management Committee (8 people x 2 hours/month x 12 month/year x $30/hour)					5,760
					$490,109
					(roughly $500,000 per year)

ments are made only on random samples. For example, the crack-and-patch survey is conducted on ½-mi subsections within each 5-mi section. Therefore, the cost per mile of a crack-and-patch survey is actually the cost of evaluating two 5-mi sections.

The costs of operating and administering the IDOT pavement management system are listed in Table 2. These costs have increased dramatically in the past few years because of increased pavement management activity. In 1987 it cost IDOT roughly $500,000 to operate and administer the IPMIS, while in 1985 only $225,000 was spent on the operation and administration of the system.

ARIZONA PAVEMENT MANAGEMENT SYSTEM

The Arizona Department of Transportation (ADOT) began investigating the development of a pavement management system in the mid and late 1970s. At that time, there were two primary management issues that ADOT hoped to address through the use of a pavement management system (*4*):

1. Estimates of preservation needs and maintenance decisions were mostly based on the judgment of district engineers. The concern was that judgmental decision making might lead to nonuniform pavement conditions across the state. Also, the state government was aware of the subjective nature of these decisions and was reluctant to appropriate additional funds when resource allocation decisions were made in this manner.
2. A method for predicting the long- and short-term effects of funding shortages on road conditions and a systematic procedure to cope with budget cuts were needed.

In 1978, ADOT hired a consultant to develop a pavement management decision-making tool for Arizona. The focus of this system is at the network level. The optimization uses a Markov chain model, which forecasts the proportion of the highway network that will change from one condition state to another during a given year. A linear program is then used to select treatments and allocate resources each year. Unfortunately, the network optimization forecasts only proportions of the entire highway network that will be in a specific condition state. In other words, the identity of each section is lost.

The network optimization system forms the focal point of Arizona's current pavement management system. However, ADOT has developed programs to augment the original system and collects data in its pavement evaluation that are not used by the original model. For example, one of ADOT's key pavement management tools is a heuristic algorithm used to predict the condition of a specific pavement section and automatically select rehabilitation treatments. When results of the heuristic algorithm are compared to those of the optimization, the cost forecasts are generally very close.

Pavement Condition Data Collection

ADOT collects and maintains data files for several types of pavement condition data:

- Surface distress measured through a visual survey of the first 1,000 ft^2 of the pavement at each milepost,
- Skid resistance measured using a Mu-Meter,
- Roughness measured using a Mays Ride Meter mounted on the rear axle of a specially equipped passenger car, and
- Structural adequacy measured using either a Dynaflect unit or a Falling Weight Deflectometer.

Role of Pavement Management at ADOT

ADOT's pavement management system is currently managed within the Materials Section, which is part of the Highway Division. The Highway Division is divided into two groups: the Highway Development Group and the Highway Operations Group. The Materials Section is part of the Highway Operations Group. The Materials Section contains three areas: Geotechnical Services, Testing Services, and Pavement Services. Pavement Services includes the Pavement Management Branch and the Pavement Design Branch.

The Pavement Management Branch has 11 employees and is managed by a pavement management engineer. This branch is responsible for collecting pavement condition data and managing the pavement management data base and the pavement management programs.

The primary management responsibility of the Pavement Management Branch is the identification of pavement preservation projects. In 1987, ADOT's pavement preservation budget was roughly $62,000,000. At the start of each fiscal year (July 1), the pavement management engineer meets with the district engineers to begin developing a preservation program. In these meetings, pavement projects and priorities are discussed. Over the next few months, a draft preservation program is developed and the pavement management data base is updated with condition data collected during the summer. After the data base is updated, the network-level models are run and the pavement management engineer refines the preservation projects based on current data. Another meeting is then held with the district engineers to settle on a final preservation program. This program is then presented to the priority planning subcommittee at the beginning of the year, to be included in the 5-yr construction program, which is forwarded to ADOT's board for final approval.

The past pavement management engineer estimated that between 70 and 80 percent of the projects selected through the pavement management system agree with those selected by the district engineers. These groups tend to agree more on the dollars programmed for preservation and less on the specific miles identified for restoration. This is because more expensive projects (such as Interstate restoration) are more easily identified.

The pavement management system was placed in the Materials Section because this section has always performed pavement testing. Pavement management was simply considered an extension of this role. Interestingly, pavement management has largely been used as a network-level pavement restoration planning tool. Even though this planning function is based outside of the Planning Division, ADOT does not intend to change this structure.

System Inputs, Outputs, and Processes

Figure 3 is a data flow diagram of ADOT's pavement management system. The network optimization is a sophisticated program that involves the use of Markov chains and a linear programming model.

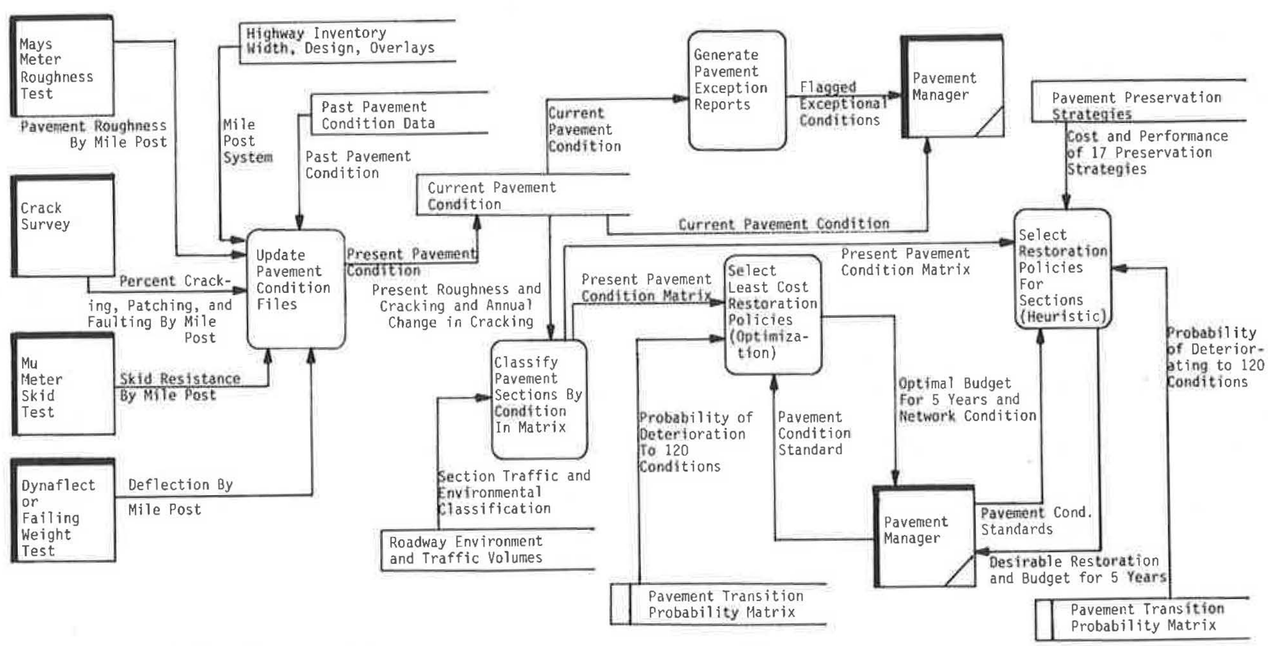

FIGURE 3 Data flow diagram of the Arizona pavement management system.

TABLE 3 ARIZONA COST OF PAVEMENT CONDITION TESTS

Evaluation Test	Cost/2-Lane Mile ($)
Mays Meter Roughness Test	3.48
Cracking and distress visual inspection	4.85
Mu-Meter skid resistance test	5.77
Dynaflect deflection test	21.78
Falling weight deflectometer test	53.22

Costs

To develop the network optimization, ADOT spent roughly $300,000 on consulting services in 1979. Temporary staff were hired for a total of about 13 man-years to work on the pavement management system during its development.

The costs of performing pavement condition tests per mile are listed in Table 3. These figures include labor cost, vehicle rental rates, and employee per diem. They do not reflect the cost of survey equipment depreciation. The cost of visual crack-and-distress tests are low because ADOT inspects only the first 83 ft of each mile of roadway surface (12-ft lane width $\times$ 83 ft = 1,000 ft^2). The annual labor cost of operating the ADOT pavement system is roughly $275,000 (11 staff members).

PENNSYLVANIA'S SYSTEMATIC TECHNIQUE TO ANALYZE AND MANAGE PAVEMENTS

Before 1983, the Pennsylvania Department of Transportation (PennDOT) made several overtures toward the development of a pavement management system. Various committees were appointed to investigate pavement management, but little progress was made. Finally, in 1983, the Pennsylvania secretary of transportation named an eight-person task force to investigate the possibility of developing a pavement management system for PennDOT. If the task force determined that a system was feasible, it would assume responsibility for the development.

The task force members were all mid- to upper-level managers (a district engineer, assistant district engineers, and division managers). Until their first meeting, none of the members knew the identity of the others.

Once the task force had decided that it was feasible to develop a pavement management system, the members were relieved of their normal duties and sequestered for the duration of the project, which took 9 mo to complete. The prototype system took roughly 6 man-years of the cumulative task force members' time.

The original pavement management system designed by the task force was given the name "Systematic Techniques to Analyze and Manage Pennsylvania's Pavements" (STAMPP). The computer program used to automate STAMPP was written in BASIC and run on a microcomputer (5). During the development phase, a demonstration of STAMPP was conducted by applying the system to a single county. Once STAMPP was refined and tested, it was considered ready for application to the remaining highway system.

The PennDOT philosophy on pavement works from the bottom up. The pavement management system is used by the county manager to set pavement maintenance and betterment priorities within the county. An assistant district engineer considers the county manager's recommendations when making project selections for the district. All project-level pavement management analysis is conducted at the district level, whereas network-level pavement management analysis is conducted at PennDOT headquarters. The involvement of headquarters in the process ensures consistency between districts. If a district recommendation deviates from the action recommended by STAMPP, ample justification must be given for not following the program's recommendations. Because STAMPP has only been in operation a short time, PennDOT has not yet developed performance curves to forecast future performance of the system.

Pavement Condition Data Collection

PennDOT has divided the state highway system into approximately 90,000 inventory segments that are roughly ½ mi long. The segment divisions are located at physical changes in the pavement or changes in the characteristics of the traffic loadings (such as an intersection). The beginning and ending of segments are marked by inventory posts, and the segments are used to identify the highway system for all other inventories (such as accident locations and traffic control device locations).

PennDOT collects several types of condition data:

- An extensive visual inspection of the pavement condition is conducted by two individuals (a driver and an evaluator) in a moving vehicle. Five percent of the sections are resampled for quality control. Each year, the entire pavement section is rated and all sections are inspected. Visual evaluations cost slightly less than $13 per mile.
- Roughness is measured using Mays Ride Meters.
- Skid resistance is measured using locked wheel skid trailers.
- Structural adequacy is measured using a Falling Weight Deflectometer on PCC pavements and a Road Rater on AC pavements. These tests average around $88 per mile.

The Role of Pavement Management at PennDOT

In 1983, PennDOT was reorganized to structure the Department by function. The management function of the highway system was placed in a new bureau called the Bureau of Bridge and Roadway Technology. This bureau has three divisions:

1. The Engineering Technology Division, which is responsible for electronic data processing, value engineering coordination, new product evaluations, experimentation and evaluation projects, and technology transfer;
2. The Bridge Management Systems Division, which is responsible for bridge system evaluation and bridge experimentation projects; and
3. The Roadway Management Division, which is responsible for pavement management, pavement design practice, and pavement experimentation projects.

Although these three management divisions control the development of roadway and bridge design and maintenance prac-

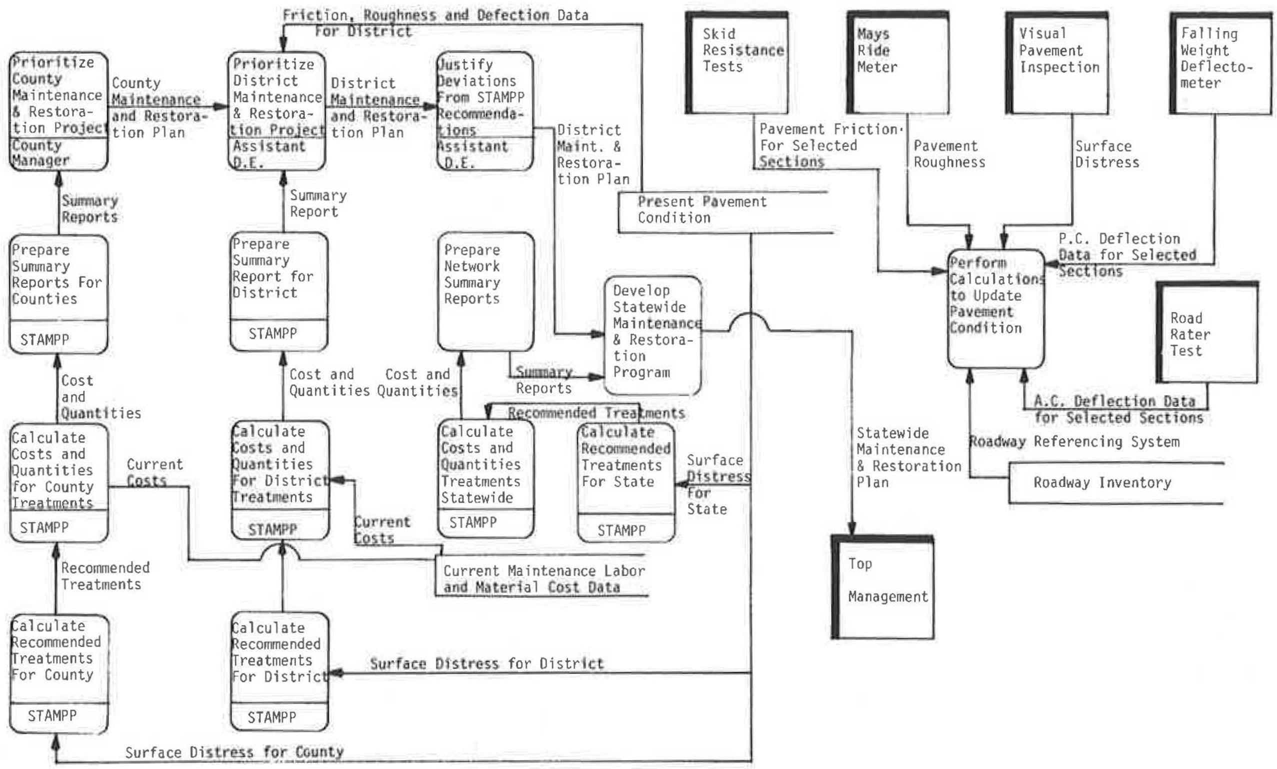

FIGURE 4 Data flow diagram of the Pennsylvania pavement management system.

tice, actual design and maintenance are conducted by the Bureau of Design and the Bureau of Maintenance and Operations.

By reorganizing, PennDOT has avoided orienting the pavement management system toward the objectives of a functional area (such as maintenance, materials, design, or planning). Instead, the system is a management tool available to all functional areas.

System Inputs, Outputs, and Processes

Figure 4 is a data flow diagram of STAMPP. Although STAMPP was originally designed as a standalone system, it is currently a module of the PennDOT roadway management system (RMS). RMS is a computerized information system that integrates pavement management, roadway information (data covering descriptions of the roadway and construction history), special processes (traffic data, accident data, and others), computer-generated straight-line diagrams, and other management functions. Development and testing of the RMS is expected to cost approximately $20 million.

FINDINGS

Each of the three case studies provides a distinctly different approach to the development and administration of a pavement management system. The Iowa system was developed in-house. It has been slow to evolve over its 9-yr history, but progress now seems to be more rapid. Arizona's system was developed by a consultant and later modified in-house. In this highly centralized system, the pavement preservation program is initiated at headquarters, then reviewed and critiqued by the field districts. The Arizona system's primary emphasis is at the network level, and it is principally used in project planning and programming. Pennsylvania's system was developed in-house by a committee of mid- to upper-level managers. It is very decentralized and begins at the county level. This system focuses on the selection of individual projects and is not currently capable of projecting pavement conditions for planning purposes.

RECOMMENDATIONS

From the case studies, general and specific recommendations were made for the state that originally sponsored the research. Many of these recommendations were unique to that state. For example, one critical issue was the pavement management process flow. Should the field divisions begin the annual and 5-yr programming and planning process, following the Pennsylvania model, or should the central office start the process, as in the Arizona model? In the sponsor state, the field divisions had enjoyed a good deal of autonomy in selecting maintenance and restoration projects for the non-Interstate state highway system. Many field division personnel felt that centralizing the processes would erode their ability to direct resources effectively using judgmental factors that could only be known through local experience. Therefore, it was recommended that the project planning and programming process should start within the field divisions and that uniformity between these divisions should be governed by that process.

Other recommendations that involved the unique characteristics of the sponsor state regarded whether the system should be developed in-house or by a consultant and whether the system should initially focus on the development of network-level capabilities or on identifying and prioritizing projects.

The following recommendations can be applied to all agencies:

- Top managers must be committed to the systematic management of pavements. They should be willing to commit a significant level of human and capital resources to the planning, design, and implementation of the system, and system maintenance, operation, and improvement must receive a substantial and continuous flow of resources.
- General education on the pavement management process should be conducted during the initial planning stages to reduce misconceptions by staff members and facilitate receptiveness to the process.
- A committee of mid-level managers and engineers should be appointed to guide the planning, design, and implementation of the pavement management process. These individuals must be relieved of enough routine duties that they can devote a substantial level of effort to their committee assignments. They should attend in-depth pavement management training programs through as many different organizations as possible to expose them to a variety of pavement management philosophies. The training should also include site visits to other states.

In addition, the study identified six major issues that should be addressed in the management plan for a system's development, implementation, and operations. The plan must

1. Establish clearly defined objectives with quantifiable measures of accomplishment. The functions of the pavement management system should be apparent through the objectives. For example, one objective might be that the system should be able to allocate funds, budget, and program projects for up to 5 yr with the goal of minimizing the life-cycle costs of the pavement network. Implied in this objective is that the system will be able to conduct adequate pavement performance forecasts, estimate revenue, establish priorities, and optimize the allocation of funds.
2. Identify output requirements for the various divisions of the department. For example, if one objective is to have the pavement system automatically estimate budgets, the system must be able to output the desired maintenance treatment for pavements calculated by areal measurement.
3. Identify data requirements for the desired outputs. For example, if the system is to select maintenance actions based on the thresholds of deteriorating pavement conditions, then the appropriate conditions must be included in the data collected.
4. Recommend appropriate changes or improvements to current data collection practices. For example, the sponsoring agency is likely to require better collection of truck axle load data. Therefore, improved data collection procedures are necessary.
5. Identify the management positions and staffing levels needed to operate the pavement system. The permanent positions of an engineer-manager, other professionals, and technicians as well as temporary pavement condition survey labor, represent a significant, recurring cost.
6. Determine an oversight role for a pavement management committee. This committee should be responsible for the review and guidance of the permanent staff.

It was recommended that the sponsoring agency should undertake a list of eight activities once the management plan is completed and the above issues are addressed. These activities were all given deadlines and range from top management initiating the pavement management system development process to long-term system development activities.

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Effects of Routine Maintenance Expenditure Level on Pavement Service Life

TURKI I. AL-SULEIMAN, KUMARES C. SINHA, AND THOMAS KUCZEK

This research was undertaken to determine how the level of routine maintenance expenditure affects pavement surface condition and pavement service life. The relationship between pavement roughness and pavement age was examined under different levels of routine maintenance expenditure. Surface roughness was used as a measure of pavement surface condition, and pavement age at terminal roughness value was considered as a measure of pavement service life. The effects of traffic loading and regional factors were included in this relationship. Data on a selected number of pavement sections from the Indiana state highway system were used to develop appropriate prediction models. The results of these models indicated that, if patching and crack-sealing expenditure increases from low to high levels, resurfacing can be postponed 1 to 3.3 yr for overlaid pavements and 1.6 to 8 yr for flexible pavements.

Resurfacing a highway section usually causes an immediate reduction in the need for pavement routine maintenance (*1*). However, past research has not revealed how long resurfacing can be postponed if appropriate levels of pavement surface maintenance are undertaken. This research effort studied the effects of routine maintenance expenditure level on pavement surface condition and resurfacing need. A relationship between pavement roughness as a measure of pavement surface condition and pavement age as a measure of pavement service life was introduced under different levels of routine maintenance expenditure. This relationship was used to relate pavement resurfacing needs to the level of routine maintenance. An assumption was made that improvements in pavement condition are positively related to the level of routine maintenance expenditure.

BASIS OF THE APPROACH

Pavement performance is a result of the combined effects of traffic load, environment, age, initial design and construction, and past maintenance. The most widely used aggregate pavement performance model is the relationship between axle loading and pavement deterioration developed through the AASHO Road Test (*2*). An approach proposed by Fwa and Sinha (*3*), which was based on the serviceability performance concepts developed by the AASHO Road Test, measures pavement performance in terms of Present Serviceability Index–Equivalent Single-Axle Load (PSI–ESAL) loss. In the research documented in this paper, the following initial assumptions were made:

- Pavement roughness can be used instead of PSI as a direct quantitative measure of pavement performance. This assumption is derived from the conclusion of several studies (*4,5*) that the use of roughness measurements is often sufficient for predicting the serviceability index. Roughness data are readily available to most highway agencies. Also, the general public perceives pavement roughness as more critical than structural adequacy in determining the timing for pavement improvement (*6*).
- Pavement age, as measured from the most recent reconstruction or resurfacing, can be used to represent the combined effects of traffic and environment for a small range of traffic volume as well as for a small variation in climatic conditions. Since pavement age alone can account for about 80 percent of the variations in damage responsibilities (*3*), this assumption is reasonably valid. Consequently, pavement age at terminal roughness value can be used as a measure of pavement service life.
- Pavement type and highway class represent initial design and construction.

CONCEPTUAL RELATIONSHIP BETWEEN PAVEMENT ROUGHNESS AND PAVEMENT AGE

To predict the effect of routine maintenance expenditure level on pavement service life, pavement performance must be considered over time under different levels of routine maintenance. Since routine maintenance expenditure level can be expected to represent both the quality and quantity of maintenance work, it can be used as a measure of the level of routine maintenance performed on a given pavement. On the basis of this assumption and the pavement performance and maintenance relationship developed by Fwa and Sinha (*3*), pavement roughness can be related to different expenditure levels of routine maintenance (L_i) as shown in Figure 1. Figure 2 shows pavement performance over time under three different maintenance levels. Pavement service life (n) under zero-maintenance can be determined on the assumption that, when pavement roughness reaches a terminal value (RN_T), the pavement needs to be resurfaced or reconstructed. Resur-

T. I. Al-Suleiman, Jordan University of Science and Technology, Irbid, Jordan. K. C. Sinha and T. Kuczek, Purdue University, West Lafayette, Ind. 47907.

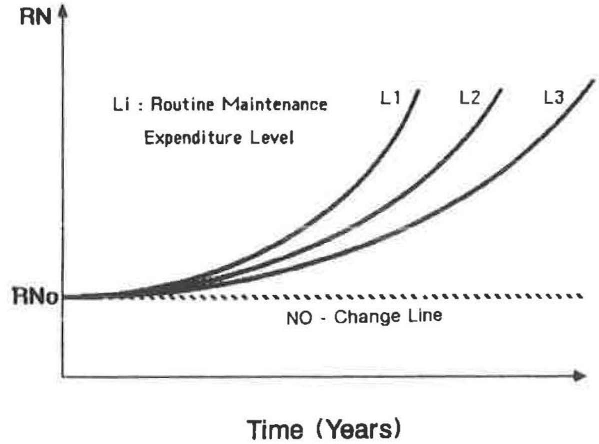

FIGURE 1 Pavement roughness curves for different levels of routine maintenance expenditure.

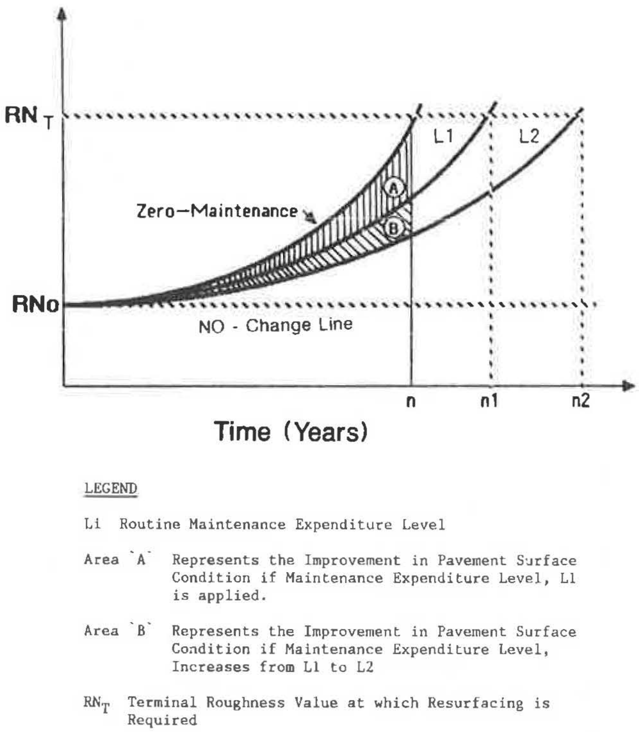

FIGURE 2 Effects of routine maintenance expenditure levels on pavement service life.

facing is defined in this research as the placement of additional surface material over an existing pavement to improve serviceability or to provide additional strength. It is considered a first level of improvement (one that significantly affects serviceability) as opposed to higher levels of improvement such as restoration or rehabilitation (requiring improvement of structural support) and reconstruction (where an old pavement structure is removed and replaced).

RN_T is equivalent to 2.0 or 2.5 PSI, depending on pavements type and highway class. Area A in Figure 2 represents the improvement in pavement surface condition over time (n) if expenditure level L_1 is applied instead of zero-maintenance. Area A is also equivalent to the reduction in deterioration of pavement surface condition if L_1 is applied. Area B represents the improvement in pavement surface condition over time (n) if the expenditure level is increased from L_1 to L_2. The n_1 and n_2 represent pavement age at terminal pavement roughness (RN_T) for expenditure levels L_1 and L_2, respectively, and $(n_1 - n)$ is the increment in pavement service life due to the routine maintenance expenditure at level L_1. For expenditure level L_2, pavement service life increases by $(n_2 - n)$.

IDENTIFICATION OF TRAFFIC AND MAINTENANCE EXPENDITURE LEVELS

A data base was developed for pavement routine maintenance, pavement condition, and pavement characteristics of selected sections of Indiana highways (*7*). On the basis of earlier studies, maintenance activities were identified by pavement contract section units, which are smaller than the highway sections currently used by the Indiana Department of Highways (IDOH) (*1,3*). A contract section is the portion of highway pavement that is assigned to one contractor for a specific activity, such as resurfacing. The pavement characteristics within these sections are generally uniform. In contrast, a highway section may include a series of contract sections, each with different pavement characteristics.

The data base covered 10 out of 37 maintenance subdistricts in Indiana. It included two highway classes (Interstate and Other State Highways), three pavement types (flexible, rigid, and rigid with flexible overlay), and two climatic regions (north and south). In all, 550 contract sections were selected, including 126 sections in Interstate and 424 in Other State Highways (OSH). The 1984 and 1985 data on PCA Roadmeter roughness measurement (counts per mile) for each contract section were obtained from the computer files of IDOH's Division of Research and Training. The amount of routine maintenance applied between two dates of roughness measurements was determined from each subdistrict's crew day cards. Data on average daily traffic (ADT) and percentage of trucks were obtained from IDOH's Division of Planning.

Four routine maintenance activities were considered in this research: shallow patching, deep patching, sealing longitudinal cracks and joints, and sealing cracks. Pavement contract sections were grouped by the type of routine maintenance applied during the study period. It was determined that very few sections received only joint and crack sealing activities; these procedures were usually performed along with the other types of maintenance. Therefore, to include implicitly the effect of the expenditure level of different maintenance activities on pavement service life, the following two maintenance categories were used in the analysis:

1. Shallow and deep patching (*P*), and
2. Patching and joint and crack sealing (*PS*). (For flexible and overlaid pavements, PS means patching and crack sealing.)

The conceptual relationship between pavement roughness and pavement age, presented in Figure 2, was assumed valid for small variations in traffic loading. Therefore, both traffic and maintenance expenditure were divided into two levels—low and high—to examine the relationship separately for each

TABLE 1 CUTOFF VALUES BETWEEN LOW AND HIGH LEVELS OF MAINTENANCE EXPENDITURE AND TRAFFIC LOADING

Highway Class	Pavement Type	Maintenance Expenditure ($/lane-mile/year) Patching and Jt. & Crack Sealing	Maintenance Expenditure ($/lane-mile/year) Patching	Traffic ESAL (thousands)
Interstate	Rigid	165	80	400
	Overlaid	255	90	215
Other State Highways	Flexible	412	122	20
	Rigid	355	196	55
	Overlaid	268	102	35

traffic/maintenance level combination. Mean annual ESAL values were used as a measure of traffic loading and were computed on the basis of available ADT and truck percentage data. To represent both north and south regions in Indiana with adequate sample sizes, the cutoff value between the low and high traffic levels for each highway class/pavement type combination was computed as follows:

$$\text{Cutoff value} = (\text{Avg. ESAL in North} + \text{Avg. ESAL in South})/2$$

The same procedure was used to determine low and high expenditure levels for each maintenance category. The cutoff values between low and high levels of maintenance expenditure and traffic loading for each highway class/pavement type combination are listed in Table 1.

A preliminary analysis of pavement contract sections that did not receive maintenance work during the study period was conducted to investigate the possibility of developing zero-maintenance curves. The available information was insufficient to predict the effect of zero maintenance on pavement service life, so these contract sections were excluded from further analysis.

EFFECT OF PAVEMENT TYPE ON RELATIONSHIP BETWEEN PAVEMENT ROUGHNESS AND PAVEMENT AGE

As shown in Figure 2, the relationship between pavement roughness and age was assumed to be nonlinear. Since the definition of pavement roughness varies depending on whether the measuring system is response-type or pavement profile, this assumption needed to be tested. Therefore, the data were analyzed separately by maintenance category, climatic region, and highway class/pavement type combination. To evaluate the effects of traffic and maintenance expenditure levels, the data were further subdivided based on the following traffic/maintenance level combinations:

- LL—low maintenance expenditure, low traffic;
- LH—low maintenance expenditure, high traffic;
- HL—high maintenance expenditure, low traffic; and
- HH—high maintenance expenditure, high traffic.

No observations were available in some cells, especially for the Interstate highway class. Also, because very few contract sections were found, these cases were not considered in the analysis. Regression analysis was performed for the remaining traffic/maintenance level combinations using pavement roughness as the dependent variable and pavement age as the independent variable.

In most instances, the general goodness-of-fit represented by the coefficient of multiple determination (R^2) was used to select the best model. For each case, linear and nonlinear models were developed and the R^2 values of these models were compared. The relationship between pavement roughness and age was found to be more related to pavement type than to region, highway class, maintenance category, or traffic/maintenance level combination. Therefore, the following general regression models were adopted:

For flexible and rigid pavements:

$$\text{RN} = a + b(\text{age}) \tag{1}$$

For overlaid pavements:

$$\log_{10}(\text{RN}) = c + d(\text{age}) \tag{2}$$

where

RN = roughness measurement in 1985 (counts/mi),
age = pavement age since construction or resurfacing (yr), and
a, b, c, d = regression parameters.

From Equations 1 and 2, it can be stated that the relationship between pavement roughness and age was found linear for flexible and rigid pavements and nonlinear for overlaid pavements.

PREDICTION MODELS FOR EFFECTS OF ROUTINE MAINTENANCE EXPENDITURE LEVEL ON PAVEMENT SERVICE LIFE

Two general prediction models were developed to determine whether pavement age or total accumulated ESAL at resurfacing is a better representation of pavement service life. In addition to maintenance expenditure level and climatic region, pavement age and ESAL were considered in the first model and total accumulated ESAL was considered in the second model. (ESAL was included in the first model to test the effect of variations in annual traffic loading.) Pavement roughness was used as the dependent variable in both models. Furthermore, the models were developed by routine maintenance category and for each highway class/pavement type combination.

The two models were compared on the basis of two criteria:

1. The coefficient of multiple determination (R^2), and
2. The level of significance of pavement age and ΣESAL.

In general, a much higher R^2 was obtained for the first model than for the second. Pavement age was more significant than ΣESAL in all cases except Interstate rigid pavements. In many cases, especially for OSH pavements, ΣESAL was found not significant at a level of significance of $\alpha = 0.10$. Pavement age was therefore considered more suitable than ΣESAL to represent pavement service life. This conclusion was confirmed by the observation made by Schoenberger (*8*).

On the basis of this finding, linear prediction models were developed for flexible and rigid pavements and nonlinear models for overlaid pavements. To obtain the best models, the following steps were taken:

1. Insignificant models were excluded on the basis of a level of significance, with $\alpha = 0.05$.
2. For OSH pavements, separate models were developed for each region because R^2 values of these models were found not high when region was used as a dummy variable. Region was retained as a dummy variable in Interstate models because of the limited number of observations and the limited amount of routine maintenance work on these pavements regardless of region.
3. If a model was found significant but the variable of expenditure level was not found significant at $\alpha = 0.05$, then the model was eliminated.
4. The effect of patching was found not significant in all models. In some cases, mainly in rigid pavement models, patching expenditure level was found positively correlated with pavement roughness.
5. The remaining significant models in which ESAL was found insignificant at $\alpha = 0.10$ were reexamined after excluding this variable.

On the basis of these steps, the following regression models were adopted:

For Interstate overlaid pavements:

$$\log_{10}(\text{RN}) = 2.9 - 0.002\ \text{PS} + 0.19\ \text{age} - 0.004\ \text{ESAL} + 0.124\ Z \qquad (3)$$

For OSH flexible pavements—north:

$$\text{RN} = 1{,}551 - 1.23\ \text{PS} + 57.1\ \text{age} - 15\ \text{ESAL} \qquad (4)$$

For OSH overlaid pavements—north:

$$\log_{10}(\text{RN}) = 2.81 - 0.0005\ \text{PS} + 0.047\ \text{age} \qquad (5)$$

where

PS = patching and joint and crack sealing expenditure level ($/lane-mi/yr),
ESAL = mean annual equivalent single-axle load (thousands), and
Z = dummy variable representing climatic region: 0 for north and 1 for south.

Table 2 provides a summary of the regression characteristics of the models presented in Equations 3 to 5. The models in which ESAL was found significant, Models 3 and 4, were further investigated. After omitting the ESAL variable from both models, R^2 in the Interstate overlaid model decreased from 0.95 to 0.91 and R^2 in the OSH flexible model decreased from 0.53 to 0.42. The decrease in R^2 in the Interstate overlaid model is so much less than that of the OSH flexible model because, as shown in Table 2, ESAL was found more significant in Equation 4 than in Equation 3.

TABLE 2 STATISTICAL CHARACTERISTICS OF PAVEMENT SERVICE LIFE PREDICTION MODELS (EQUATIONS 3–5)

Criterion	Eq. 3	Eq. 4	Eq. 5
Number of Observations	10	19	19
Coeff. of Determination (R^2)	0.95	0.53	0.77
Adjusted Coeff. (adj. R^2)	0.93	0.47	0.76
Linearity Test			
F Value	24.32	5.67	26.95
α Level	0.002	0.008	0
Significance Test for Coefficients			
PS			
F Value	15.15	5.18	2.89
α Level	0.012	0.040	0.100
Age			
F Value	14.81	10.98	48.55
α Level	0.012	0.005	0
ESAL			
F Value	4.46	3.71	-
α Level	0.090	0.070	-
Region			
F Valuc	0.80	-	-
α Level	0.410	-	-

On the basis of these findings, separate models were developed for low and high traffic loading levels for OSH flexible pavements in the north. ESAL was excluded from the Interstate overlaid model because of the limited number of observations and because eliminating ESAL did not significantly affect R^2. The resulting models are given in Equations 6 to 8:

For Interstate overlaid pavements:

$$\log_{10}(\text{RN}) = 2.5 - 0.001\ \text{PS} + 0.09\ \text{age} - 0.156\ Z \qquad (6)$$

For OSH flexible pavements—low traffic level—north:

$$\text{RN} = 1{,}521 - 1.24\ \text{PS} + 48\ \text{age} \qquad (7)$$

For OSH flexible pavements—high traffic level—north:

$$\text{RN} = 497 - 0.45\ \text{PS} + 85\ \text{age} \qquad (8)$$

The statistical characteristics of the models presented in Equations 6 to 8 are given in Table 3. Equations 5 to 8 were employed to relate the time of resurfacing to routine maintenance expenditure level.

APPLICATION OF PAVEMENT SERVICE LIFE PREDICTION MODELS

Knowledge of the effects of routine maintenance on pavement service life is important to the management of highway pavements at both network and project levels. One application of the prediction models developed in this research was esti-

TABLE 3 STATISTICAL CHARACTERISTICS OF PAVEMENT SERVICE LIFE PREDICTION MODELS (EQUATIONS 6–8)

Criterion	Eq. 6	Eq. 7	Eq. 8
Number of Observations	10	13	6
Coeff. of Determination (R^2)	0.91	0.41	0.75
Adjusted Coeff. (adj. R^2)	0.88	0.36	0.68
Linearity Test			
F Value	19.63	3.49	4.39
α Level	0.002	0.07	0.13
Significance Test for Coefficients			
PS			
F Value	8.31	3.34	0.29
α Level	0.028	0.098	0.63
Age			
F Value	29.39	4.77	8.48
α Level	0.002	0.054	0.06
Region			
F Value	8.85	-	-
α Level	0.025	-	-

mation of the need for resurfacing under different routine maintenance expenditure levels. As shown in Figure 2, pavements need resurfacing when surface roughness reaches the terminal value. Terminal roughness (RN_T) can be defined as the roughness level at which a pavement section's serviceability is too low and, hence, the pavement is in need of improvement.

Earlier studies (*9,10*) indicate that PSI values of 2.0 for secondary roads and 2.5 for Interstate and primary highways can be considered minimum values of acceptable pavement serviceability. In this research, a terminal serviceability index of 2.5 was used for Interstate pavements and 2.2 for OSH pavements.

Three successive studies were conducted by Purdue University and IDOH (*11, 12, 13*) to establish a comprehensive model of statistical correlation between Roadmeter roughness numbers and PSI for the Indiana state highway system. The results of this research are given in Equations 9 and 10:

For flexible and overlaid pavements:

$$PSI = 8.72 - 1.96633 * \text{Log}_{10}(RN) \quad (9)$$

$$r^2 = 0.71$$

For rigid pavement:

$$PSI = 11.73 - 2.83369 * \text{Log}_{10}(RN) \quad (10)$$

$$r^2 = 0.68$$

where r^2 equals coefficient of simple determination.

The suggested terminal serviceability indices were used in Equations 9 and 10 to determine the terminal roughness values. The results were 1,460 counts/mi for Interstate overlaid pavements, 2,070 for OSH flexible and overlaid pavements, 1,808 for Interstate rigid pavements, and 2,307 for OSH rigid pavements. Since prediction models were not developed for rigid pavements, only the first two terminal values were used to determine the time of resurfacing or pavement improvement.

The prediction models in Equations 5 to 8 were used to compute pavement roughness under low and high PS expenditure levels and for different pavement ages. In these models, $200 and $300/lane-mi/yr were selected to represent low and high expenditure levels, respectively, for Interstate overlaid pavements. Because routine maintenance expenditure level on OSH pavements was found higher than that on Interstate pavements, $300 and $600/lane-mi/yr were selected to represent low and high PS expenditure levels for OSH flexible pavements. The corresponding values for OSH overlaid pavements were $150 and $450/lane-mi/yr. The terminal roughness values were then used to determine the pavement service life or resurfacing timing under each expenditure level.

The effects of routine maintenance expenditure level on pavement roughness and consequently on resurfacing decisions can be best demonstrated through the graphical presentations in Figures 3 to 7. These figures clearly show that pavement service life increases as the maintenance expenditure level increases. However, the amount of this increase varies. For example, as shown in Figures 3 and 7, if PS expenditure increases from low to high, the increase in service life for Interstate and OSH overlaid pavements is about 1 yr and 3.3 yr, respectively. It should be pointed out that service lives of Interstate and OSH pavements cannot be directly compared because of the difference in traffic and maintenance expenditure levels.

The results can be used to evaluate the effect of the region on resurfacing needs. It was found that pavements in the northern region need resurfacing earlier than pavements in the southern region, possibly due to the more severe weather in the northern region. At a low expenditure level ($200/lane-mi/yr), Interstate overlaid pavements need resurfacing after 9.7 yr in the north and 11.4 yr in the south (see Figures 3 and 4).

Figures 5 and 6 demonstrate the effect of traffic loading on expenditure levels. If PS expenditure increases from $300 to $600/lane-mi/yr, the increase in service life of OSH flexible pavements for low traffic loading is approximately 8 yr, using Equation 7. The corresponding value for high traffic loading is 1.6 yr, using Equation 8. The difference indicates the aggregated effect of higher traffic loading on pavement surface condition. Consequently, highly travelled OSH pavements require more frequent maintenance or resurfacing than those with low traffic loading.

To indicate the variability of predicted pavement service life values, prediction bands were developed for the effect of each PS expenditure level in Figures 3 to 7. The prediction bands were obtained by adding and subtracting one standard error of estimates of the model parameters. In general, the prediction bands were wide and overlapped in the same figure. Moreover, their width increased as pavement age increased; in other words, the models became less predictable as pavement age increased. Consequently, the results cannot be treated as entirely conclusive.

The results presented in this paper are applicable at an aggregated network level; they cannot be used in the actual scheduling of individual resurfacing projects. Resurfacing decisions for individual sections should be based on a comparison of resurfacing cost and routine maintenance cost along

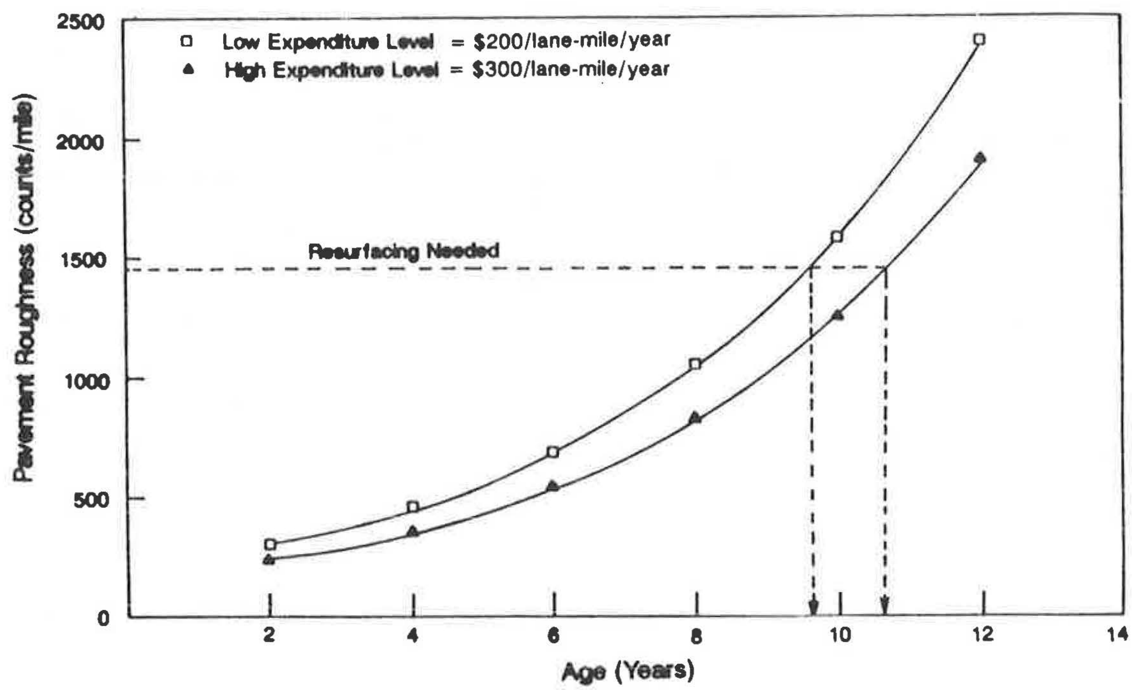

FIGURE 3 Estimated effect of patching and crack sealing expenditure level on service life of Interstate overlaid pavement in northern region (Equation 6).

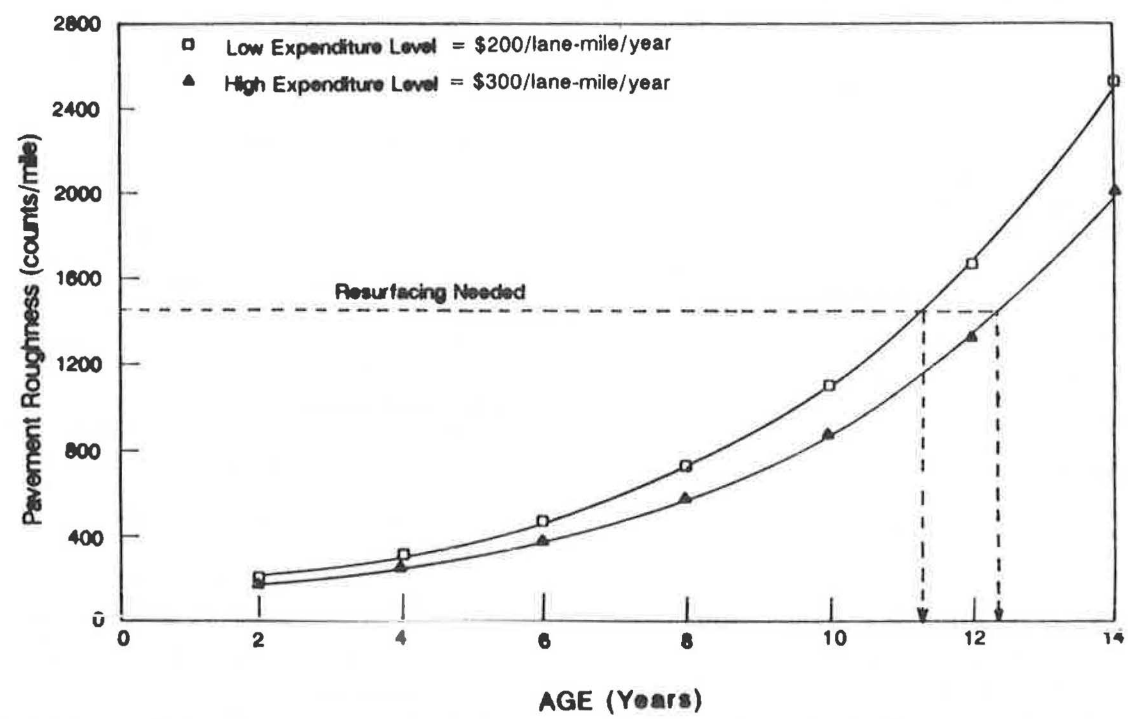

FIGURE 4 Estimated effect of patching and crack sealing expenditure level on service life of Interstate overlaid pavement in southern region (Equation 6).

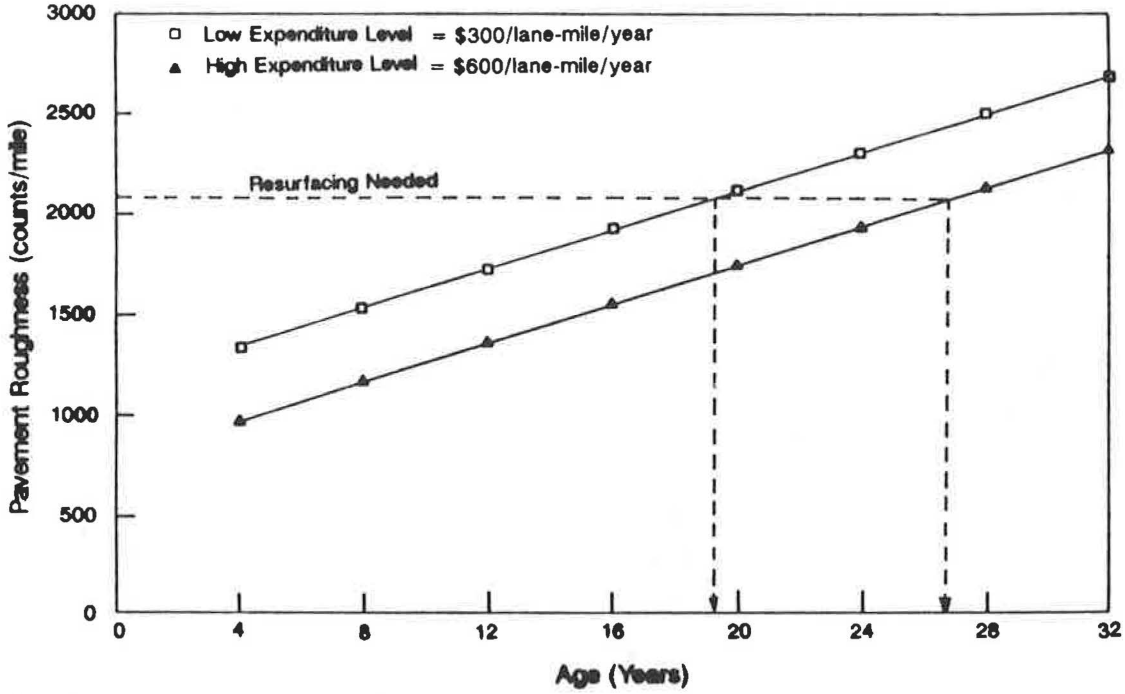

FIGURE 5 Estimated effect of patching and crack sealing expenditure level on service life of OSH flexible pavement in northern region—low traffic level (Equation 7).

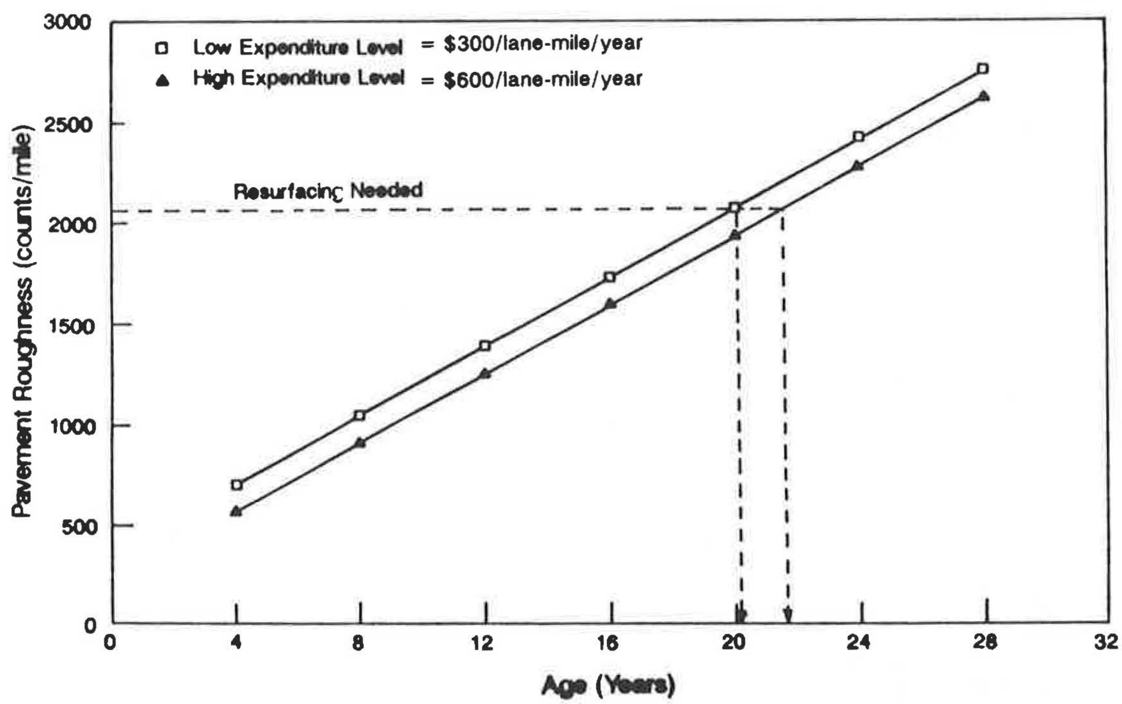

FIGURE 6 Estimated effect of patching and crack sealing expenditure level on service life of OSH flexible pavement in northern region—high traffic level (Equation 8).

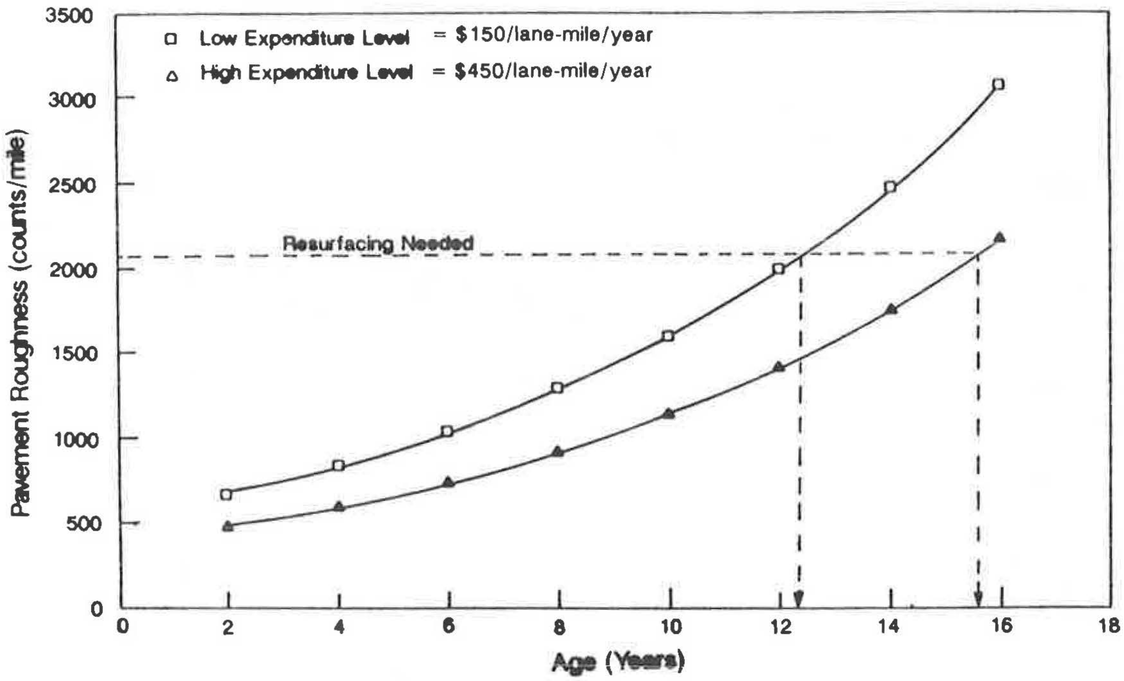

FIGURE 7 Estimated effect of patching and crack sealing expenditure level on service life of OSH overlaid pavement in northern region (Equation 5).

with a consideration of appropriate resurfacing design procedures. The prediction models can provide guidance in the preliminary analysis of pavement life-cycle costing.

To improve the prediction models developed in this research, the following factors should be considered:

- Thickness of overlay,
- Flexible pavement structural capacity,
- Rigid pavement slab thickness,
- Rigid pavement type (jointed plain concrete, jointed reinforced concrete, or continuous reinforced concrete), and
- Resurfacing cost and resurfacing design procedures.

SUMMARY AND CONCLUSIONS

The effects of routine maintenance expenditure on pavement service life were examined in this paper. The relationship between pavement roughness and pavement age was investigated under different traffic/maintenance expenditure level combinations. The relationship was found linear for flexible and rigid pavements and nonlinear for overlaid pavements. It was also determined that, for a small range of traffic loading, pavement age was a better variable than total accumulated ESAL to explain variations in pavement roughness.

Prediction models were used to examine the effect of maintenance expenditure level on pavement service life. The patching expenditure level was found insignificant in all models. Models in which mean annual ESAL was highly significant were reexamined, and separate models were developed for low and high traffic levels. The results demonstrated that resurfacing can be deferred or postponed by increasing the maintenance expenditure level. Routine maintenance was more effective in increasing the service life of OSH pavements than Interstate pavements. Also, it was found that pavements in the northern region needed resurfacing earlier than those in the southern region.

The prediction bands of the models were found to be wide, and their width increased as pavement age increased. Therefore, the results of the models cannot be treated as entirely conclusive. The models presented are applicable only to network-level decision making and should not be used to make resurfacing decisions for individual sections. To improve pavement service life prediction models, factors such as pavement thickness and cost of resurfacing should also be considered.

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A Combined Life Cycle Cost and Performance Approach for Selection of Optimal Flexible Pavement Strategies

Omar Elfarouk Azmy, Essam A. Sharaf, and Hani L. Lotfi

Pavements are complex structures subjected to diverse loading and environmental conditions. Pavement structural design should handle this complexity in a rational way. The evaluation procedure used must enable the selection of a pavement design option that provides adequate performance as well as adequate distribution of life cycle cost. The research described in this paper was based on the development of an evaluation model that incorporated future maintenance activities in the initial design concepts to achieve structural safety, riding comfort, and economical costs during the life cycle span of flexible pavement structures, which is the predominant type of pavement in Egypt. Due to the complexity of the problem and the amount of data to be analyzed, a computer program was developed to calculate the life cycle costs of different flexible pavement design alternatives. A second program was developed to transform the first program from a cost model to a decision support model. This program uses two decision support models to select the design option that achieves the best combination of cost, performance, and time for the considered maintenance policy. Other models are provided to help the decision maker analyze the information and make the optimal selection among all possible maintenance policies.

The road network in Egypt includes about 27 000 km of roads, of which approximately 13 000 km are paved. The Egyptian Roads and Bridges Authority (RBA) is currently improving the road maintenance and rehabilitation standards of the paved roads. Due to these efforts, pavement condition has improved; it is estimated that about 60 percent of the network is now in good condition (including most of the dual carriageways), 30 percent is in fair condition, and 10 percent is in poor condition (*1*).

These improvements are a result of the maintenance program recommended by the Development Research and Technological Planning Center (DRTPC) of Cairo University for the analysis period (1982 to 1991) (*2–5*). To facilitate the improvements, RBA established a project to review and evaluate existing manpower and training facilities so a systematic approach could be developed to meet future manpower and training demands (*6*). It is believed that a practical training management system can have a significant impact on improved standards of performance, increased productivity, and maximum cost effectiveness of highway maintenance.

Although improvements have been made, traffic volumes have doubled in the past 4 to 5 yr, which accelerates the rate of pavement performance loss. In addition, severe economic restraints have been imposed on the local highway network due to decreased revenues, high inflation, and an increase in the need for maintenance and rehabilitation on the existing network. Also, in spite of the new program, pavement-related activities (design, construction, maintenance, and rehabilitation) are still being conducted on the basis of subjective assessment of engineering experience.

RESEARCH OBJECTIVES

The primary objective of this research was to incorporate future maintenance activities in initial pavement design concepts to achieve structural safety, riding comfort, and economical costs during the life cycle span of flexible pavement structures. Accordingly, the second objective was to develop an evaluation model to help RBA evaluate and select the optimal combination of pavement structural design and maintenance policy to produce flexible pavement structure with adequate performance as well as adequate distribution of life cycle cost.

DESIGN AND DETERMINATION OF LIFE CYCLE COST

To achieve the research objectives, the following two tasks were completed:

1. An initial design/overlay procedure that included the development of a model to forecast serviceability/time (traffic repetitions) on any given pavement structure during the analysis period. This procedure involved serviceability predictions as well as prediction capabilities for overlaid sections. In the development of this task, the AASHTO pavement design-analysis concept (*7*) was used as the initial methodology because of its broad experience base and general acceptance in Egypt. Some modifications were made regarding pavement strength coefficients and subgrade effects on this strength as determined by the multilayered elastic theory concepts. In addition, the remaining life concept was used in association with the AASHTO design equation to allow serviceability/time to be forecast over the life of the overlaid pavement structure.
2. The establishment of costing models to estimate the pavement's cost and design life. This task included the establishment of the following models:

O. E. Azmy, Engineering Consultant Group, Cairo, Egypt. E. A. Sharaf and H. L. Lotfi, Cairo University, Giza, Egypt.

— A construction cost model using current unit costs of the selected materials;
— An overlay cost model using current unit costs of the selected materials and adapted models that estimate leveling costs and traffic handling costs at the time of overlay;
— A salvage cost estimation;
— A routine maintenance cost responsive model, designed to assist in the maintenance management system (MMS) as a part of the overall pavement management system (PMS), that can be defined as "a technique or operational methodology for managing, directing, and controlling maintenance resources for optimal benefits" (*8*) by providing desired maintenance policies based on specific standards;
— A user cost model to predict the added user costs associated with overlay construction; and
— A user cost model based on locally available data to predict the running user costs from normal operation of specific two-lane roads, in addition to other sets of models for other types of roads.

LIFE CYCLE COST PROGRAM

The models described above were aggregated to develop the required life cycle cost (LCC) computer program. An original version developed for the Maryland State Highway Administration (*9*) was modified to reflect Egyptian conditions.

The LCC program contains two subsystems:

1. Structural design/overlay (based on AASHTO concepts), and
2. Highway cost (based on local and internationally adapted models developed for construction, routine maintenance, and user costs).

The program can consider an initial construction (and overlay) problem or an overlay (over an existing pavement structure) problem only.

Any combination of inflation and discount rates can be considered, and the equivalent single-axle load (ESAL) of any traffic record can be computed. Routine maintenance costs can be calculated for any number of responsive maintenance policies. Also, a single model for computing costs of routine maintenance can be used. User costs are calculated for different area and road types, and updating facilities are included for all models.

DECISION SUPPORT METHODOLOGY

As part of this research, the benefit of the LCC program was generalized from a cost model to a decision support model. A decision support model not only generates the needed information but helps the decision maker analyze the information and make the optimal selection. An evaluation procedure must be followed to determine the desirability of the different alternative strategies and to provide the information to the decision maker in a useful, comprehensive form. Furthermore, the evaluation procedure should be tailored to the agency's objectives and goals.

Selection Within One Maintenance Policy

Optimization techniques that ensure a least cost or maximum benefit/cost ratio for each agency should be considered while meeting minimum condition management constraints. As the PMS is used, the identification of future budget needs is likely to be a significant step toward allocating the current year's budget. A comparison between the agency's actual cash flow and the expected cash flow for each alternative will economically finalize the selection of the optimal alternative.

Figure 1 illustrates the performance curve for any given pavement. Costing and serviceability values are shown on the curve. As can be seen, some costs result from constructing the new pavement (or rehabilitating the old pavement) and some from keeping the pavement in good condition. Thus, utility (U) can be divided into two phases. The first phase

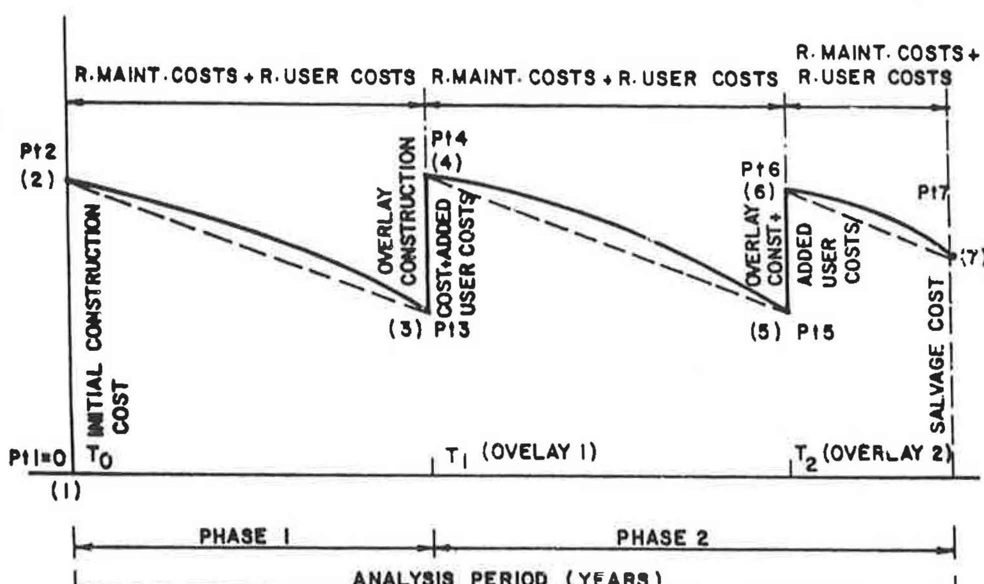

FIGURE 1 Performance curve and costing items (initial construction).

includes initial construction, routine maintenance, and running user costs. Through this phase, serviceability is decreased from P_{t2} to P_{t3}. Thus the utility of this phase can be presented by the sum of (initial construction costs + routine maintenance costs + running user costs) divided by the drop in the level of serviceability ($P_{t2} - P_{t3}$) for time T_1. In the next phase, several utilities may be incurred when each overlay is constructed at time T_2. At the end of the second phase, negative salvage cost is considered as a negative utility. On the basis of Figure 1, the following model can be obtained, assuming a linear drop in serviceabilities and a linear relationship between utility and time:

$$U_j = \left[\frac{C(I + M + R)/T_1}{P_{t(k)} - P_{t(k+1)}} + \sum_{i=1}^{n-1} \frac{C_i(O + M + R)/T_i}{P_{t(k)} - P_{t(k+1)}} + \frac{(C_n(O + M + R) - SC_j)/T_n}{P_{t(k)} - P_{t(k+1)}} \right]_j \quad (1)$$

where

U_j = utility of alternative j;
$C(I + M + R)$ = initial construction costs + routine maintenance costs + running user costs, calculated for alternative j during the intercepted time (T_1);
$P_{t(k)}$ and $P_{t(k+1)}$ = two successive serviceability levels measured at the beginning and end of the regarded intercepted time;
$C_i(O + M + R)$ = overlay construction costs + routine maintenance costs + running user costs, calculated for overlay i executed in alternative j during an intercepted time (T_i);
$C_n(O + M + R)$ = same as above but calculated for the last overlay (n) executed in alternative j during the intercepted time (T_n); and
SC_j = salvage cost value of the analysis period, which may be positive, zero, or negative.

On the basis of the LCC and cost/performance models, the considered number of alternatives was limited to two (or possibly one):

- One representing the least cost/time, and
- One (which may be the same) representing the highest cost/performance ratio.

If two alternatives are available, the decision maker will have to select one of these two options. This selection should be based on a technique known as time stream analysis, by which the second decision model is developed.

The AASHTO performance equation (*7*) illustrates that time is an important parameter affecting the performance of flexible pavements. On the basis of this concept, time stream analysis should be performed for each alternative to measure the effect of time on the performance of the considered pavement, in other words, to measure how the degree of desirability for the given strategies varies with time.

Because the number of performance curves may vary from one alternative to another, the average value of the performance/time ratio is considered for each alternative as follows:

$$PT_j = \sum_{i=1}^{n} PT_{i,j}/n \quad (2)$$

where

PT_j = average performance/time value for alternative j,
$PT_{i,j}$ = performance/time value for stage i of alternative j, and
n = total number of stages included in alternative j.

The term $PT_{i,j}$ can be determined by measuring the inclination of the chord of performance curve (stage), i.e., chords 2–3, 4–5, or 6–7 in Figure 1. Thus,

$$PT_{i,j} = (P_{t(k)} - P_{t(k+1)})_{i,j}/T_{i,j} \quad (3)$$

where

$(P_{t(k)} - P_{t(k+1)})_{i,j}$ = drop in serviceabilities with ranks k and $k + 1$ for stage i of alternative j, and
$T_{i,j}$ = intercepted time between the above serviceabilities, i.e., the time of stage i of alternative j.

Consequently, the lower the value of PT_j, the better the performance/time ratio. This concept can be used to choose between the two alternatives that were selected according to the minimal cost/time ratio and maximum cost/performance ratio. The selection of the final alternative is based on a summary module in which the two strategies are ranked according to the number of times they have been chosen. Thus, the first rank is given to the strategy that has been chosen twice (a score of 2), while the second rank is given to the strategy that has been chosen once (a score of 1).

Final Selection Among Policies

The LCC program provides the user with a set of feasible strategies for one maintenance policy or for a group of suggested maintenance policies. This group can be executed consecutively for the same ordinary data or for various ordinary data.

Maintenance policies are applied according to the regulations of the agency's MMS. These systems are used by agency directors and field managers to plan, control, and evaluate road maintenance programs. The basic components of an MMS include performance standards, inventory of maintenance features, budgeting, scheduling, and a management information reporting process. Because many factors influence the performance of an agency's MMS, the level of certainty decreases when comparing several maintenance policies. Moreover, the level of uncertainty increases with the following parameters (*10*):

- The length of the planning horizon,
- The amount of resources committed for a given course of action, and
- The difficulty of reversing a decision once implementation begins.

When the decision making is done by the same user (agency), then these concepts of uncertainty can be reduced and limited. This limitation must be directed by factors that are beyond the agency's control. Therefore, the effective measure that should be considered is the financial measure.

In the life cycle cost analysis of pavement, the financial

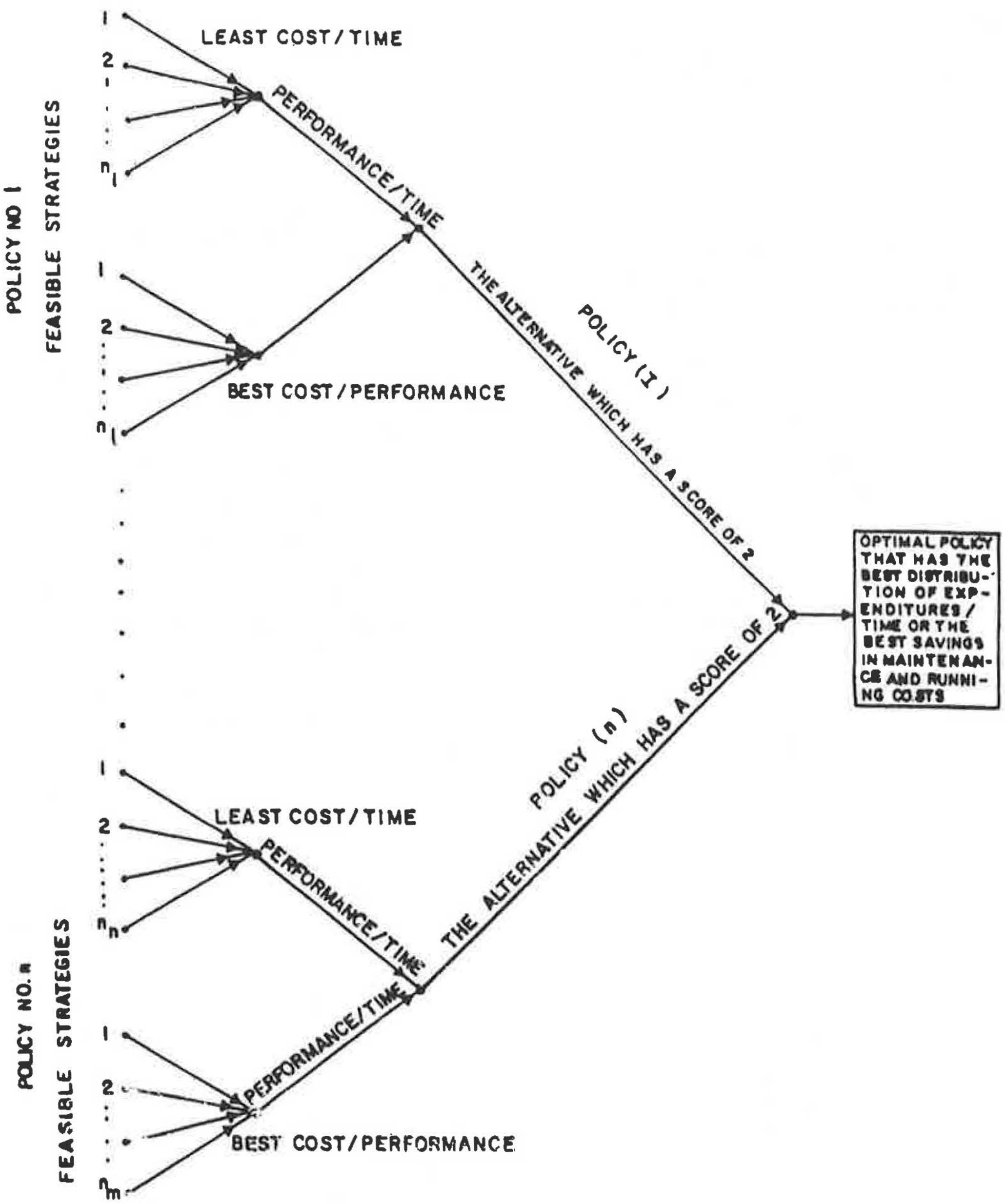

FIGURE 2 Decision optimization tree.

measure is affected by time. Figure 2 summarizes the steps in the decision optimization tree for a set of maintenance policies provided by the LCC program.

It can be concluded that the timing of various costs is an important element in choosing a pavement maintenance policy. A policy in which the costs are evenly distributed and the benefits occur in an early life cycle stage may be preferable over one in which the initial costs constitute the bulk of the expenditures. Because of this, a time stream analysis component should be used to illustrate the differences in the timing of costs and benefits among the available policies. The uniformity of expenditures can then be defined to achieve an adequate balance between the budget and the life cycle expenditures of a road. By using the LCC program, the life cycle cost of a road can be determined for the expected future phases of the road's anticipated useful life span. If the initial budget can be invested at a certain interest rate, then a uniform rate of return can be expected. Consequently, future returns and expenditures will be uniformly distributed over the useful life of the road, and budget deficiencies can be limited.

When a new road is built or an existing road is improved, three different effects can be expected (*11*):

1. A redistribution of traffic flows between existing roads and the new road and the generation of new traffic flows,
2. A transformation of the production structure in the area crossed by the road, and
3. Social consequences linked to the increased access to public facilities enjoyed by the area's population.

In most developing countries, indirect road benefits are related primarily to the redistribution of traffic flows and only marginally to development resulting from the transformation of the area's production structure. In other words, indirect road benefits can be regarded as amounting to user savings and road maintenance savings (*11*). These two types of savings constitute a large part of total road benefits; in the evaluation of a road project, they can be safely assumed to account for their entirety.

In this paper, several maintenance policies are evaluated for one project (i.e., project level). Thus, the above two types of savings are a suitable tool in the final evaluation among the suggested policies. To establish this concept, the policy that contains the maximum sum of maintenance and running user costs should be determined first. Second, the maintenance and user costs for the remaining policies should be subtracted from the values of this policy. The relative saving/cost ratios can be determined as follows:

$$B_{i/j} = \frac{MC_i - MC_j}{MC_j} + \frac{RC_i - RC_j}{RC_j} \tag{4}$$

where

$B_{i/j}$ = savings (benefits) in maintenance and running user costs obtained when using policy i with respect to policy j, which represents the maximum sum of the two costs;
MC_i = total maintenance costs of policy i;
MC_j = total maintenance costs of policy j;
RC_i = total running costs of policy i; and
RC_j = total running costs of policy j.

The candidate policies can be ranked according to the values of benefits in a descending form. Consequently, two evaluation tools are available for each policy: the uniformity of expenditures and the savings in routine maintenance and running user costs. The optimal decision must consider the policies of the agency and the circumstances of the particular project. Therefore, weights should be assigned to the above measures so the maintenance policies can be rated and the optimal pavement strategy can be selected for the project. It should be noted that this procedure is not applicable when computing initial costs only, since in this case the least-cost policy would be the optimal one.

Decision Support Program

To transfer the LCC program from a cost model to a decision support model, a decision support program (DSP) was developed. The previous decision models are used in this program. Figure 3 shows the flow chart of the DSP.

As shown in the figure, the DSP can compute the data needed to make the final selection among the given maintenance policies. A number of items are determined and printed in the DSP report:

- The discrete costs and the percentages of cumulative costs for each policy,
- The equation of the least-squares line and the equation of the straight line for the percentages of cumulative costs,
- The existed median for the first line and the ideal median for the second line,
- The percentage error between the two medians, and
- The total routine maintenance costs and running user costs.

These factors can then be used to choose the optimal policy, as demonstrated by the following sample problem.

SAMPLE PROBLEM

An example was constructed to demonstrate the method. In this example, it was assumed that in 1987, a flexible-pavement four-lane (divided) rural highway was to be constructed to accommodate traffic for a 30-year period. Using the LCC and DSP programs, the 10 best alternatives were to be selected for five suggested maintenance policies based on the initial, overlay, routine, maintenance, added user, and running user costs. A discount rate of 24 percent and an inflation rate of 19 percent were used in the economic analysis. The prevailing rate of exchange during 1987 was 2.20 £E/$. The traffic expected over the 30-year analysis period is as follows:

Average daily traffic = 10,000 vpd (both directions)
Directional split = 50 percent
Percent trucks on road = 15 percent
Traffic growth rate per year = 10 percent
Traffic count base year = 1986
ESAL/100 trucks = 0.64

Based on 1986 rates, it was found that added user costs have increased by an average value of 10 percent. Running user costs have increased by an average value of 20 percent during 1987.

The design California Bearing Ratio (CBR) for the subgrade is 1.5, and the regional factor is 0.4. The suggested material/layer combinations for the initial construction are shown in Table 1. All feasible alternatives must have a minimum of three layers. Table 1 also shows the suggested combinations for overlay construction, in which all feasible alternatives must have at least one overlay.

The suggested maintenance policies (1001, 1002, 1003, 1004, and 1005) are shown in Table 2. The ranges of terminal serviceability, the minimum times required for the overlays, and the minimum times required between any two successive overlays are provided for each maintenance policy.

The DSP output shows that alternative 1 is optimal for policies 1001, 1003, and 1004, while alternative 9 is best for policy 1002. Policy 1005 was excluded because no feasible strategies could be obtained for it. The alternatives selected achieve the least life cycle cost, the best cost/performance relationship, and the best performance/time relationship for their related maintenance policies. The values of discrete costs, percentages of cumulative costs, existing cost/time relationship, existing median, ideal cost/time relationship, ideal median, percentage error, total routine maintenance costs, and total running user costs are also given for each optimal policy.

Figures 4a and 4b show the discrete time streams and the cumulative time streams, respectively, for the five maintenance policies based on the results of the output.

By comparing the percentage of error for the median of each relationship with the median of its related linear relationship, the five policies can be ranked in the following ascending order:

1. Policy 1003,
2. Policy 1004,
3. Policy 1001,
4. Policy 1002, and
5. Policy 1005 (excluded).

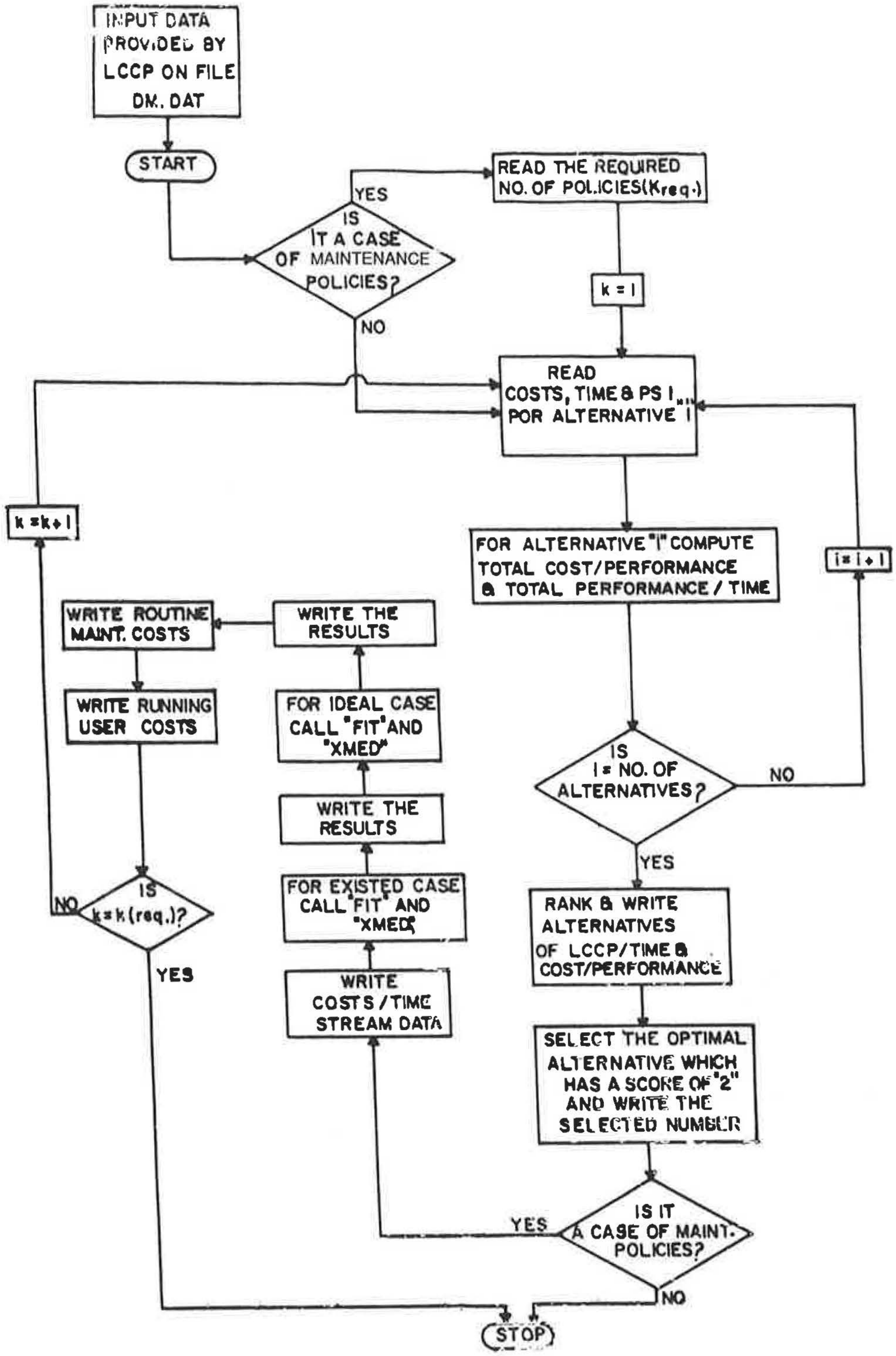

FIGURE 3 Flow chart of DSP.

TABLE 1 MATERIAL/LAYER COMBINATIONS FOR INITIAL AND OVERLAY CONSTRUCTIONS (SAMPLE PROBLEM)

Construction Type	Material ID No.	Material Name	Construction Cost (£E/m³)	Layer No.	Layer Name	Coefficient (a)	Coefficient (b)	Allowable Thickness (mm) Minimum	Maximum	Increment
Initial	10	Asphalt concrete	100.0	1	Surface course	0.45	0.00	40.0	50.0	10.0
	22	Premix	80.0	2	Binder course	0.40	0.00	100.0	100.0	10.0[a]
	30	Crushed stone	20.0	3	Base course	0.14	0.00	200.0	300.0	50.0
	50	Crushed stone (bigger size)	20.0	4	Subbase course	0.11	0.00	300.0	400.0	50.0
Overlay	10	Asphalt concrete	100.0	1	Surface overlay	0.45	0.00	40.0	40.0	10.0[a]
	22	Premix	80.0	2	Base overlay	0.40	0.00	50.0	100.0	25.0

[a]To be assumed greater than 0.0.

TABLE 2 SUGGESTED RESPONSIVE MAINTENANCE POLICIES (SAMPLE PROBLEM)

Description	Policy No. 1	2	3	4	5
Code number	1001	1002	1003	1004	1005
Base type[a]	GR	GR	GR	GR	GR
Patching of unpatched cracks (%)	50.0	65.0	75.0	85.0	90.0
Maximum patched area (m²)	100.00	80.0	60.0	50.0	30.0
Patching unit cost[b] (£E/m²)	3.50	3.50	3.50	3.50	3.50
Type of surface dressing[c]	PR	AC	AC	PR	PR
Percentage of cracking and patching in the road (%)	8.0	10.0	20.0	30.0	40.0
Minimum years/one dressing	2.0	3.0	4.0	5.0	6.0
Maximum years/one dressing	5.0	6.5	7.0	8.0	9.0
Maximum analysis period/one dressing (yr)	20.0	25.0	25.0	30.0	30.0
Number of layers/one dressing	2	1	2	1	1
Unit cost/dressing (£E/m²/layer)	1.25	1.50	1.50	1.25	1.25
Unit lump sum cost of other routine maintenance activities (£E/km/yr)[d]	0.0	0.0	0.0	0.0	0.0
Minimum time for first overlay (yr)	5.0	10.0	15.0	15.0	20.0
Minimum time between overlays (yr)	10.0	10.0	7.5	5.0	5.0
Pt (min.)	2.5	2.5	2.4	1.8	2.5
Pt (max.)	3.0	3.0	3.0	2.4	3.0
Pt (increment)	0.50	0.50	0.60	0.6	0.5

[a]GR = granular base.
[b]Unit cost is divided: 50% skin patching and 50% deep patching.
[c]PR = premix; AC = asphalt concrete.
[d]This cost is considered to be negligible.

On the other hand, on the basis of the largest amount of savings obtained by applying Equation 4, the policies can be ranked in the following descending order:

1. Policy 1002,
2. Policy 1001,
3. Policy 1004,
4. Policy 1003, and
5. Policy 1005 (excluded)

Thus, policy 1003 gives the best uniformity of expenditures, while policy 1002 gives the best savings (benefits) in routine maintenance and running user costs.

If, for example, the agency is interested more in the concept of uniformity of expenditures than in the concept of savings, then the second concept would have a weight of 0.4 if the first had a weight of 0.6. In addition, the following ratings can be assumed for the policies according to their ranks as included in the first concept:

1. Policy 1003 = 100,
2. Policy 1004 = 75,
3. Policy 1001 = 50, and
4. Policy 1002 = 25.

The following ratings can be assumed for the policies according to their ranks as included in the second concept:

1. Policy 1002 = 100,
2. Policy 1001 = 75,
3. Policy 1004 = 50, and
4. Policy 1003 = 25.

Consequently, the following function can be used to provide a single aggregate desirability measure for the preferred policies (*9*):

$$U_i = \sum_{j=1}^{m} e_{i,j}\, w_j,\ i = 1\ (1)\ m \tag{5}$$

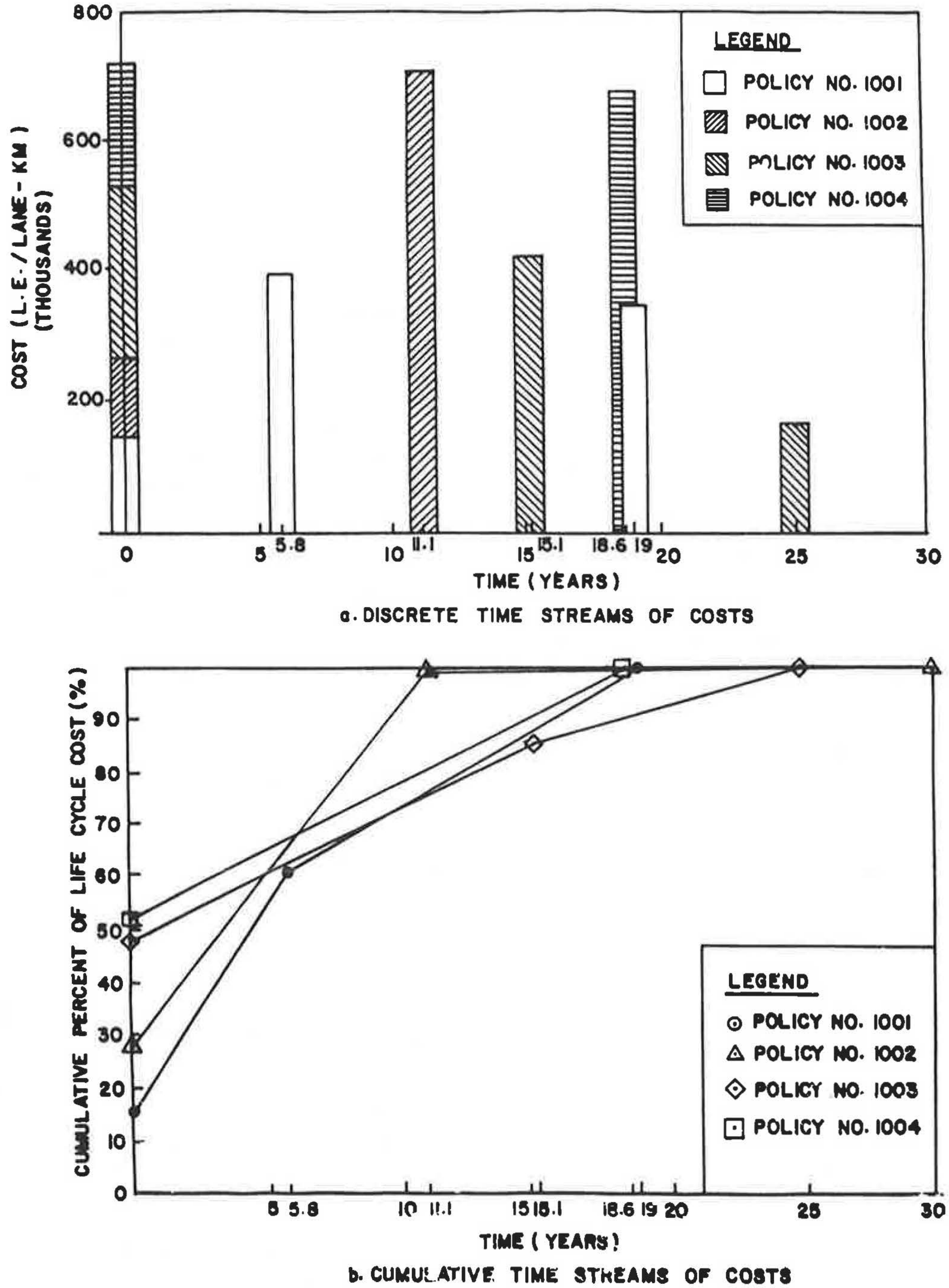

FIGURE 4 Distribution of current life cycle costs (sample problem).

TABLE 3 DECISION-MAKING REPORTS AND FINANCIAL DATA (SAMPLE PROBLEM) POLICY 1003

Ranking	
LCC	Cost/Performance
1	1
2	2
3	3
4	4
5	5
6	6
7	7
8	8
9	10
10	9

NOTE: Alternative 1 is optimal for Policy 1003.

TABLE 4 FINANCIAL DATA OF POLICY 1003

Time (yr)	Costs (£E/Lane-km) Discrete	Cumulative	Percent
.0	539977.7	539977.7	47.71
15.1	423133.0	963110.8	85.09
25.0	168755.5	1131866.0	100.00
30.0	−5525.7	1126340.0	100.00

NOTE: The existing relation is Cost (%) = 51.41 1.82 * time, R^2 = .9458; existing median = 17.52 years. The ideal relation is Cost (%) = 47.94 1.74 * time, R^2 = 1.0000; ideal median = 17.56 years. Percentage error = −.21634%; Total routine maintenance costs = 1986.2; Total running user costs = 956686.2.

where

U_i = summary score of strategy (or policy) i,
$e_{i,j}$ = rating of strategy (or policy) i with respect to measure j, and
w_j = weight of measure j.

Thus,

$$U_{1003} = 100 * 0.6 + 25 * 0.4 = 70$$

and

$$U_{1002} = 25 * 0.6 + 100 * 0.4 = 55$$

According to this calculation, policy 1003 is optimal. Tables 3 and 4 show the decision made by the DSP in selecting the optimal alternative for policy 1003 and display the financial data for this policy. On the basis of Tables 5 to 7 and the material listed here, the useful pavement life of 30 yr will be composed of two successive phases (18.6 yr and 11.4 yr):

Responsive Maintenance Policy 1003: Policy Description

Note: involved base is granular.

1. Patching 75.00 percent of unpatched cracks, but not more than 60.00 m²/km/yr; and at a present unit cost of 3.500 £E/m².
2. Asphaltic concrete surface dressing is applied when cracking and patching exceed 20.00 percent of the roadway, but not less than 4.00 yr/dressing, and not more than 7.00 yr/dressing, but not after analysis year 25. Required number of layers per one surface dressing = 2, at a present unit cost of 1.500 £E/m²/layer.
3. Other routine maintenance activities are also applied. They include drainage, vegetation, shoulders, and other miscellaneous activities. These activities are scheduled once per year and are estimated at a present (lump sum) cost of .000 £E/km/yr.
4. Overlay should be done when the value of PSI is between 2.40 and 3.00. Minimum allowable number of layers per overlay = 1. These layers are as prescribed above.

Figure 4b indicates that 51.40 percent of the current life cycle cost will be assigned for the first phase and 48.52 percent for the second phase. A surplus amount of 0.08 percent of the current life cycle cost will be inflated for 30 yr and deducted from the next life cycle cost. Thus, excluding the added and running user costs, the budget needed for construction and maintenance activities for the next 30 yr can be developed. The adequate rate of return can then be determined and the financial strategy investigated. The performance of the optimal strategy is illustrated in Figure 5.

CONCLUSIONS

The major conclusions drawn from the research can be summarized as follows:

- When studying flexible pavements for a specific time, the lowest life cycle cost is not the only factor that can be used to evaluate alternatives at a project level. The lowest cost/performance utility ratio should also be considered.
- The final choice of a maintenance policy should be based on the alternative that has one of the above two ratios in addition to the least value of performance/time, in other words, the best performance for the analysis period.
- If several maintenance policies are being evaluated, the optimal selection is the policy that has more uniformity of expenditures (i.e., an adequate investment rate) through the

TABLE 5 INITIAL PAVEMENT STRUCTURE (OPTIMUM ALTERNATIVE FOR SAMPLE PROBLEM)

Layer Number	ID No.	Material Type	Thickness (mm)	Layer Coefficient
1	10	AC surface course	40	.45
2	22	Premix binder course	100	.40
3	30	Crushed stone base	300	.14
4	50	Crushed stone subbase	300	.11

NOTE: Optimum alternative 1. Structural number = 4.64.

TABLE 6 OVERLAYS (OPTIMUM ALTERNATIVE FOR SAMPLE PROBLEM)

Overlay Number	Layer Number	ID No.	Material Type	Thickness (mm)	Layer Coefficient	Time of Overlay	Serviceability Before	Serviceability After	Structural Number Before	Structural Number After
1	1	10	AC surface overlay	40	.45	15.1	2.40	4.08	3.64	5.92
	2	22	Premix base overlay	100	.40					
			Wedge/leveling	27						
2	1	10	AC surface overlay	40	.45	25.0	3.00	4.16	5.48	7.76
	2	22	Premix base overlay	100	.40					
			Wedge/leveling	19						

NOTE: Serviceability at 30.00 years is 3.68.

TABLE 7 PRESENT WORTH COSTS (OPTIMUM ALTERNATIVE FOR SAMPLE PROBLEM)

	Initial Construction	Overlay Construction: Wedge/ Leveling	Overlay Construction: Overlay	Overlay Construction: Traffic Handling	Routine Maintenance	Added User	Running User	Salvage
Initial construction	72461.2				1451.7		466064.9	
Overlay 1		24344.1	11329.3	1567.7	534.5	2850.1	382507.3	
Overlay 2		10980.5	6988.7	847.7	.0	41824.6	108114.0	
TOTAL	72461.2	35324.6	18318.0	2415.4	1986.2	44674.7	956686.2	−5525.7

NOTE: Total Cost = 1126340.0. All costs in £E/lane-km.

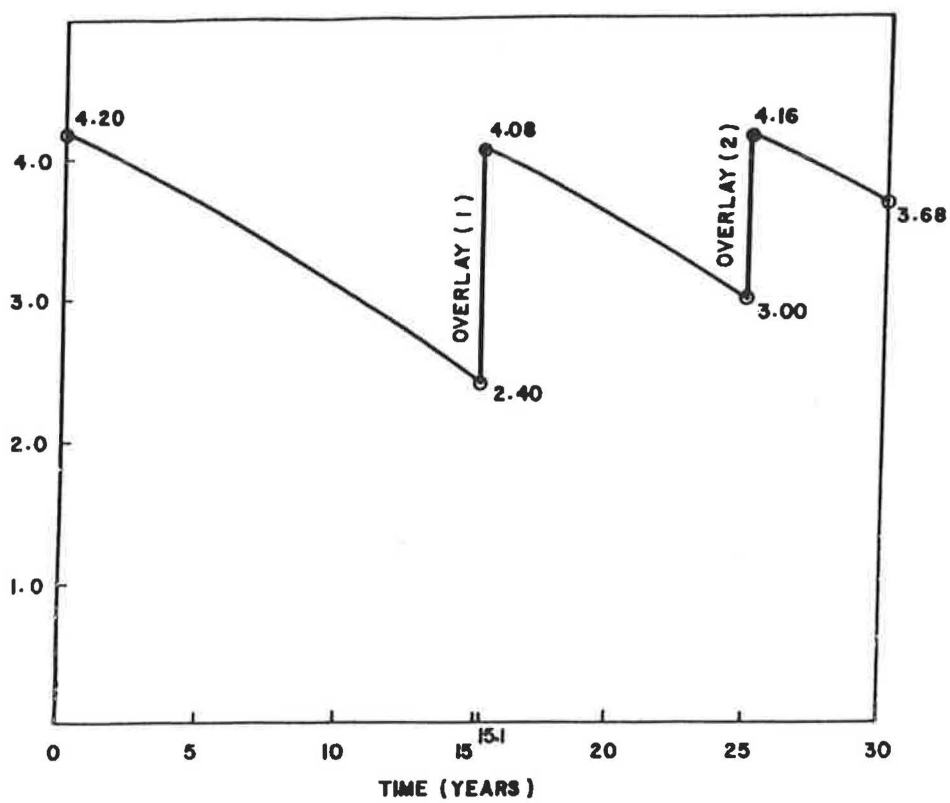

FIGURE 5 Performance of the optimal strategy (sample problem).

analysis period, based on the financial data provided by the program.

- This decision should be supported by estimating the benefits, as represented by the savings in routine maintenance costs and running user costs.
- The agency's policies and the circumstances of the project must also be considered in selecting the optimal routine maintenance policy for the PMS. Weights should be specified by the agency for use in the final choice among policies.

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A Simplified Pavement Maintenance Cost Model

SAMEH M. ZAGHLOUL, ESSAM A. SHARAF, AND AHMED A. GADALLAH

This paper presents a summary of an effort to develop a simplified pavement maintenance cost model for the Roads and Bridges Authority (RBA) of Egypt. The model was based primarily on modeling maintenance costs as a function of pavement condition. The study site included the highway network of the East-Delta district as well as several individual highways in the Delta region. The data collected for this study included information on pavement condition in terms of distress type and class. Pavement condition data were obtained from a previous large-scale study conducted in Egypt. Maintenance practices and unit costs were obtained from RBA's files, as well as from extensive interviews with experts from highway construction companies. The results indicated a significant sensitivity of maintenance costs to pavement condition at a certain condition range. It was determined that substantial savings in maintenance costs can be obtained by keeping pavement condition from reaching this condition range or at least by prolonging the period before this range is reached. In addition, it was recommended that the presence of such maintenance cost models can draw the attention of top management, particularly in developing countries, to the importance of the systematic monitoring of pavement condition as well as to the fact that maintenance budget allocation should not always follow the rule of "the worst is first."

Pavement maintenance management (PMM) is the process of coordinating and controlling a comprehensive set of activities to maintain pavements. Simply stated, it enables the best use of available resources by minimizing costs and maximizing benefits (*1*).

A successful PMM scheme should include a maintenance cost model that is sensitive and responsive to pavement condition. Reliable cost estimates can then be obtained based on actual factors affecting pavement condition (such as materials, design, quality control, and policies).

In some U.S. highway departments and in Egypt, as well as in several other developing countries, pavement maintenance cost estimates are often based primarily on previous experience. This usually leads to a wide gap between the estimates and actual project costs. It is believed that the use of a successful cost model can reduce this gap. A cost model can quantify the consequences of different pavement maintenance activities. In addition, it can specify those condition regions at which pavement maintenance costs are most sensitive to pavement condition. Policies can be set to keep pavements from reaching these critical regions or at least to prolong the period before they are reached.

This study was initiated to develop a simplified pavement maintenance cost model to be used by the Egyptian Roads and Bridges Authority (RBA) in the development of a Maintenance Management System (MMS). The model presented in this paper employs condition data (represented by the most common method of pavement evaluation—pavement surface condition assessment) and detailed costing of different maintenance activities and practices.

Public Works Department, Cairo University, Giza, Egypt.

DEVELOPMENT OF MAINTENANCE COST MODEL

Purpose of Model

The main purpose of the maintenance cost model is to determine the costs required to restore pavement surface condition to its "as-constructed state" for various levels of serviceability. This information is important as feedback for planning, design, and construction. The type and degree of maintenance can influence the rate of serviceability loss of pavement. The maintenance cost model can help pavement managers plan, direct, and control maintenance activities so an acceptable level of service, consistent with the class of pavement, can be achieved. In addition, it can assist in evaluating the methods and materials used in maintenance so that efficient, economical practices can be developed.

Network and Section Identification

The network considered in the development of this model included all paved highways in the East-Delta District as well as a set of individual highways representing different areas of the overall Delta paved network. The network length was 1592 km, of which 1547 km were managed by RBA and 45 km by the Ministry of Reconstruction. The network was divided into 327 homogeneous sections on the basis of the following factors:

- Pavement types and age,
- Layer types and thicknesses,
- Traffic volumes,
- Geometric characteristics (such as number of lanes, lane width, and shoulders), and
- Highway class.

Table 1 shows the highway network links considered in this study, while Figure 1 presents a map of the study network.

TABLE 1 HIGHWAY NETWORK LINKS

Highway Name	Length (km)	Authority	District Name
. Cairo-Alex. (Agriculture)	204 x 2 (2-Way)	RBA	146km C. Delta Dis. 126km M. Delta Dis. 200km W. Delta Dis.
. Cairo-Alex (Desert)	205	RBA	90.5km C. Delta Dis. 114.5km W. Delta Dis.
. Cairo-Ism. (Desert)	98	RBA	34.0km C. Delta Dis 64.0km I. Delta Dis
. Cairo-Suez	118	RBA	40km C. Delta Dis. 78km I. Delta Dis.
. Bnha-Mansoura Damietta . Damietta-Dibh . Dibh-Port Said	149 45 18	RBA Ministry of Reconstruction RBA	East-Delta --------- East-Delta
. Banha-Zagazig - Salehia	106	RBA	East-Delta
. Hawata-Sherbin-Blqas	79	RBA	East-Delta
. Talkha-Sherbin-Damietta	90	RBA	East-Delta
. Abu Hammad-Zagazig -Mit Ghamr	53	RBA	East-Delta
. Belbis-Mansoura	75	RBA	East-Delta
. Abu Kebir-Senbe llawen	28	RBA	East-Delta
. Dekerness-Mansoura	28	RBA	East-Delta
. Faqous-Hessenia	25	RBA	East-Delta
. Talkha-Blqas	18	RBA	East-Delta
. Esbt Bata - Daheria	49	RBA	East-Delta

Total=1592

Condition Data

The data included in this study were based on a condition survey conducted in the Delta Study (*2*) in 1981. In this study, the most common distress types in Egypt were found to be

- Cracking (longitudinal, transverse, map cracks),
- Surface damage (holes, bleeding, crumbling edge), and
- Deformations (rutting, unevenness).

The distress types and their codes are shown in Table 2.

The Texas condition survey method (*3*) was used in the Delta Study. The original Texas method defined the distress by its type, severity, and density. However, in the Delta Study, the distress severity and the distress density were combined and called the distress class (in other words, the distress was defined by its type and class only). Pavement distress class ranged from 0 to 3, where 0 meant no distress, 1 meant low density, 2 meant medium density, and 3 meant high density. Table 3 shows the percentages of deteriorated areas (densities) corresponding to each distress (type and class), known as the density matrix.

In the research described in this paper, the 327 sections were evaluated separately, and a condition rating index (CRI) was calculated. The CRI followed the survey method used in the Delta Study.

On the basis of professional judgment, the Texas "scores" were modified as shown in Table 4. The CRI can be calculated by summing the score for each distress (in the same section) and determining a general rating for the section (see Table

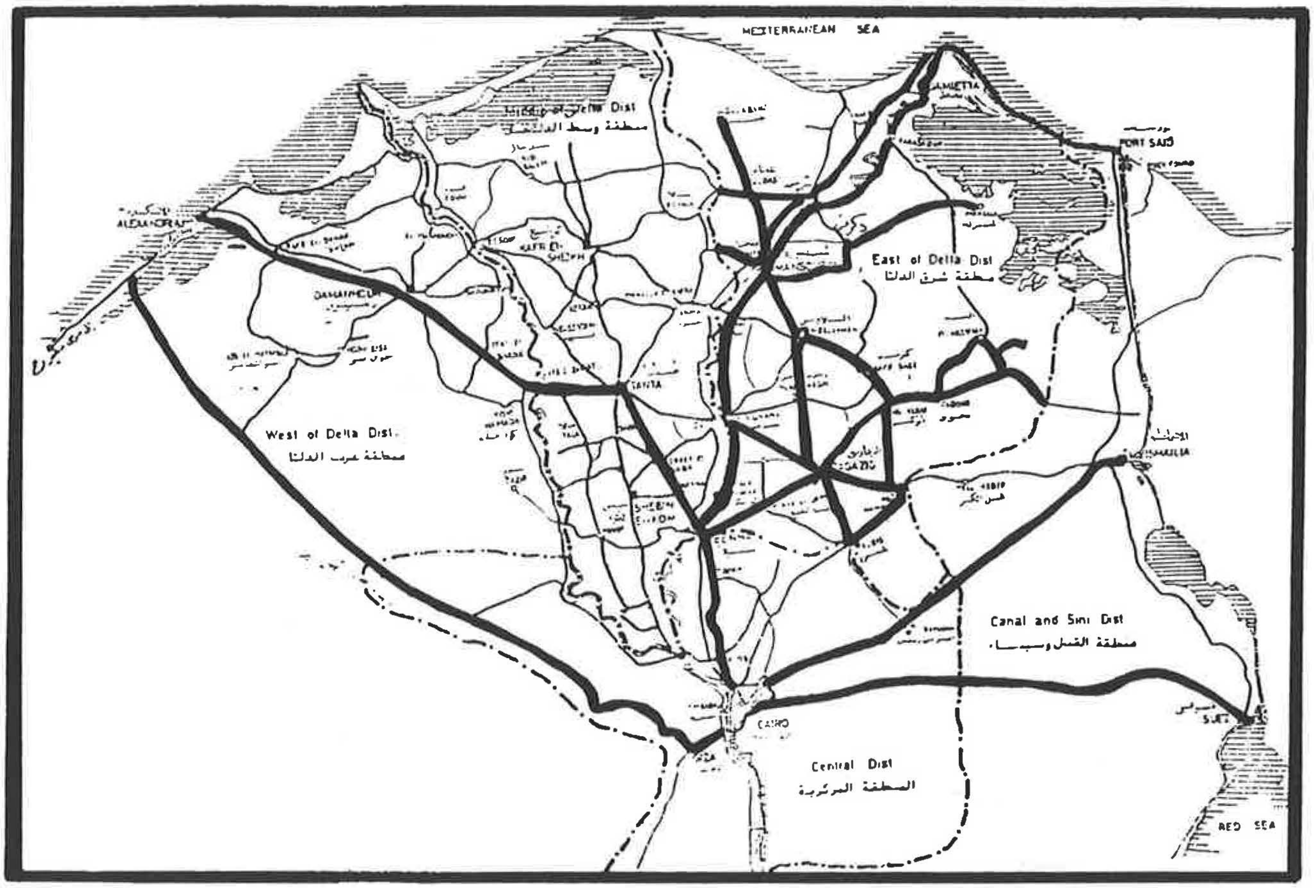

FIGURE 1 Highway network links.

TABLE 2 DISTRESS TYPES AND CODES

Distress Code	Distress Type (Name)
1	Longitudinal cracks
2	Transverse cracks
3	Map cracks
4	Potholes
5	Bleeding
6	Crumbling edges
7	Longitudinal unevenness
8	Rutting

TABLE 3 DENSITY MATRIX

Distress Code[a]	Density by Distress Class[b] 0	1	2	3
1	0	10%	15%	20%
2	0	10%	15%	20%
3	0	5%	15%	25%
4	0	0.03%	0.15%	0.3%
5	0	10%	17.5%	25%
6	0	10%	20%	30%
7	0	10%	20%	30%
8	0	10%	20%	30%

[a]Refer to Table 2.
[b]The distress classes are as follows: 0 = no distress, 1 = low density, 2 = medium density, and 3 = high density.

5). For example, if the condition survey results of a section were as follows,

Distress Code	*Distress Class*
1	1
2	1
3	2
4	1
5	0
6	3
7	1
8	3

then, using the scores given in Table 4, the CRI is

$$CRI = 10 + 7 + 15 + 15 + 0 + 10 + 5 + 7 = 69$$

where the score corresponding to distress 1/class 1 equals 10, distress 2/class 1 equals 7, and so on, as indicated in Table 4.

The CRI score of 69 means that the section's general rating is fair (see Table 5). A high CRI score represents a poor condition, and a low score represents a good condition.

Maintenance Activities and Unit Costs

Interviews with experienced highway engineers in Egypt, as well as results of previous studies (*2,4*), indicated that the most common maintenance activities in Egypt are

- Sealing cracks,
- Scarifying,

TABLE 4 WEIGHTING FACTORS FOR DIFFERENT DISTRESS TYPES

Distress Code[a]	Modified Score by Distress Class[b]			
	0	1	2	3
1	0	10	15	20
2	0	7	12	15
3	0	10	15	25
4	0	15	20	25
5	0	2	5	7
6	0	5	7	10
7	0	5	7	10
8	0	2	5	7

[a]Refer to Table 2.
[b]The distress classes are as follows: 0 = no distress, 1 = low density, 2 = medium density, and 3 = high density.

TABLE 5 SECTION RATINGS

CRI	Section Class
≤30	Very good
30–60	Good
60–70	Fair
70–85	Poor
≥85	Very poor

TABLE 6 AVAILABLE MAINTENANCE ACTIVITIES

Maintenance Activity Code	Maintenance Activity Name
X1	Sealing cracks
X2	Scarifying
X3	Seal coating
X4	Skin patching without applying wearing surface
X5	Skin patching
X6	Deep patching without applying wearing surface
X7	Deep patching
X8	Overlay

- Seal coating,
- Skin patching,
- Deep patching, and
- Overlaying (see Table 6).

Unit cost computation approaches for each of these activities are briefly described in the following sections.

Sealing Cracks

Cracks Less Than 3 mm in Width The common practice is to fill cracks less than 3 mm wide with a rapid-curing cutback liquid asphalt (RC-5) or with an asphalt cement (60/70 or 80/100 penetration grades). The rate of application used is approximately 0.25 kg/m^2.

Cracks Wider Than 3 mm Two methods are commonly used for cracks wider than 3 mm. In the first method, a liquid asphalt or an asphalt cement is applied, then clean sand is spread over the cracked area. The second method uses a sand mix to fill the cracks. The unit cost calculation for this activity is based on an average condition; therefore, the cost is calculated for an application of 0.25 kg/m^2 of asphalt cement and 0.5 cm of clean sand.

Scarifying

Scarifying is defined as spreading a fine aggregate (sand) during hot weather and then scarifying this aggregate using a scraper. The cost calculation is based on the cost of spreading the sand (at an average depth of 2 cm) and the cost of scarifying the sand.

Seal Coating (Surface Dressing)

Two alternatives are given for seal coating:

1. Sand coat (1 cm sand + 1.5 kg/m^2 of liquid asphalt), and
2. Seal coat (1 cm crushed stone + 1.5 kg/m^2) of liquid asphalt.

Skin Patching

Skin patching is defined as removing the top 5 cm of the deteriorated surface and replacing it with a new asphalt concrete surface mix. Cost calculations are based on three items:

1. Cost of removing, loading, and transporting the top 5 cm of the old surface,
2. Cost of tack coating the vertical sides of the cut with asphalt (0.5 kg/m^2), and
3. Cost of securing, placing, and compacting the new asphalt concrete surface mix.

Deep Patching

In Egypt, deep patching is defined as the removal of the full pavement depth (surface, binder, base course layers, and 15 cm of the upper portion of the subgrade) and the placement of new courses. Cost items include the following:

- Cost of removing, loading, and transporting the full depth of pavement (approximately 45 cm thick),
- Cost of tack coating the vertical sides of the cut with asphalt (0.5 kg/m^2),
- Cost of securing, placing, and compacting 20 cm of pit-run gravel as a base course,
- Cost of prime coating the base course surface with a liquid asphalt (1.5 kg/m^2),
- Cost of securing, placing, and compacting 5 cm of an asphalt concrete binder course mix,
- Cost of tack coating the binder course surface with a liquid asphalt (0.5 kg/m^2), and
- Cost of securing, placing, and compacting 5 cm of an asphalt concrete surface course mix.

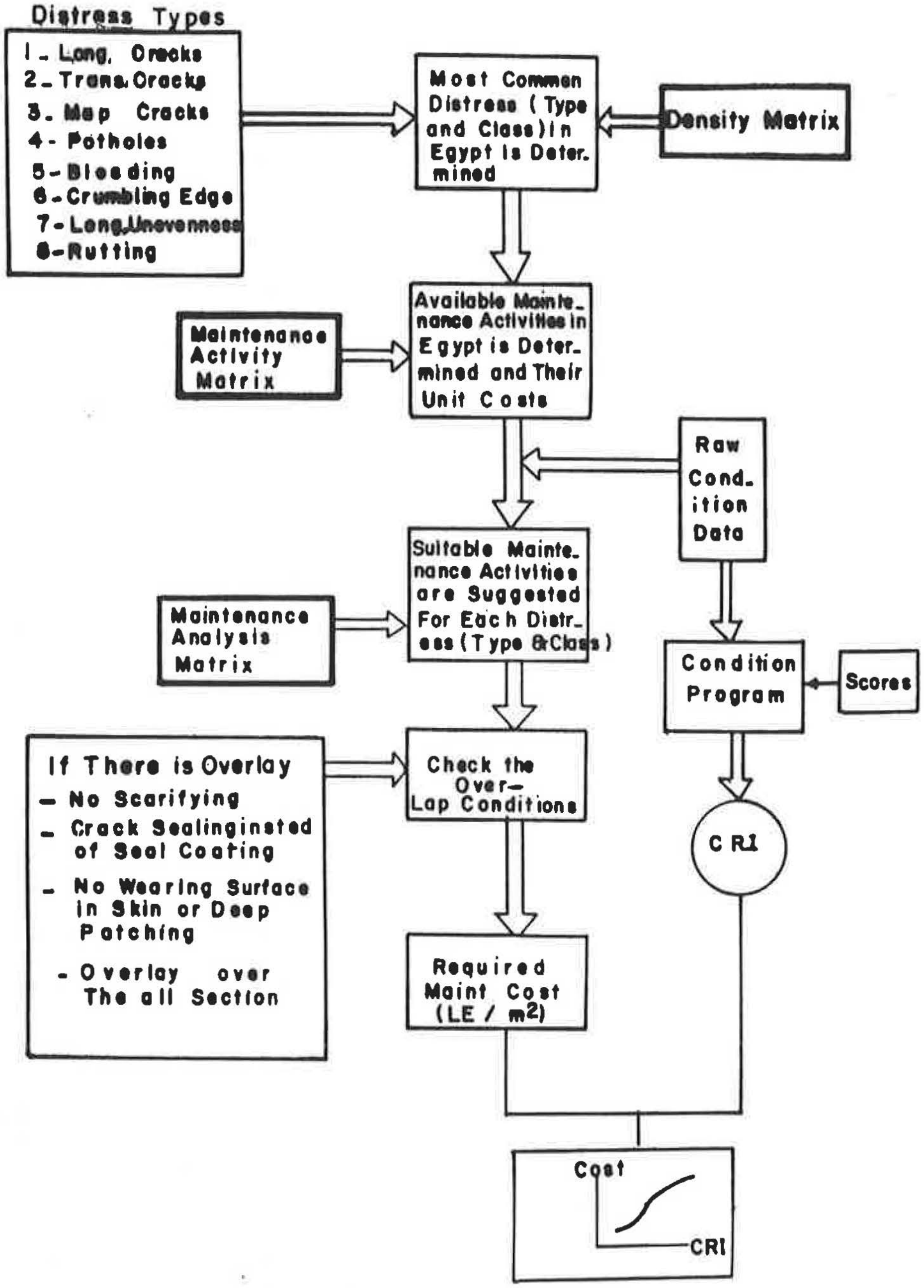

FIGURE 2 Maintenance cost model.

Overlaying

The overlay includes two layers: a 5-cm premixed macadam and a 3-cm asphalt concrete mix for the surface. It is suitable for pavements that have relatively even surfaces and low traffic volumes.

Maintenance Cost Model

As mentioned earlier, the main purpose of the maintenance cost model (MCM) was to determine the cost (per square meter) required to restore pavement surface condition to its as-constructed state. Six steps were followed to develop this model (see Figure 2):

1. The most common distresses (types and classes) in Egypt were identified on the basis of the results of the Delta Study (*2*) and meetings with the East-Delta district engineers (see the previous section on Condition Data).
2. For each distress (type and class), a maintenance activity was suggested (considering that distress as the only one in the section). Table 7 shows the suggested maintenance activities for the different distress types and classes.
3. After identifying the suggested maintenance activities individually, the overlap between these activities was considered.
4. To prevent any overlap between maintenance activities on a section, the following conditions were established:

   a. If the section required an overlay, other maintenance activities over this section would be specified as follows: (1)

TABLE 7 SUGGESTED MAINTENANCE ACTIVITIES FOR EACH DISTRESS (TYPE AND CLASS)

Distress Code[a]	Maintenance Activity Code by Distress Class[b]			
	0	1	2	3
1	None	X1	X1	X3
2	None	X1	X1	X3
3	None	X5	X5	X8
4	None	X7	X7	X7
5	None	None	X2	X2
6	None	None	X5	X5
7	None	X5	X5	X5
8	None	None	X5	X5

[a]Refer to Table 2.
[b]The distress classes are as follows: 0 = no distress, 1 = low density, 2 = medium density, and 3 = high density.

TABLE 8 MAINTENANCE ACTIVITY UNIT COSTS

Maintenance Activity Code[a]	Maintenance Activity Cost (£E/m^2)
X1	0.18
X2	0.20
X3	0.92
X4	1.15
X5	5.30
X6	7.50
X7	22.50
X8	6.90

[a]Refer to Table 6.

no scarifying would be conducted; (2) sealing cracks would replace seal coating; and (3) skin or deep patching would be applied without a wearing surface.

b. The overlay would be carried over the entire section (density equals 1.0).

The mathematical representation of these conditions is as follows:

If there is X8, then

X2 = 0

X3 = X1

X5 = X4

X7 = X6

$D_8 = 1.0$

5. The maintenance activity costs (Egyptian pound £E/m^2) were obtained from the East-Delta district files and were verified by the Arab Contractors Company engineers. Table 8 shows the unit cost (£E/m^2) associated with the different maintenance activities.

6. The section maintenance cost (£E/m^2) was then calculated as follows:

$$C_T = \Sigma D_i * c_i$$

where

C_T = total unit cost (£E/m^2) of the entire section,
D_i = density of the ith distress type class (from Table 3), and
c_i = maintenance unit cost corresponding to the ith distress type class (from Table 8).

Application of the MCM

The application of the MCM can be best illustrated through examples. The two examples provided below include actual condition cases. In each example, the distress data (type and class) are presented first. The steps described above are applied in a systematic manner to determine the section maintenance unit cost.

Example 1, Highway 5, Station 38.6

Distress Code	*Distress Class*
1	3
2	3
3	3
4	1
5	2
6	2
7	2
8	1

The maintenance cost was calculated as in Table 9. Because there was an X8 maintenance activity code, X2 = 0, X3 = X1, X5 = X4, and X7 = X6, and the density of X8 = 1.0 (overlap condition).

Example 2, Highway 50, Station 159

The distress class was 1 for all distress codes. The maintenance cost was calculated as shown in Table 10. There was no X8 maintenance activity code (overlap condition).

RELATIONSHIP BETWEEN PAVEMENT CONDITION AND MAINTENANCE COST

Although a pavement condition rating can provide the decision maker with a clear picture of how the network is behaving, it does not indicate how much it will cost to repair the network. Without a clear model of the relationship between pavement condition and repair cost, the ultimate goal of pavement condition assessment cannot be achieved.

The purpose of this study, therefore, was to develop the relationship between the CRI and the corresponding repair and maintenance cost based on the MCM. The following procedure was used to develop this relationship:

1. Each of the samples stored in the condition data base was considered.
2. The existing distresses and their corresponding classes were identified, and the CRI was calculated.
3. The identified distresses were processed through the

TABLE 9 CALCULATION OF MAINTENANCE COST FOR EXAMPLE 1: X8 (OVERLAP) CONDITION PRESENT

	Distress Code							
	1	2	3	4	5	6	7	8
Class	3	3	3	1	2	2	2	1
Density (from Table 3)	20%	20%	25%	0.03%	17.5%	20%	20%	10%
Maintenance activity (from Table 7)	X3	X3	X8	X7	X2	X5	X5	—
Modified density (according to overlap condition)	0.20	0.20	1.0	0.0003	0.175	0.2	0.2	0.1
Modified maintenance activity (according to overlap condition)	X1	X1	X8	X6	—	X4	X4	—
Modified maintenance activity unit cost (according to overlap condition)	0.18	0.18	6.9	7.5	—	1.15	1.15	—
Modified density * modified maintenance activity unit cost	0.036	0.036	6.9	0.002	—	0.23	0.23	—

NOTE: All costs £E/m². Because there is X8, X2 = 0, X3 = X1, X5 = X4, X7 = X6, and the density of X8 = 1.0. Total maintenance cost = 0.036 + 0.036 + 6.9 + 0.002 + 0.23 + 0.23 or 7.434 £E/m² (CRI for this section = 96).

TABLE 10 CALCULATION OF MAINTENANCE COST FOR EXAMPLE 2: NO X8 (OVERLAP) CONDITION

	Distress Code							
	1	2	3	4	5	6	7	8
Class	1	1	1	1	1	1	1	1
Density (from Table 3)	10%	10%	5%	0.03%	10%	10%	10%	10%
Maintenance activity (from Table 7)	X1	X1	X5	X7	—	—	X5	—
Modified density (according to overlap condition)	0.1	0.1	0.05	0.0003	0.1	0.1	0.1	0.1
Modified maintenance activity (according to overlap condition)	X1	X1	X5	X7	—	—	X5	—
Modified maintenance activity unit cost	0.18	0.18	5.3	22.5	—	—	5.3	—
Modified density * modified maintenance activity unit cost	0.018	0.018	0.265	0.007	—	—	0.53	—

NOTE: All costs £E/m². Total maintenance cost = 0.018 + 0.018 + 0.265 + 0.007 + 0.53, or = 0.84 £E/m². (CRI for this section = 56).

MCM, and the most suitable maintenance action was identified.

4. The cost of applying the selected maintenance activity was determined.

5. The maintenance cost for a particular sample with a known CRI was determined.

6. By repeating the first five steps for all samples in the condition data base, a set of observations was obtained, containing CRI values and their corresponding maintenance costs.

7. The average maintenance cost for each CRI class (as indicated in Table 11) was calculated.

Figure 3 is a graphical representation of the maintenance cost versus CRI relationship developed on the basis of the results presented in Table 11. This figure indicates several important results:

1. The cost versus CRI relationship can be divided into three basic regions with respect to the rate of the cost increase. In the first region (CRI < 60—good condition), the maintenance cost increases very slowly as the CRI increases (i.e., gets worse). In the second region (60 < CRI < 95—medium condition), the maintenance cost increases sharply as the CRI increases. Finally, in the third region (CRI > 95—bad con-

TABLE 11 PAVEMENT CRI GROUPS AND CORRESPONDING MAINTENANCE COSTS

Condition Group	CRI Range	Average Maintenance Cost (£E/m²)
1	<10	—
2	10–20	0.67
3	20–30	0.68
4	30–40	0.71
5	40–50	0.88
6	50–60	1.16
7	60–70	2.03
8	70–80	3.53
9	80–90	5.79
10	90–100	7.65
11	100–120	7.86

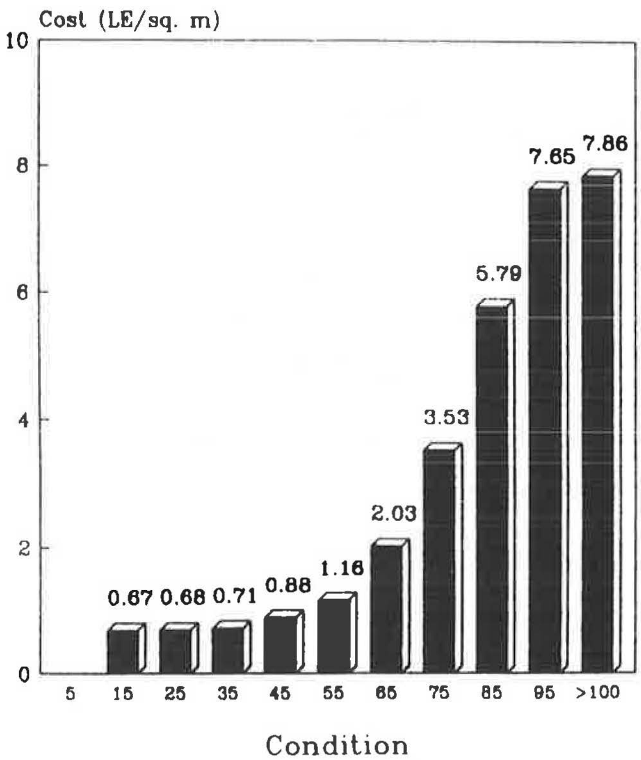

FIGURE 3 Relationship between maintenance costs and pavement CRI groups.

dition), the rate of the maintenance cost increase becomes small again. Mathematically, these three rates can be considered as follows:

$$R = \frac{C_2 - C_1}{CRI_2 - CRI_1} \qquad (\text{£E/m}^2\text{/CRI point})$$

where

R = rate of maintenance cost increase due to 1-point increase in CRI,
C_1 = required maintenance cost at the first stage,
CRI_1 = CRI at the first stage,
C_2 = required maintenance cost at the second stage, and
CRI_2 = CRI at the second stage.

Thus, the rate of the maintenance cost increase in the first region is equal to

$$\left(R_1 = \frac{1.16 - 0.67}{55 - 15}\right) = 0.012 \qquad \text{£E/m}^2\text{/CRI point}$$

Similarly, the corresponding rates of increase in the second and third regions are $R_2 = 0.162$ and $R_3 = 0.014$, respectively. These rates are graphically displayed in Figure 4.

2. According to the rates described above, the following can be used as a guide when developing a maintenance program:

- To avoid a sharp increase in maintenance cost, pavements should not be allowed to reach the second stage of deterioration, if possible. This emphasizes the importance of conducting preventive maintenance to prolong the period before the pavement reaches the critical stage.
- If the pavement CRI does reach the second stage, the appropriate maintenance must be carried out immediately, because any delay in maintenance causes a sharp increase in the corresponding maintenance cost.
- Maintenance of pavements in the third stage can be delayed, particularly for roads carrying low volumes or having low strategic importance, without a considerable maintenance cost increase.

3. The highway maintenance priority (from a maintenance cost point of view) can be ranked as follows (see Figure 5):

- 1st—Highways at the end of the first stage or in the second stage,
- 2nd—Highways in the first stage, and
- 3rd—Highways in the third stage.

CONCLUSIONS

On the basis of the research described in this paper, it was concluded that a successful maintenance program cannot be applied without a pavement monitoring program, at least for pavement surface condition. This is particularly important in developing countries, where maintenance programs usually follow an ad hoc approach and the predominant rule in priority setting is "the worst is first." The results of this study indicate that this rule is not always valid.

In addition, the results strongly suggest the use of a network ranking approach rather than a section-by-section approach. This can be illustrated through a very simple example. If the section-by-section approach is followed, then a section in the third (worst) stage may be selected for repair with approximately 7 £E/m². If, however, a network ranking approach is used, the repair of these sections might be deferred and other sections just approaching the critical stage (stage 2) might be selected. In this case, the average cost to repair the selected sections would be about 1 £E/m². In other words, with the same budget as that allocated to the badly deteriorated section, seven other sections (each with the same area as the badly deteriorated one) can be repaired, their condition can be preserved, and the period before they reach the critical stage can be delayed. Undoubtedly, this would lead to a better network condition.

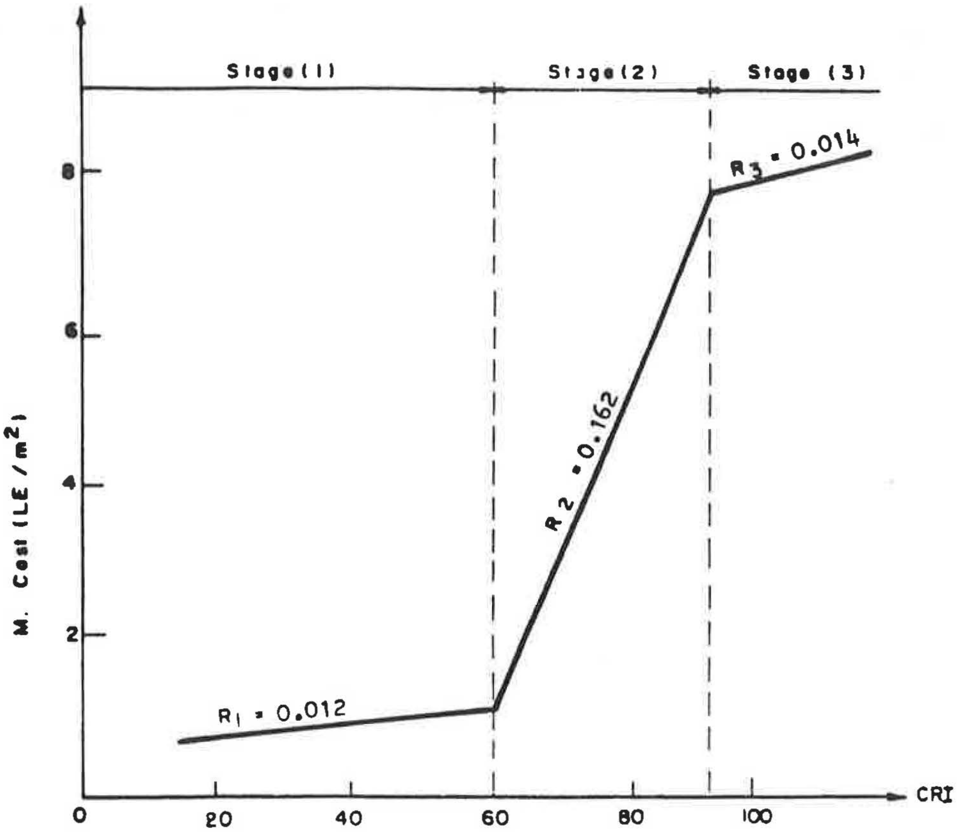

FIGURE 4 Relationship between rate of maintenance cost increase and CRI change.

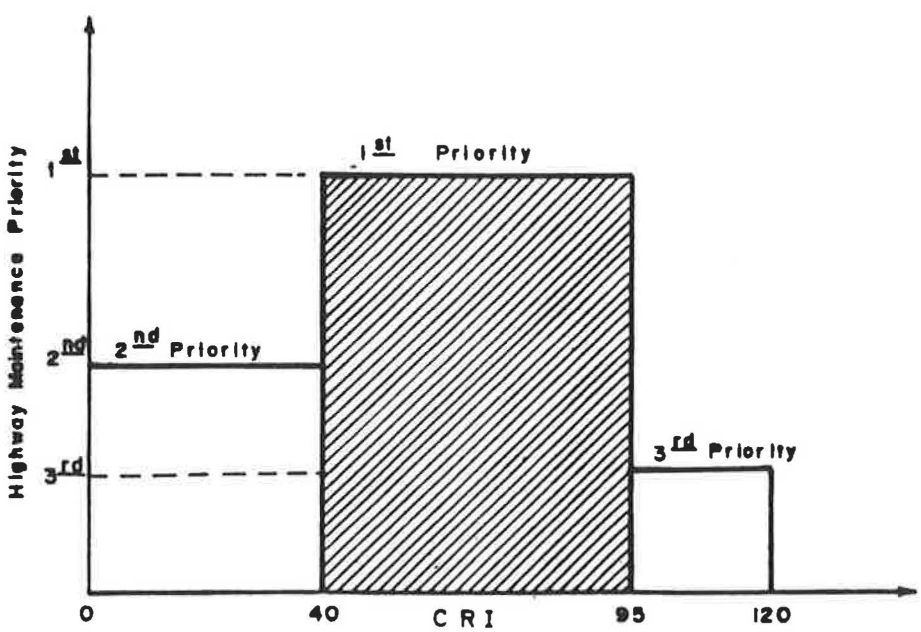

FIGURE 5 Highway maintenance priority.

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Measurement of Highway Maintenance Patrol Efficiency: Model and Factors

ALEX KAZAKOV, WADE D. COOK, AND Y. ROLL

A model for evaluating the relative efficiency of a set of highway maintenance patrols is discussed. The particular model structure used, the data envelopment analysis (DEA) method, is currently being implemented in Ontario. The paper concentrates primarily on the factors (inputs and outputs) that are appropriate for use in evaluating maintenance patrols. Sample results from the pilot study are discussed.

This paper investigates the problem of evaluating the efficiency of highway maintenance patrols and discusses a tool for performing such an evaluation.

Efficiency evaluation has considerable benefit for highway departments and maintenance units. From the perspective of top management, this tool provides a means of distinguishing good managers from less effective ones. Moreover, it can provide an understanding of the impact of such factors as climatic condition, pavement health, and degree of privatization on maintenance effectiveness. In this manner, an efficiency monitoring tool can aid in budget planning and in the design of maintenance policies and practices. From the point of view of the decision-making unit (the maintenance patrol), particularly the maintenance engineer, routine efficiency evaluation facilitates a closer monitoring of how the patrol is conducting its business. The engineer receives an annual status report showing the patrol's standing relative to other patrols. Furthermore, the model provides an efficient subset (peer group) of patrols for comparison. Thus the engineer has a barometer for evaluating the patrol's current status and for choosing a direction for future changes.

Because of the need to consider qualitative factors such as climatic condition, road condition, and extent of privatization, "production" standards are difficult, if not impossible, to establish. This being the case, the usual industrial engineering approaches to productivity do not apply. The model that has been adopted for examining patrol maintenance in Ontario is referred to as the data envelopment analysis (DEA) approach. The DEA model was developed by Charnes et al. (*1*) specifically for evaluating the relative efficiency of a set of decision-making units. In particular, the technique has been applied to hospitals, schools, courts, airforce maintenance units, and so on. The ideal setting for this model occurs when there are similar decision-making units (such as maintenance patrols) with multiple inputs and outputs, where qualitative (noneconomic) factors need to be considered.

Because the model has been discussed at length in the literature, only brief mention of its structure is made here. The primary thrust of this paper is a discussion of the factors (inputs and outputs) that are appropriate for the maintenance area. In addition, the difficulties surrounding the quantification of some factors and the associated problem of collapsing subfactors into overall composite factors for use in the DEA model are addressed. Some preliminary results from the Ontario study are given.

A. Kazakov, Research and Development Branch, Ontario Ministry of Transportation, Downsview, Ontario, Canada. W. D. Cook, York University, Toronto, Canada. Y. Roll, Technion–Israel Institute of Technology, Haifa, Israel.

PATROL OPERATION

Most of the routine maintenance activities on Ontario's highways fall under the responsibility of the 244 patrols scattered through the province. Each patrol is responsible for a fixed number of highway lane-kilometers and oversees the activities associated with that portion of the network. More than 100 different categories of operations/activities exist. They are divided into five areas: surface, shoulder, right of way, median, and winter operations.

The current system for monitoring patrol activities within the Ontario Ministry of Transportation is known as the maintenance management system (MMS). The MMS is a computerized recordkeeping system that keeps track of total work accomplished by type of operation, patrol, and highway class. This system is similar to those used in other Canadian provinces and in the United States.

METHODS FOR MEASURING EFFICIENCY

The productivity, or efficiency level, of any decision-making unit (DMU) (such as a factory, government department, or maintenance patrol) is a measure of the extent to which that DMU makes the best possible use of a given set of inputs (resources) to produce some set of outputs. In this context, "best possible use" loosely means getting the most out of available resources within a given set of circumstances.

In an industrial setting, efficiency or productivity is usually approached from an engineering perspective on the basis of on production standards. In this case, the productivity of a DMU is the ratio of standard or required inputs (needed to create the current level of output) to the actual inputs used.

An alternative to these absolute measures of efficiency is a measure that evaluates a DMU relative to some comparison group. Such an approach is not only realistic but may be the only one applicable in many not-for-profit environments. This is the principle on which the DEA approach is based. DEA is capable of handling a variety of factors, such as number of accidents, maintenance dollars, cars per day, average age of pavement, and so on, and allows for measurement of these factors on different scales. This approach seems particularly suited to the maintenance area because factors such as traffic intensity, safety parameters, and average age of pavements are an important part of the picture. Formally, the DEA method is defined as follows:

Given a set of J DMUs, the model determines for each DMU_0 the best set of input weights $\{\nu_{i0}\}_{i=1}^{I}$
and output weights $\{\mu_{r0}\}_{r=1}^{R}$

such that the ratio of total weighted outputs to total weighted inputs is maximized. This is done subject to two constraints: that the corresponding ratio for each DMU_j (including the one in question) does not exceed 1, and that the weights μ_{r0} and ν_{i0} fall within reasonable bounds. The ratio e_0 is the relative efficiency rating for DMU_0. Let the following notation be adopted:

Y_{rj} = value of output factor r for DMU_j,
X_{ij} = value of input factor i for DMU_j,
μ_{r0}, ν_{i0}, = "weights" for the corresponding factor,
$Q1_r, Q2_r, P1_i, P2_i$ = bounds imposed on weights, and
T = transformation factor.

In mathematical terms, the DEA model involves solving the J fractional programming problems:

Max

$$\left\{ e_0 = \frac{\Sigma_r \mu_{r0} Y_{r0}}{\Sigma_i \nu_{i0} x_{i0}} \right\}$$

Subject to:

$$\frac{\Sigma_r \mu_{r0} Y_{rj}}{\Sigma_i \nu_{i0} X_{ij}} \leq 1 \qquad \text{for all DMUs} \qquad j = 1, 2, \ldots, J$$

$$Q2_r \leq \mu_{r0} \leq Q1_r \qquad \forall\, r = 1, 2, \ldots, R$$

$$P2_i \leq \nu_{i0} \leq P1_i \qquad \forall\, i = 1, 2, \ldots, I$$

It can be shown that this ratio model reduces to a linear programming problem. Details can be found in the work of Charnes et al. (*1*) and Cook et al. (*2*).

In choosing weights for any patrol, the DEA model tries to present the patrol's position in the most favorable light. In this setting, then, if a patrol can be shown to be efficient (a ratio of 1) by some reasonable set of weights, it should be efficient in reality. A patrol will only be declared inefficient if it is dominated by other patrols or combinations of patrols. Thus, DEA should be viewed as a technique for identifying inefficiency.

SELECTION OF FACTORS

The process of selecting factors in a DEA model should concentrate on finding effects of maintenance activities together with a set of explanatory, or causal, factors that allow these effects to be created. Outputs should measure the effectiveness of the patrols' actions. Potential candidates would be number of vehicles served, accidents (or reduction thereof), level of pavement quality, and so on. Inputs are of two types:

1. Controllable factors, such as the size of the budget and the percentage of work done under private contract; and
2. Factors not under the control of the patrol or district, such as environmental measures (for example, inches of snowfall) and average age of pavements.

These latter factors describe the circumstances under which a patrol is forced to operate and may have a strong effect on the outputs. In the Ontario study, maintenance staff have aided in the selection of factors.

After choosing the factors to be used in describing cause and effect for patrol activities, the issue of quantification must be addressed. While the DEA structure does not require that factors be reducible to a common unit, they must be quantified on some scale. For example, if safety is a principal consideration with regard to maintenance effort, some reasonable method of capturing safety (such as skid resistance, number of accidents, or number of fatal accidents) must be found. Severity of the environment is likely to be an important determinant of the extent to which patrol efforts are effective. Yet there is no obvious single measure of environmental impact. Again, quantification is a pressing issue in the selection of factors.

For the analysis of relative efficiency of maintenance patrols in Ontario, the following set of factors was chosen:

Outputs

Size of System

This factor is intended to capture the size of the task facing patrol crews. It considers the amount of road surface to be tended, the shoulder and right-of-way area, and winter maintenance requirements. Specifically, the assignment size factor (ASF) is the sum over all road sections serviced by the patrol of

Length · Two Lane Equivalents (TLE) · Coefficient for Road Type
+ Length · TLE · Coefficient for Winter Operations
+ Length · Shoulder Width · Coefficient for Shoulder Type
+ Length · Coefficient for Other Operations (right of way, median, etc.)

Components of the assignment size were weighted as follows:

- For surfaces, per 1000 km TLE:

Type	*Coefficient*
1	1.97
2, 3	1.72
4	.92
5	.59
6, 7	.31

- For winter operations, a coefficient of 3.14 per 1000 km TLE;
- For shoulders, per 100 m^2 of shoulder:

Type	*Coefficient*
2	.18
4	.12
6	.14

TABLE 1 ACCIDENT FACTORS

District & Patrol	Total Number of Accidents	Road State: Good	Under Repair	Under Construction	Other	Adjustment*	Pavement Markings: Good	Faded	Obscured	Not Visible	No Markings	Not Applicable	Adjustment*	Surface Condition: Dry	Wet	Loose Snow	Slush	Packed Snow	Ice	Mud	Loose sand gravel	Adjustment*	Adjusted Number of Accidents
Kingston (8)		1	2	1.5	1		1	1.2	1.5	1.5	2	1		1	.8	.7	.7	2	.5	.8	1		
#1	79	78	-	-	1	0	63	4	3	8	1	-	+ 7.3	46	12	7	5	2	7	-	-	-10.1	76.2
#2	15	14	-	-	1	0	14	1	-	-	-	-	+ .2	7	5	1	-	-	2	-	-	- 2.3	12.9
#3	134	134	-	-	-	0	113	2	2	15	2	-	+10.9	63	38	7	4	3	19	-	-	-21.3	123.6
#4	83	83	-	-	-	0	63	5	3	11	1	-	+ 9	44	20	7	4	1	7	-	-	-11.1	80.9
#5	52	52	-	-	-	0	43	2	3	3	1	-	+ 4.4	31	10	4	2	0	5	-	-	- 6.3	50.1

- For rights of way, medians, and so on, a coefficient of 2.30 per 1000 km of road.

The types are those used in the highway inventory data, and the coefficients were determined from the corresponding expenditures in fiscal year 1986–87. Coefficients represent the relative proportions of the total maintenance expenditure on the various components. For example, surface type 4 work cost approximately three times as much as work on surface types 6 and 7 (.92 versus .31).

Average Traffic Serviced

This factor recognizes that greater maintenance efforts may be required on roads with higher traffic. This is true for two reasons. First, larger crew sizes are needed for multilane roads than for lower volume roads. Second, a higher standard of serviceability is often needed on the higher traffic roads. The average traffic serviced (ATS) factor is the sum over all road sections of

$$\text{Length} \cdot \text{AADT} \cdot 10^{-4}$$

Accidents

Maintenance crews are primarily occupied with the removal of problem areas that could result in accidents (such as washouts or potholes) or with work that results from accidents (such as repairs to damaged guardrails). One difficulty encountered with this factor is that accidents fall into different categories. In the model, therefore, accidents in a patrol are separated according to three groupings (see Table 1). The first group, Road State, includes four headings:

1. Good,
2. Under repair,
3. Under construction, and
4. Other.

For example, if there were 100 accidents in a patrol, it may turn out that 50 were on good roads, 20 on roads under repair, 20 on roads under construction, and 10 on other types of roads.

The second group, Pavement Markings, contains six headings:

1. Good,
2. Faded,
3. Obscured,
4. Not visible,
5. No markings, and
6. Not applicable.

The third group, Surface Condition, is divided into eight headings:

1. Dry,
2. Wet,
3. Loose snow,
4. Slush,
5. Packed snow,
6. Ice,
7. Mud, and
8. Loose sand gravel.

To obtain an accident statistic for a patrol, a set of importance weights were assigned to each heading under each of the three groups. The overall accident statistics (A) is then given as

$$A = \text{no. of accidents} + \sum_{i=1}^{3} (\text{adjustments})$$

where

$$\text{adjustment} = \sum_{j=1}^{k} (\text{no. of events}) \times (\text{Factor-1})$$

Here, $i = 1,2,3$ are the three groupings and $j = 1,2,\ldots,k$ are the headings under any given grouping.

Table 1 shows all factors and weights and illustrates a typical calculation.

Change in Pavement Condition

Because both maintenance and rehabilitation expenditures are inputs (discussed below), one of their major observable effects is the resulting change in the condition of the pavement. Specifically, the model uses the change in a patrol's average pavement condition rating from its level in the previous year to its current level.

Inputs

Maintenance Expenditures

This factor is divided into two different inputs: expenses incurred in-house and those arising from work done by private contractors. This distinction is made because the proportion of privatized work may greatly influence a patrol's productivity standing. It is also pointed out that, if efficiency is being examined in terms of winter maintenance, for example, only that portion of the expenditure figures relating to winter work is used.

Rehabilitation Expenditures

Because rehabilitation and maintenance expenditures go hand in hand, the total expenditure on rehabilitation (capital) is an important input. One problem with this factor has to do with when the rehabilitation was conducted. If, for example, maintenance expenditures for the year 1986 are used, the need for these expenditures is, to an extent, a function of the capital work done not only in 1986 but in several years preceding 1986. This being the case, capital expenditures for 5 yr (1982–1986) were taken in total and used as the rehabilitation budget input. Technically, a weighted total should be used (for example, capital expenditures in 1982 may have less influence than those of 1985). In this study, however, the simple sum was applied.

Climatic Input

There is unanimous agreement that climatic conditions influence the need for maintenance. Not only do frost heaves necessitate surface work but snowfall clearly influences winter maintenance activities (such as snow removal and salting).

Subfactors

Although no clear relationship has been established between pavement damage and such factors as frost depth, depth of water table, and number of freeze/thaw cycles, it is believed that these and other factors *do* influence the extent of damage. For the Ontario study, four subfactors were combined to arrive at an overall climatic impact parameter:

1. Number of major freeze/thaw cycles,
2. Number of minor freeze/thaw cycles,
3. Number of days where rainfall exceeded 10 mm, and
4. Total snowfall.

Standard definitions have been adopted within the Ministry of Transportation concerning freeze/thaw cycles. A major cycle occurs when there is significant thawing followed by full freezing. This phenomenon leads to water being trapped in the base and subbase of the pavement, causing volume shifts and pavement blow-ups. A threshold number of degree days for each thaw and freeze portion was chosen. A minor cycle is a similar phenomenon but with fewer degree days, meaning that the freeze/thaw is nearer the surface. This leads to chipping and separation of the asphalt.

Rainfall has two effects. First, precipitation during a freeze/thaw cycle can contribute to the severity of that cycle. Second, rain washes away unpaved shoulders, necessitating maintenance work.

Finally, snowfall is believed to have only a winter maintenance impact. The important statistic is the number of plowings. On the basis of Ontario experience, the total snowfall was divided by 2.5 cm to determine the number of times snow removal equipment would need to pass over the road.

The raw data used to compute the above parameters were obtained from Environment Canada. The information came from several hundred weather stations located throughout the province.

Scaling the Input Factors

To combine the four subfactors into one overall climatic factor, it is necessary to take some form of weighted total factor value. One potential problem of combining the input factors is the scale difference in the numbers. Cycles, for example, may number 1, 2, or 3 per year. Snowfall, however, may be 200 or 300 cm per year. In a linear programming framework (used in DEA), vast scale differences can cause roundoff problems and lead to erroneous results. It is desirable, therefore, for the scales of numbers to be relatively similar.

One important feature of the DEA model structure is its scale variance characteristic. For example, if snowfall is 100.5, 173.2, and 98.4 cm, the same efficiency measures would arise if the numbers 1005, 1732, and 984 were used. Therefore, regardless of the size of the raw data numbers, they can be adjusted (by a factor of 10, for example) up or down without destroying the meaning of the final results.

This being the case, all input factors can be expressed in roughly the same scale terms. No information is lost, and computational difficulties with the optimization procedure are avoided.

To transform the four inputs to similar scales, four weights (transformation parameters) were chosen:

$$\alpha = 50 \qquad \beta = 300 \qquad \gamma = 20{,}000 \qquad \delta = 1{,}000$$

In choosing these values, an attempt was made to reflect the perceived degree of importance of each parameter. Maintenance staff, for example, feel that major cycles have an important impact on spring road conditions while minor cycles have significantly less importance.

Beyond these two considerations (scale difference and perceived importance), the choice of transformation parameters was arbitrary for this phase of the study. The next section of this paper describes a more structured procedure for deriving parameters.

Rather than taking a weighted sum of the four climatic subfactors, a reciprocal model was used in this study. Specifically, the station factor F is computed as follows:

$$F = \frac{\alpha}{M_1} + \frac{\beta}{M_2} + \frac{\gamma}{S} + \frac{\delta}{R}$$

where

M_1 and M_2 = number of major and minor cycles, respectively,
S = number of snow plowings,
R = number of heavy rain days, and
α, β, γ, δ = weights.

The rationale for using reciprocals of the four data parameters is that, since F is to be an input, it should become smaller as the climate becomes more severe.

A typical calculation for a station is $M_1 = 1$, $M_2 = 2$, $S = 54.6$, and $R = 16$. Therefore,

$$F = \frac{50}{1} + \frac{300}{2} + \frac{20{,}000}{54.6} + \frac{1{,}000}{16} = 629$$

To get a patrol factor, those stations within and near the patrol boundaries were combined. In some instances, only one station could reasonably be used to represent a patrol. In those cases, the climatic factor for that station became the patrol factor. When more than one station was used for a patrol, a weighted average of the values for those stations was applied, and the station weights were taken as proportional to their distances from the center of the patrol.

WEIGHTING SUBFACTORS: A STRUCTURED APPROACH

One difficulty encountered in determining factor values, particularly accident and climatic factors, is that of arriving at appropriate weights for subfactor combinations. In the case of accidents, for example, a weight must be supplied to each of the stated surface conditions. Because there is no reliable data comparing the chances for an accident on ice and one on packed snow, weights must be primarily subjective.

One framework that can be used to obtain weights for a series of choices, options, or criteria is based on pairwise comparisons. In trying to determine the likelihood of an accident on each of the surface conditions, the only option may be to solicit expert opinions (for example, maintenance staff or police). The most convenient form in which to capture these opinions is by comparing pairs of options using a ratio scale. Specifically, the expert would be requested to supply a value a_{ij} where a_{ij} is the extent to which option i dominates option j. If, for example, $i \equiv$ packed snow and $j \equiv$ slush, then if $a_{ij} = 3.5$, an accident is 3.5 times as likely to occur on packed snow as on slush. Of course, if $a_{ij} = 3.5$, then

$$a_{ij} = \frac{1}{a_{ij}} = \frac{1}{3.5}$$

Thus, it can be argued that it is easier to supply such ratio-scale values as a_{ij} than to actually provide a numerical weight W_i (probability of an accident occurring on surface type i, for example).

A possible matrix A for all surface conditions might be

A	1	2	3	4	5	6	7
1	1	2	.5	.4	.3	.8	2
2	8	1	3	2	4	6	7
3	2	.33	1	2	.8	2	4
4	2.5	.5	.5	1	.8	3	3
5	3.3	.25	1.25	1.25	1	2	3
6	1.25	.17	.5	.33	.5	1	2
7	.5	.14	.25	.33	.33	.5	1

One property that a rational set of comparisons should possess is transitivity. Specifically, if option 2 is four times as likely as option 5 ($a_{25} = 4$) and option 5 is two times as likely as option 6 ($a_{56} = 2$), then it should be true that option 2 is eight times as likely as option 6 (that is, $a_{25} \times a_{56}$ should equal a_{26}). However, $a_{26} = 6$. Thus, the results are intransitive. This phenomenon is very common, since inconsistencies in reasoning are bound to happen in any situation.

To arrive at a set of consistent results that will lead to weights, various approaches can be taken. One of the simplest, as suggested by Barzilai et al. (*3*) and Crawford and Williams (*4*), is to use the geometric mean of row i to get weight W_i.

That is,

$$W_i = \left(\prod_{j=1}^{7} a_{ij}\right)^{1/7}$$

So, for the example,

$$W_1 = (1 \times 2 \times .5 \times .4 \times .3 \times .8 \times 2)^{1/7}$$
$$= .56$$

Similarly,

$W_2 = 3.61$
$W_3 = 1.36$
$W_4 = 1.24$
$W_5 = 1.34$
$W_6 = 0.62$
$W_7 = 0.37$

Note that these are relative weights. If they must add to 1 (for example, if they are to represent probabilities), then they would need to be normalized.

The above process gives a logical framework for deriving importance weights when subjective information must be considered.

The next section provides the results of a pilot study conducted in Ontario.

PILOT STUDY OF EFFICIENCY

The general structure of the DEA model was presented earlier. To illustrate how the model works, an example is provided of one patrol from district 2 in Ontario (the province is divided into 18 geographical districts). In the pilot study, the following output and input values were used for the patrol:

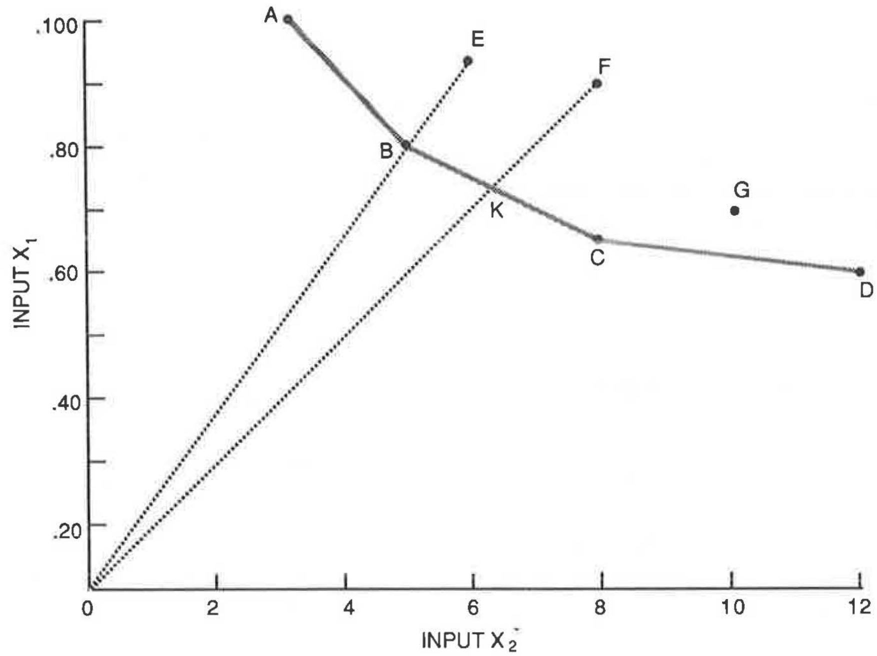

FIGURE 1 Efficient frontier.

Outputs:

1. Size of system = 404,
2. Traffic served = 267,
3. Condition rating factor = 184, and
4. Accident factor = 331.

Inputs:

1. Maintenance budget = 585,
2. Capital budget = 264, and
3. Climatic factor = 715.

The DEA model tries to determine the set of seven factor weights or multipliers (M_1, M_2, M_3, M_4 on outputs and N_1, N_2, N_3 on inputs) that makes this patrol's efficiency ratio as large as possible, while ensuring that the corresponding ratio for all other patrols does not exceed 1.0. This restriction limits the possible values that the multipliers M_i and N_j can assume. The patrol's efficiency ratio is as follows:

$$\frac{404M_1 + 267M_2 + 184M_3 + 331M_4}{585N_1 + 264N_2 + 715N_3}$$

The DEA model finds the set of multipliers that maximizes this ratio. For this particular patrol, the values of the seven multipliers are $M_1 = 206$, $M_2 = 308$, $M_3 = 1{,}747$, $M_4 = 720$, $N_1 = 209$, $N_2 = 103$, and $N_3 = 1{,}190$. The efficiency ratio is then

$$e = \frac{404 \times 206 + 267 \times 308 + 184 \times 1{,}747 + 331 \times 720}{585 \times 209 + 264 \times 103 + 715 \times 1{,}190}$$

$$= .725$$

Therefore, the best that can be said of this patrol is that its efficiency does not exceed 72.5 percent, compared with other patrols. That is, in the process of searching for multipliers M_i and N_j, no better set than the ones shown above can be found. In fact, some patrols must have a ratio of 1.0 relative to this set since this was the constraint imposed in deriving the multipliers.

Geometrically, this process can be illustrated as follows. Suppose there is only a single output (number of lane-kilometers serviced) and two inputs (maintenance budget and climatic conditions). Further, assume the patrols all service exactly 100 lane-km of road. On a two-dimensional graph, the pair of inputs for each patrol might be plotted as shown in Figure 1. Those points (patrols) closest to the origin are the most efficient since they involve the least amounts of inputs for the same level of output. Patrol E is, for example, less efficient than patrol B since B is using less of each input than E (to service the same size network). Patrols A, B, C, and D are considered efficient since there are no others closer to the origin that "dominate" them. However, patrol E is dominated by B while patrol F is dominated, in a sense, by patrols B and C. At least, a hypothetical patrol K could be defined whose inputs were linear combinations of those of B and C, then F would be dominated by K.

In summary, the DEA model would compute a ratio of 1.0 for patrols A, B, C, and D. The ratio of F would equal OK/OF. Thus, the "efficient frontier" made up of the line segments joining A, B, C, and D defines the highest level of efficiency obtainable. Anything on this frontier would have a ratio of 1.0 and would be considered efficient. Any patrol behind the frontier (E, F, and G) would have a ratio less than 1.0 and would be considered inefficient.

In the process of finding the best set of multipliers for patrol F (suppose F is patrol 1 in the above numerical example), the ratios for B and C would have been driven to 1.0, which would have limited the possible choice of multipliers for F. Thus, B and C are said to constitute the "peer group" for patrol F because they are the efficient patrols that are most like patrol F in terms of resource consumption (input values).

As an example of the likely results from a DEA of patrol

TABLE 2 RESULTS OF DEA

Patrol	Efficiency Rating	Peer Group
1	1.00	1
2	.99	1,4
3	.80	4,8
4	1.00	4
5	.86	1,4
6	.93	8
7	.89	4,8
8	1.00	8
9	.91	1,4
10	.72	1,4
11	.87	1,4,12
12	1.00	12
13	1.00	13
14	.62	1,4,12

efficiency, Table 2 displays the ratings and peer groups for the 14 patrols in the pilot district chosen for the Ontario study. This indicates that patrols 1, 4, 8, 12, and 13 are efficient (have a ratio of 1.0). The others are considered inefficient; some to greater degrees than others. For example, compared to the others, patrol 14 cannot be rated any higher than 62 percent. One interpretation of this number is that patrol 14 should be able to do better—either by servicing a larger network with the same resources or by consuming fewer resources.

CONCLUSIONS

In this paper, a model for examining maintenance patrol efficiency was presented, and relevant factors upon which to base this model were discussed. The model provides a way to calibrate the impact of various factors and gain a better understanding of the circumstances within which patrols operate.

This approach offers a framework for further investigation of a patrol's operations if the patrol appears inefficient. In addition, it can provide possible explanations for that inefficiency.

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Criteria for Selecting Desirable Quantities of Coal Tar Emulsion Seal Coat Components

SHAWN W. JENKINS, M. STROUP-GARDINER, AND DAVID E. NEWCOMB

The use of coal tar seal coats often creates difficulties in the field. In the research described by this paper, tests were developed or modified to measure workability (Brookfield viscosity), cure time (scuff resistance), skid resistance (scuff resistance), cracking (cyclic freeze-thaw conditioning), debonding (adhesion), and fuel resistance. Guidelines were established for determining the preliminary optimum quantities of additive, additional water, and sand for a given set of materials. These procedures are applicable to a wide variety of coal tar sources and types of additives. The procedure may also be used to refine optimum quantities after the preliminary analysis by reducing the range of variables. The reliability of these procedures will be tested on various field sections at the general aviation airport in Stead, Nevada.

Coal tar emulsion sealers have historically been used to protect asphalt concrete pavements from fuel, oil, water intrusion, and weathering. Because of the sealers' ability to resist fuel, they have been used extensively on airport taxiways and fueling areas. They are also used on automobile parking lots to resist motor oil drippage, which can soften asphalt concrete pavement. The sealers provide an impermeable surface to prevent water intrusion which can lead to raveling and stripping of the pavement. They also prevent weathering by protecting the pavement from sunlight and oxidation.

Sand is used with coal tar emulsions to enhance skid resistance. The level of skid resistance is influenced by the gradation and shape of the sand; therefore, a large, coarse sand is typically used. Sand loadings (i.e., quantities) have been increased in recent years in an attempt to provide an even rougher surface. However, the higher quantities of sand are difficult to keep suspended in the coal tar emulsions. Also, the sand-sealer interface has provided a path for petroleum products to penetrate the sealer.

Previous experimentation has shown that the use of latex polymeric additives in the coal tar emulsion can increase its ability to hold the sand in suspension (*1*). The latex also increases the sealer's flexibility. This flexibility allows the sealer to move with the underlying pavement as it contracts and expands due to thermal changes and traffic loads.

Although coal tar sealers have been used for many years, they have created some difficulties. Interviews with manufacturers, suppliers, contractors, and owners have identified several problems, including

- Workability (the ability to place the material),
- Cure time (when to open a new surface to traffic),
- Skid resistance,
- Cracking of the surface,
- Debonding of the sealer with the underlying pavement, and
- Fuel resistance.

Civil Engineering Department, University of Nevada-Reno, Reno, Nev. 89557.

METHODOLOGY

A review of the literature revealed a limited amount of research on the testing of coal tar emulsions used as seal coats on asphalt concrete pavements. The objective of this research was to evaluate and develop test procedures to define desirable properties of coal tar emulsions. This was accomplished by

- Identifying industries that use test methods relating to seal coat performance,
- Developing or modifying the identified test methods, and
- Evaluating the potential of the selected tests to define desirable properties of coal tar seal coats.

In addition to the coal tar industry, the paint, asphalt cement, asphalt concrete, and slurry seal industries were identified as having applicable or adaptable test methods. Tests chosen for evaluation or modification from these industries were

- Brookfield viscosity,
- Thomas-Stormer viscosity,
- Scuff resistance (ASTM D3910–84 and International Slurry Seal Association (ISSA) TB139 (*2*)),
- Cyclic freeze-thaw (*3*),
- Flexibility (ASTM D2939–78),
- Wet flow (shrinkage) (ASTM D2939–78),
- Measuring adhesion by tape test, Method A (ASTM D3359–83),
- Kerosene resistance (ASTM D3320–79), and
- Fuel drip followed by the wet track abrasion procedure (*4*).

INITIAL FIELD TEST SECTIONS

Before starting the laboratory testing program, major coal tar suppliers were invited to place field test sections on the University of Nevada-Reno (UNR) campus. The test sections were placed on a parking lot that experienced low traffic

TABLE 1 FIELD TEST SECTION FORMULATIONS

Section	Prime Coat	No. of Base Coats	Top Coat w/out Sand	Quant. Coal Tar, gal.	Quant. Additive, gal.	Quant. Water, gal.	Quant. Sand, lb/g CT*
1	No	2	Yes	100	8.2	80	13
2	No	1	Yes	100	8.2	80	13
3	Poly oil & water	2	Yes	100	8.2	80	13
4	Yes	2	No	80	--	20	4
5	No	2	No	asphalt emulsion (20% cut)			
6	No	2	No	15% coal tar, 85% asphalt emulsion			
7	No	2&3	No		Fass - Dri		5.4
8	No	2	Yes	100	25	25	10
			Top Coat	100	10	25	---
9	Water	2	Yes	100	10	20	5
			Top Coat	100	10	25	---
10	J220	2	No	100	10	20	5
11	No	2	No	100	10	20	5
12	No	2	Yes	100	4	40	2
13	No	2	Yes	100	6	50	6
14	No	2	Yes	100	5	40	4
15	No	2	Yes	100	7	50	8
16	Yes	2	No	100	15	45	7
17	Yes	2	No	100	10	90	6.2

* - Coal Tar Emulsion

volume so that weathering effects could be monitored without the influence of traffic loads. The parking lot, which was approximately 8 mo old, provided a large, uniform surface for the application of the test sections.

Field samples were collected for laboratory cyclic freeze-thaw analysis by taping asphalt roofing shingles to the pavement prior to test section application. The samples were removed and returned to the laboratory after 24 hr of field curing.

Seventeen field test sections of varying sizes were placed by four suppliers between September 9th and 30th, 1986. The mix formulations of these test sections can be found in Table 1.

The test sections were visually monitored once a month for crack development. The following scale was developed to rate cracking:

0 = No cracking,

1 = Hairline cracking,

2 = Slight cracking,

3 = Moderate cracking, and

4 = Severe cracking.

Examples of these ratings are shown in Figure 1. This was the only testing performed at the field test site.

LABORATORY TESTING

Laboratory testing was conducted in two phases. Phase 1 included preliminary test method evaluation, while phase 2 consisted of modifying or refining the test procedures.

Phase 1

Phase 1 included the following three test stages:

1. Coal tar emulsion;
2. Coal tar emulsion, water, and additive; and
3. Coal tar emulsion, water, additive, and sand.

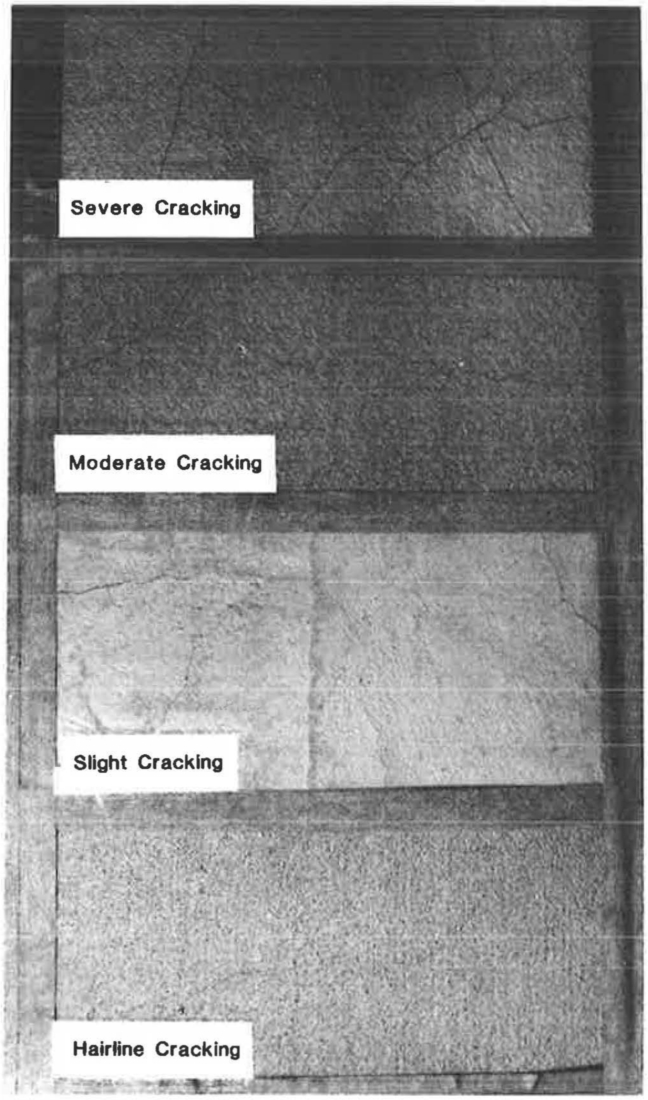


FIGURE 1 Crack rating scale developed for cyclic freeze-thaw conditioning test.

The variables considered during testing were

- Coal tar source,
- Additive content,
- Water content,
- Sand content,
- Sand gradation, and
- Sand shape.

In stage 1 testing, coal tar source was the only variable considered. The tests performed in this stage are shown in Figure 2.

The testing in stages 2 and 3 was performed according to designed experimental plans. The plan used in stage 2 was a three-factor, full factorial experiment with three levels for each factor. Source, additive quantity, and water quantity were the three factors investigated. Each factor consisted of low, medium, and high levels. The low and high limits were determined from the absolute lowest and highest manufacturer-recommended limits on the variables. The medium limit was the average between the low and high limits. Due to the large number of formulations that would result if the testing of stage 3 were conducted from a full factorial design, this plan was reduced to a partial factorial experiment with two levels for each factor. Sand gradation, sand shape, additive content, water content, and sand content were the variables considered. The two levels considered were low and high, and they were selected as described above.

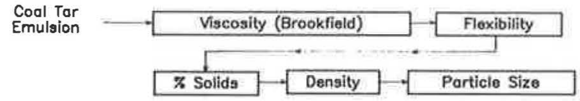


FIGURE 2 Test sequence for stage 1 of phase 1 testing.

The tests performed in stages 2 and 3 are shown in Figure 3.

Phase 2

Phase 2 was conducted by following a four-factor, full factorial experiment with three levels for each factor except sand loading, which had two levels (see Table 2). The variables, or factors, that were considered included

- Coal tar source,
- Additive content,
- Water content, and
- Sand content.

The tests performed in this phase are shown in Figure 4.

Several tests were eliminated after the phase 2 results were reviewed. The wet track abrasion procedure was dropped because it did not provide reliable results and did not indicate mix component changes. The Thomas-Stormer Viscometer, which was used to indicate settling, was also rejected. Because of the higher sand loadings, the results from the settling test were limited. After addition of the large quantity of sand, the Thomas-Stormer paddle, which is driven by weights, was unable to rotate in the mixture with the maximum weight applied. In addition, the tile fuel resistance test was eliminated. Test results indicated that, although this method could possibly discern overall fuel resistant mixtures, it was not sensitive to changes in the components of the mixtures. If this method is included in further testing, it should only be used as a final pass/fail step.

TEST METHODS AND RESULTS

All of the test methods used coal tar emulsion with sand, which is referred to as the composite system in this paper. Only the viscosity test was used for both the composite system and the coal tar emulsion, additive, and addition water (total liquids), combination. Desirable limits were established for each test method on the basis of a review of the results and extensive visual observation.

Viscosity

Test Method

Viscosity was measured using the Brookfield Viscometer DV II (see Figure 5). The Brookfield was chosen because of its

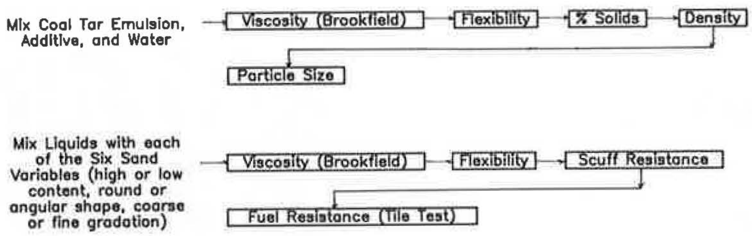


FIGURE 3 Test sequence for stages 2 and 3 of phase 1 testing.

TABLE 2 VARIABLE LEVELS USED IN EXPERIMENT

Variable	Code	Quantity or Description
Additive	L	4.0 g/100 g coal tar emulsion
	M	14.5 g/100 g coal tar emulsion
	H	25.0 g/100 g coal tar emulsion
Water	L	20.0 g/100 g coal tar emulsion
	M	55.0 g/100 g coal tar emulsion
	H	90.0 g/100 g coal tar emulsion
Sand	L	2 lbs/g coal tar emulsion
	H	13 lbs/g coal tar emulsion

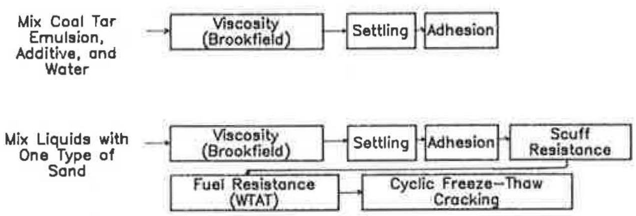


FIGURE 4 Test sequence for phase 2 testing.

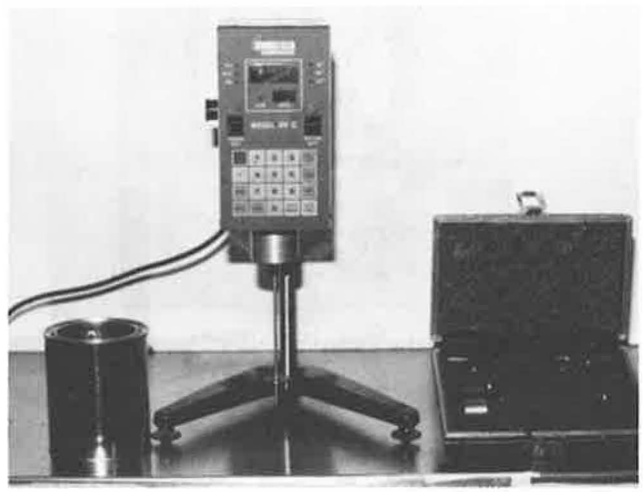


FIGURE 5 Brookfield Viscometer.

ease of use and wide range of measuring capabilities. After initial testing, it was found that coal tar emulsion mixtures exhibited shear thinning characteristics (*5*). Therefore, the testing procedure was controlled as follows.

The coal tar emulsion and water were mixed with 50 strokes of a large laboratory mixing spoon. A Brookfield spindle was then inserted into the material and allowed to rotate for 5 sec at a shear rate of 50 r/min before a viscosity measurement was taken. The additive was then introduced and stirred with an additional 50 strokes, and the viscosity was measured as before. This procedure was repeated for the addition of sand.

Repeatability

An examination of the standard deviations versus the viscosity in poises indicated a nonlinear relationship. Therefore, the coefficient of variation (CV) was chosen to represent repeatability. This statistical parameter is actually an expression of the standard deviation as a percentage of the mean viscosity. Three replicates of six materials were used to determine the CV for both the total liquids and the composite system. The CV was 3.7 percent for the total liquids and 8.0 percent for the composite system.

To find the standard deviation for any viscosity, the viscosity is multiplied by the CV (with the percentage expressed in decimal form). For example, if the testing of three samples of total liquids yields an average viscosity of 50.0 poises, then the standard deviation is $50.0 \times .037 = 1.9$ poises.

Desirable Test Limits

Desirable viscosity limits were established by evaluating the laboratory tests, a visual observation of ease of mixing, the consistency of the material, and the ability of a technician to prepare samples (*5*). When preparing samples, materials with viscosities of less than 10 poises were found to be too fluid; sands rarely stayed in suspension. These materials would tend to run off the pavement if used in the field. On the other hand, viscosities of greater than 90 poises were accompanied by one or more of the following:

- Obvious coagulation,
- Lumping,
- Inability to spread material, and
- A thick layer at the bottom of the container, indicating that the additive or the sand was thickening or settling out.

Thus, these materials would cause an uneven surface texture if squeegied and would clog spray nozzles.

Scuff Test

Test Method

The procedures and equipment for the scuff test were developed from the asphalt concrete and slurry seal industries (ASTM

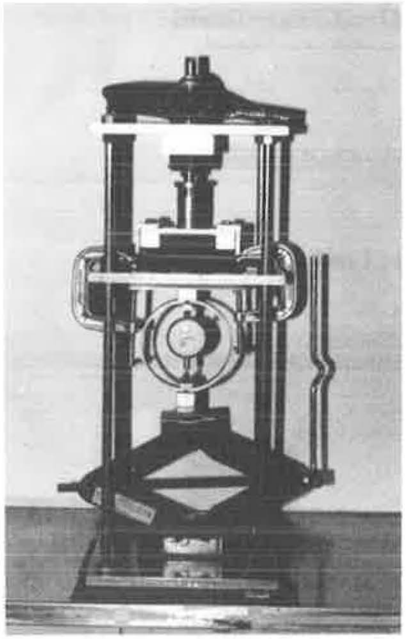


FIGURE 6 Scuff test apparatus.

D3910–84 and (*2*)). The equipment applied a constant pressure to the test specimen, then rotated a rubber abrasion foot on the specimen. The torque required to turn the foot was then measured (see Figure 6).

The sample medium used was asphalt roofing shingles, cut into 6-in. × 6-in. squares; three samples were needed for each test. A uniform film thickness of the composite system was applied using a 16-gauge sheet metal mask: a 6-in. × 6-in. square with a 4-in. × 4-in. section removed from the center. A straightedge was used to apply the material evenly within the cut-out section.

All prepared samples were allowed to cure at ambient temperature (77°F) until they were tested. One sample was tested at 4 hr, the second at 8 hr, and the last at 24 hr.

During testing, samples were held in place on the platen with "C" clamps. The platen was raised upward to the rubber abrasion head, and a normal load of 28 psi was applied. The torque wrench was then pulled through an arc of 180°, and a torque reading was taken in inch-pounds. A torque wrench with a capacity of 300 in.-lb provided an adequate range for all testing.

Repeatability

Testing to determine the repeatability of this method indicated that the standard deviation between any two tests was 13.1 in.-lb. This value was rounded up to 15 in.-lb because the torque wrench measured in increments of 5 in.-lb. The standard deviation was consistent for any test time or range of torque values.

Desirable Test Limits

Torque readings below 50 in.-lb caused material to be pushed in front of the rubber abrasion head. Values of 80 in.-lb or greater at 4 hr with a reduction in values at 8 hr also indicated that the material was moving on the shingle. The high initial readings were the result of testing the shingle and not the seal coat; as the material set (8 hr), the test began to evaluate the seal coat instead of the shingle.

Torque readings between 50 and 100 in.-lb were equated with material shearing under the abrasion head. Some of the seal coat remained adhered to the shingle, but the surface of the seal coat tended to push in front of the abrasion head.

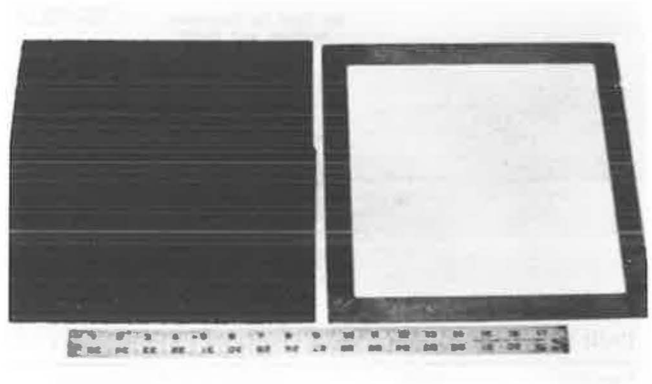


FIGURE 7 Asphalt roofing shingle and mask for cyclic freeze-thaw conditioning test.

On the basis of these observations, 8- and 24-hr limits were set as follows:

- A torque of a minimum of 100 in.-lb at 8 hr, and
- A torque greater than the 8-hr reading at 24 hr.

The limit on the 24-hr reading ensured that the 8-hr reading was actually measuring the seal coat and not the shingle.

Cracking Tendencies

Test Method

The temperatures used for the freeze-thaw cycles were derived from typical asphalt concrete testing procedures. This testing was based on typical northern pavement temperatures of 140°F or above during the summer and 10°F during the winter.

Composite systems were applied to a 12-in. × 12-in. section of asphalt roofing shingles. One layer of sealer was applied in a uniform film thickness using a 16-gauge sheet metal mask, which was 12-in. × 12-in. with a 10-in. × 10-in. section removed from the center (see Figure 7). After the sample was prepared, it was cured at 77°F and at a relative humidity of less than 20 percent for 24 hr. Samples were then exposed to a 140°F oven for 24 hr and a 10°F freezer for 24 hr. Samples were conditioned for 10 cycles, each consisting of one treatment of both temperatures. Cracking was monitored after the completion of each cycle. The same scale was used for these evaluations as for the field test sections (see Figure 1).

Repeatability

Various materials with diverse cracking tendencies were evaluated to determine the repeatability of this test. In all but a

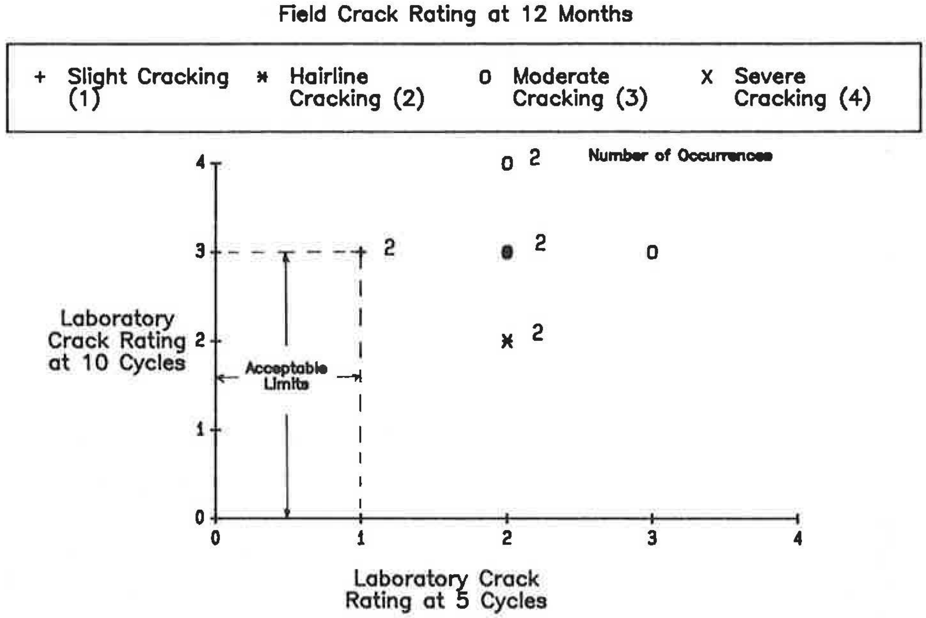


FIGURE 8 Relationship between laboratory freeze-thaw cracking and field test section cracking.

few cases, the ratings for replicates of the same material were identical. A calculation of the standard deviation for this test method was 0.29.

Desirable Test Limits

Using a comparison between field cracking of test sections and freeze-thaw cracking of laboratory samples as a basis, rating limits were chosen as follows:

- A rating of 1 or less at the end of 5 cycles, and
- A rating of 3 or less at the end of 10 cycles.

The relationship used to select these ratings is shown in Figure 8. This figure shows laboratory cracking at 10 cycles versus laboratory cracking at 5 cycles, with the symbols indicating the results of the field crack rating at 12 mo for each sample. These limits were based on field evaluations to date and have produced a crack rating of 1 or less after 1 yr in the field. Comparisons of 11 test sections comprising a wide range of coal tar sources, additives, and sand gradations and shapes were the basis for these ratings. Typical relationships between field cracking and laboratory conditioning are shown in Figures 9 and 10. It should be noted that the same crack rating system was used for both the laboratory and the field evaluations.

Adhesion

Test Method

ASTM D3359–83 describes the detailed use of the adhesion test procedure. Basically, one thickness of the composite system was placed on an aluminum panel, and the sample was cured at 77°F for 24 hr. After curing, an "X" was cut in the seal coat so that the panel was visible. A length of pressure-sensitive tape was applied so that the center of the X was covered, the tape was peeled back, and the adhesion between the sealer and the panel was rated. The ASTM rating scale, was modified for this research as follows:

5A = No peeling,
4A = Trace peeling or removal along the incision,
3A = Jagged removal along most of the incision up to 1/16 in. on either side,
2A = Jagged removal along most of the incision up to 1/8 in. on either side,
1A = Removal from most of the area of the X under tape, and
0A = Removal beyond the area of the X.

A plus sign (+) was added to indicate that sand was retained on the tape.

Repeatability

Repeatability was not established for this test method.

Desirable Test Limits

Most products tested indicated no peeling; however, at the higher sand contents, most samples demonstrated a loss of sand retention. Therefore, limits were set at a rating of 5A with no sand being retained on the tape.

USE OF DESIRABLE PROPERTIES TO DEFINE OPTIMUM COMPONENT QUANTITIES

Preliminary optimum component quantities were defined by a process of elimination based on the limits set for each test

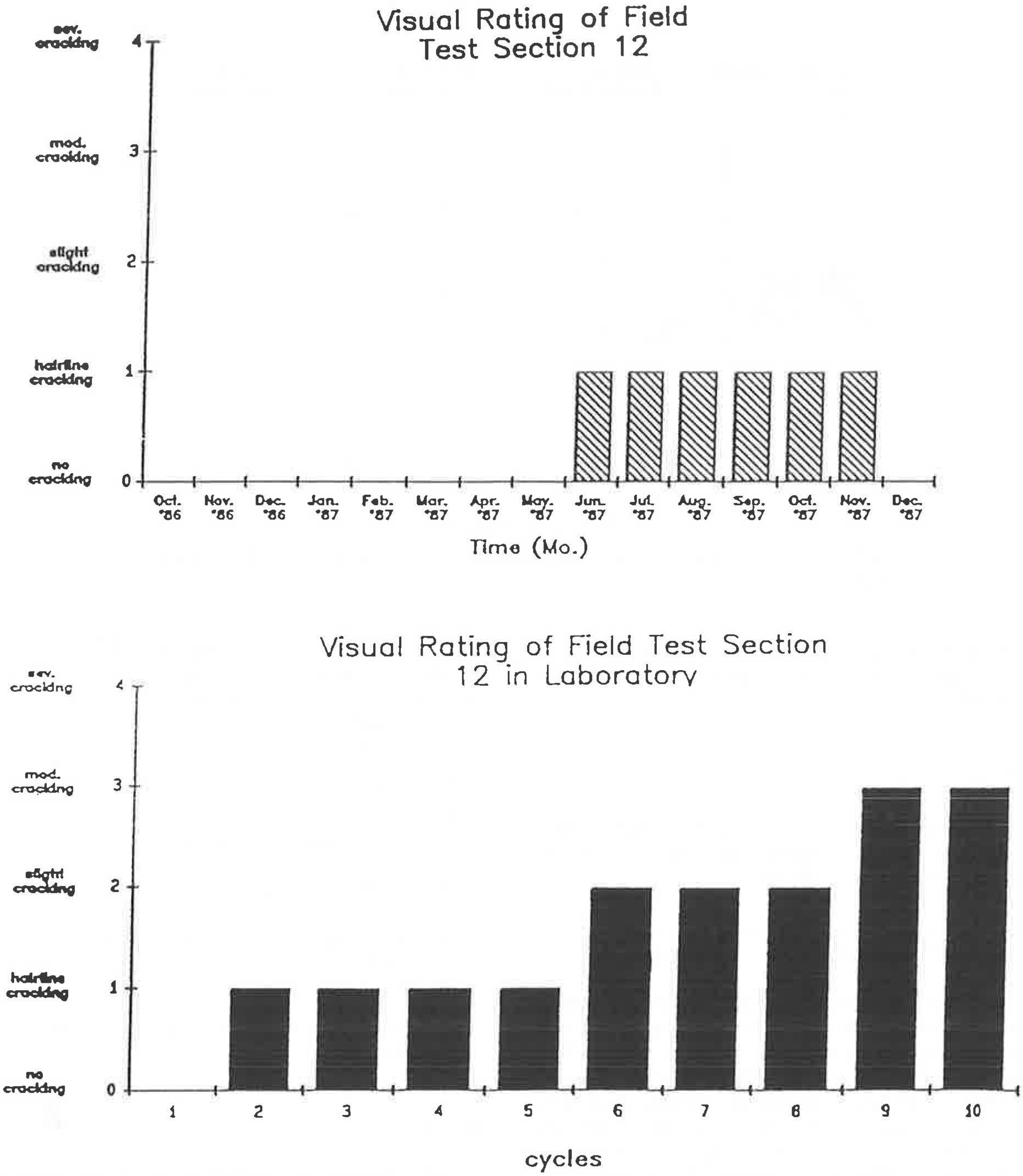


FIGURE 9 Typical relationship for field cracking versus laboratory cracking (test section 12).

(see Table 3). An example for coal tar source 2 is shown in Figure 11. Five steps were used for this process of elimination.

Step 1

In this step, incompatibilities were identified between the components making up the liquid portion of the sealers. The following criterion was considered:

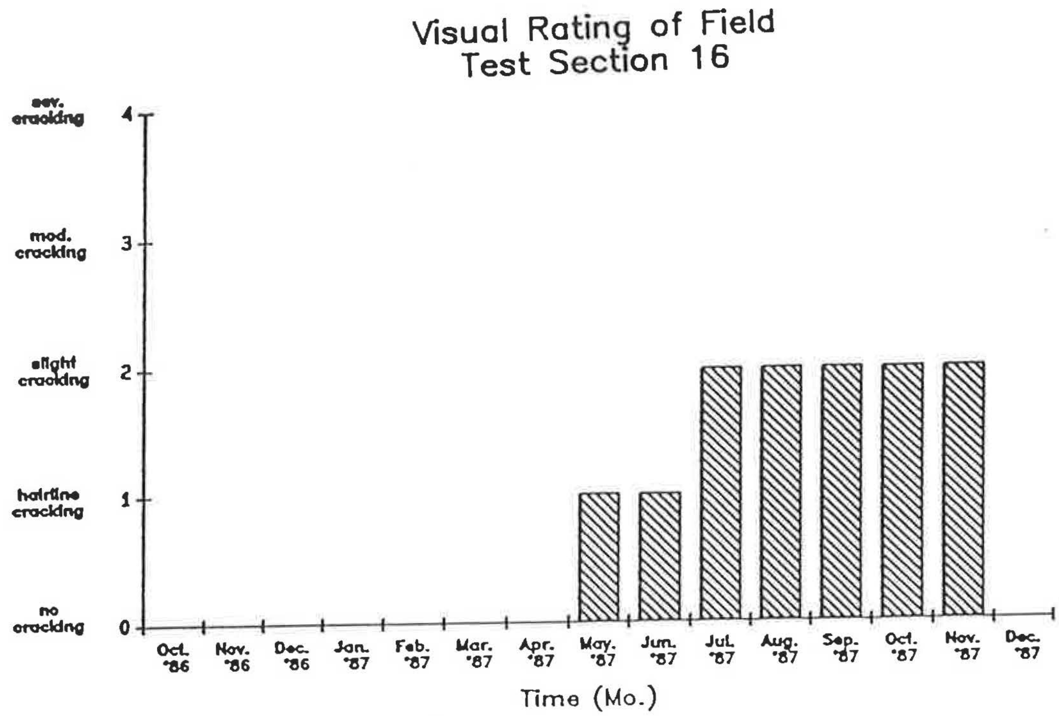
- Viscosities between 10 and 90 poises are acceptable.

Any mixtures not meeting this requirement were eliminated from the matrix for the next step. Figure 11 shows that all mixtures except the low water/low additive and low water/medium additive were eliminated.

Step 2

This step checked the workability of the mix by identifying any new incompatibilities created by the introduction of sand. The composite material could neither run off the pavement nor clog spray bars. The following limit was used:

- Viscosities between 10 and 90 poises are acceptable.



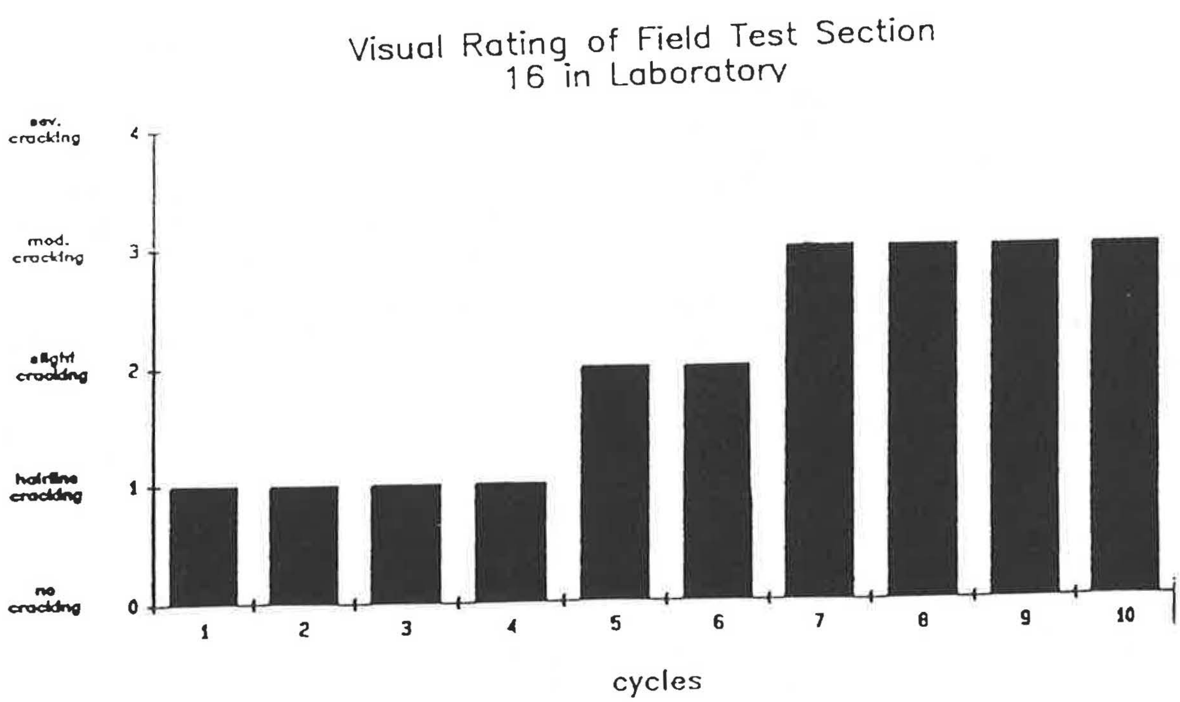


FIGURE 10 Typical relationship for field cracking versus laboratory cracking (test section 16).

TABLE 3 DEVELOPMENT OF PRELIMINARY OPTIMUM COMPONENT QUANTITIES

Step	Test Method	Performance Item	Criterion	Repeatability
1	Brookfield Viscosity @ 77°F	Incompatibility between additive and coal tar	Viscosity between 10 and 90 poises	CV = 3.7%
2	Brookfield Viscosity @ 77°F	Workability of mix	Viscosity between 10 and 90 poises	CV = 8.0%
3	Scuff Resistance	Rate of set	8 hour torque ⩾100 in-lbs	Std Dev = 15 in-lbs
		Final scuff resistance	24 hour torque ⩾8 hour torque	Std Dev = 15 in-lbs
4	Cyclic Freeze-Thaw Conditioning	Cracking	Rating⩽1 @ 5 cycles Rating⩽3 @ 10 cycles	Std Dev = 0.29
5	Tape Test	Adhesion	Rating = 5A No sand loss	N/A

Any mixtures not meeting this requirement were eliminated in the step 3 matrix.

Step 3

In this step, the initial set and final scuff resistance were checked. The seal coat was allowed to set for a maximum of 8 hr. The torque value was checked at 24 hr to ensure the best final scuff resistance for the materials used. The limits for this step were as follows:

- Torque ≥ 100 in.-lb at 8 hr, and
- Torque ≥ the 8-hr value at 24 hr (a small difference in numbers was tolerated as long as it remained within the realm of repeatability error).

The results from this step usually narrowed the acceptable combinations of components to approximately four to six mixtures. Those not meeting the requirements were eliminated from the step 4 matrix.

Figure 11 shows the 8-hr torque value in the upper left-hand corner of the cell and the 24-hr cure in the lower right-hand corner. It should be noted that the 8-hr torque value (85 in.-lb) for the medium additive, low water, and low sand mixture was left in the test matrix. Any scuff test result that was within the repeatability error was given a chance to pass the remainder of the requirements.

Step 4

The purpose of step 4 was to optimize long-term performance by limiting both the 5- and 10-cycle cracking. The following criteria were used:

- A rating of 1 or less at 5 cycles, and
- A rating of 3 or less at 10 cycles.

Figure 11 shows that only the medium additive, low water, and high sand mixture met these criteria.

Step 5

This step was used as a pass/fail test for adhesion and sand retention. Sand had to be retained by the seal coat after curing. The following limits were considered:

- No sand can adhere to the tape, and
- No debonding of the seal coat and the test medium is allowed (adhesion rating of 5A).

The only selection that met the freeze-thaw requirement also met the adhesion/sand retention check.

In general, this methodology indicated that the optimum combination of the variables investigated was coal tar source 2: a medium additive with low additional water and a high sand loading.

COMPARISON OF DESIRABLE LIMITS FOR TEST RESULTS AND SUPPLIERS' SUGGESTED OPTIMUM MIXTURES

Table 4 provides a comparison of the optimum component quantities as defined by desirable test results, before and after the sand retention check, and the corresponding suppliers' suggestions. The quantities were compared before and after the sand retention check because of the wide range between the high and low sand loadings. In other words, a mix might

TOTAL LIQUIDS

Step 1 - Check mix for incompatibility between coal tar and additive

		Additive		
		Low	Med.	High
Water	Low	29.1	30.4	4.9
	Med.	7.3	3.2	2.7
	High	2.2	2.0	low

LIMITS: Viscosity between 10 and 90 poises

COMPOSITE MIX

Step 2 - Check workability of mix

		Additive								
		Low			Medium			High		
		Water			Water			Water		
		L	M	H	L	M	H	L	M	H
Sand	L	35.1	X	X	35.6	X	X	X	X	X
	H	60.6	X	X	43.4	X	X	X	X	X

Viscosity between 10 and 90 poises

Step 3 - Check initial set and final scuff resistance

		Additive								
		Low			Medium			High		
		Water			Water			Water		
		L	M	H	L	M	H	L	M	H
Sand	L	150 110	X	X	85 120	X	X	X	X	X
	H	150 125	X	X	100 115	X	X	X	X	X

8 hour torque ⩾ 100 in-lbs.

24 hour torque > 8 hour torque

Step 4 - Limit crack development

		Additive								
		Low			Medium			High		
		Water			Water			Water		
		L	M	H	L	M	H	L	M	H
Sand	L	X	X	X	3 4	X	X	X	X	X
	H	X	X	X	0 1	X	X	X	X	X

Rating ⩽ 1 @ 5 cycles

Rating ⩽ 3 @ 10 cycles

Step 5 - Check adhesion, between mix and substrate and between sand and binder

		Additive								
		Low			Medium			High		
		Water			Water			Water		
		L	M	H	L	M	H	L	M	H
Sand	L	X	X	X	X	X	X	X	X	X
	H	X	X	X	5A,N	X	X	X	X	X

Adhesion rating = 5A

Loss of sand -(Y/N)

FIGURE 11 Selection of desirable properties for coal tar source 2.

perform well in all of the tests but fail to retain sand because of the high sand loading.

Table 4 shows that the procedure developed in this research raised the additive content compared with the suppliers' recommendations for all sources. The water content remained constant for two sources, was increased for one source, and was decreased for one source. In general, the sand content was increased before the sand retention check and decreased afterward.

It should be noted that UNR component quantities were chosen from the limits used to define desirable test results. No interpolation was made between low, medium, and high component quantities.

The process developed is only a preliminary estimation of component quantities based on a wide range of component levels. After the preliminary quantities have been found, another estimate of component quantities should be performed to refine the optimum quantities. This process would be identical to the preliminary analysis but would consider a narrower range of variables. Due to limited time and money, only the general practicality of this methodology was assessed in this study.

SUMMARY

On the basis of previous difficulties experienced with coal tar seal coats, tests were developed or modified to measure

- Workability (Brookfield viscosity),
- Cure time (scuff resistance),
- Skid resistance (scuff resistance),
- Cracking (cyclic freeze-thaw conditioning),
- Debonding (adhesion), and
- Fuel resistance.

Viscosity was selected to detect two initial problems. The first was an incompatibility between the components, which causes coagulation and an inordinate amount of thickening of the emulsion. Both of these create an increase in viscosity. Second, viscosity was used to measure the ease with which the material could be squeegied or sprayed.

A scuff test, adapted from the slurry seal and asphalt concrete industry, was designed by the University of Nevada-Reno. Limits for scuff values were set at 8 hr to provide a substantially scuff-resistant surface 8 hr after placing mate-

TABLE 4 COMPARISON OF OPTIMUM QUANTITIES DETERMINED FROM LABORATORY TESTING AND SUPPLIERS' RECOMMENDATIONS

Source/Components	Before Sand Retention Check (Steps 1-4)	After Sand Retention Check (Steps 1-5)	Supplier's Recommended Quantities
Source 1:			
Water*	55	20	80
Additive*	14.5	14.5	8.2
Sand**	13	2	13
Source 2:			
Water	20	20	20
Additive	14.5	14.5	10
Sand	13	13	5
Source 3:			
Water	20	20	50
Additive	25	25	6
Sand	13	13	6
Source 4:			
Water	90	90	90
Additive	25	25	6
Sand	13	2	6.2
Source 6:			
Water	90	90	60
Additive	4	4	3
Sand	2	2	8

* - Quantity measured in gal/100 gal coal tar emulsion
** - Quantity measured in lbs/gal coal tar emulsion

rials. A minimum scuff value was established at 24 hr to indicate optimum scuff resistance for any given set of components.

Cracking was evaluated by applying coal tar emulsions to roofing shingles, then subjecting the prepared samples to multiple cycles of freezing and thawing. Limits on cracking were set for this testing at 5 and 10 cycles of freeze-thaw. These limits were linked to field performance.

The cross-hatch test was used to identify debonding of the sealer from the test medium as well as loss of sand retention by the sealer.

Guidelines were established for determining the preliminary optimum quantities of additive, additional water, and sand for a given set of materials. These procedures are applicable to a wide variety of coal tar sources and types of additives. The procedure may also be used to refine optimum quantities after the preliminary analysis by reducing the range of variables.

FUTURE RESEARCH

The tests and limits developed in this study will be used to define the quantities of composite system components for field test sections at the general aviation airport in Stead, Nevada, which is located several miles outside Reno city limits. The materials used for these test sections will be similar to those supplied for the original field test on the UNR parking lot. Because the test methods and limits were refined with these specific materials, continuity between the initial field work, preliminary and final laboratory testing, and the final test sections will be maintained. Materials will include

- Six sources of coal tar emulsions;
- Various additives, including acrylonitrile-butadiene latex and proprietary products; and
- One sand source and gradation.

Sand source and gradation were held constant to reduce the variables in the laboratory portion of this research. Because the optimization steps did not account for various sand sources and gradations, the field mixtures will also be restricted to this sand and gradation.

ACKNOWLEDGMENTS

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Prioritization and Optimization of Pavement Preservation Treatments

J. J. HAJEK AND W. A. PHANG

This paper describes a framework for selecting the best pavement preservation treatments for an available pavement preservation budget. It includes formulation of project-specific strategies, evaluation of funding requirements, and setting of priorities. The technology is illustrated using data for 75 sections from the Stratford district. The key component of this framework is an action plan that recommends preferred and fall-back pavement preservation strategies for all individual pavement management sections. The task of preparing action plans is assigned to experienced regional staff. The action plan documents the existing pavement condition, integrates all major pavement maintenance and rehabilitation efforts into a unified preservation plan, and coordinates pavement preservation functions of different offices within the Ontario Ministry of Transportation. Linear programming is used to allocate pavement investments in a manner that yields maximum benefits to the total pavement network. The linear programming solution considers all section-specific strategies listed in the action plans. Sensitivity analyses are used to evaluate the effect on the linear programming solution of using different optimization goals and different budget constraints. While the objective function used only maximizes the technical benefits of pavement investments, it can be modified to include societal benefits.

The pavement management process can be roughly divided into three related phases:

1. A data gathering phase, in which the data required to make pavement preservation decisions are collected and stored;
2. A decision phase, in which pavement inventory data are used to make recommendations regarding pavement preservation actions; and
3. A feedback phase, in which the consequences of pavement investments are evaluated.

This paper is mainly concerned with the second phase. Its objective is to describe the methodology for recommending pavement preservation actions that was developed for the Ontario Ministry of Transportation's pavement management system. This methodology consists of the following steps:

1. Development of a preferred pavement preservation strategy, as well as alternative fall-back strategies, for every pavement management section;
2. Aggregation of funding requirements for individual pavement sections and their evaluation on district, regional, or provincial levels; and
3. Selection of pavement preservation strategies that would best use the available budget.

Research and Development Branch, Ministry of Transportation of Ontario, 1201 Wilson Avenue, Downsview, Ontario, Canada.

Step 1 represents project-level evaluation, while steps 2 and 3 represent network-level evaluation and prioritization. The application of the methodology is illustrated using recent data obtained for the Stratford district.

Any methodology for recommending pavement preservation actions depends on the amount and quality of inventory data. For this reason, the current method of gathering and storing pavement inventory data is briefly described.

DATA REQUIRED FOR PAVEMENT PRESERVATION DECISIONS

To facilitate planning of pavement rehabilitation actions, the Ministry of Transportation has been systematically rating pavement deterioration since the mid-1960s. The original rating scheme was based on a subjectively assigned pavement condition rating (*1*). It was realized in the 1970s that pavement deterioration should be evaluated using a more objective and consistent measure. This led to the development and recent introduction of the Pavement Condition Index (PCI) (*2*).

The PCI is measured on a scale of 0 to 100. Newly constructed pavements have a PCI of about 95, and rehabilitation is usually done when the PCI is between 60 and 40. The PCI comprises two different physical parameters:

1. The riding quality of the pavement surface measured by a response-type pavement roughness meter, and
2. The extent and severity of 15 pavement surface distresses evaluated against well-defined measurement scales.

These two parameters are combined using a mathematical formula.

The basic pavement management unit is a pavement management section. Pavement management sections have relatively uniform pavement structure and traffic loadings, and exhibit relatively uniform pavement deterioration. The typical length of pavement management sections is approximately 10 km.

The pavement condition surveys used to determine the PCI are done every 2 yr, and all data obtained during these surveys are stored as historical records in a pavement management data base. These include roughness data, the severity and density of 15 pavement surface distresses, and the PCI. The data base operates under the FOCUS data base management system (*3*) and contains data for approximately 2,000 pavement sections. The data base was designed to store and retrieve all basic data needed to plan specific pavement preservation treatments for individual sections.

Examples of data that can be readily retrieved from the pavement management data base are shown in Tables 1 and 2. Table 1 is an example of a pavement performance record, which summarizes past pavement deterioration data, for a section in the Stratford district.

Table 2 shows an action plan fact sheet for the same section. This fact sheet consists of two parts. The first part summarizes pavement inventory data, such as pavement structural data, functional class, and past performance. The second part lists two key estimates:

1. The expected change in PCI during the next 3 yr (ΔPCI_3). This estimate provides a systematic measure of the expected rate of pavement deterioration. It is based on the assumption that no preservation action, other than routine maintenance, will be taken during the 3-yr period. ΔPCI_3 is used for the section-specific pavement deterioration prediction to estimate when the pavement performance will reach the minimum acceptable level and when a pavement preservation action will be needed.
2. The action plan, which lists all pavement preservation actions planned within a 5-yr planning period.

The action plan is prepared for all sections. It is the key pavement management tool that documents the existing pavement condition, integrates all major pavement maintenance and rehabilitation efforts into a unified preservation plan, and coordinates pavement preservation functions of different offices within the ministry (such as district maintenance, regional rehabilitation, and Head Office policy and funding functions).

PROJECT-LEVEL PRIORITIZATION

The tasks of predicting the expected rate of pavement deterioration and recommending pavement preservation strategies for action plans are assigned to experienced regional staff. These individuals are in constant contact with the portion of the highway network over which they have responsibility and can fully exercise their engineering judgment and knowledge of local conditions. They also work closely with district staff, who are in charge of pavement maintenance, to coordinate and plan pavement maintenance. To help the regional staff, detailed guidelines have been prepared for these tasks (*4*). Salient features of the guidelines for recommending pavement preservation strategies are briefly described below.

These guidelines provide a structured procedure on how to identify, for each section:

- A preferred pavement preservation strategy, and
- Its alternative, fall-back (or contingency) strategies.

The preferred pavement preservation strategy is defined as the one that addresses in the most cost-effective way the problem of keeping the pavement in an adequate state of pavement preservation. It encompasses both major maintenance treatments (such as patching) and rehabilitation treatments (such as overlays), taken singly or in combination with each other. The preferred strategy must take into account existing resources and procedures and is usually based on life-cycle economic analysis. Preferred strategies are usually recommended for implementation before the pavement reaches a minimum acceptable serviceability level.

The fall-back strategies provide recommendations of what to do if a postponement or rescheduling of expenditures is required for the preferred strategy. Compared with the preferred strategy, the fall-back strategies would likely result in lower pavement performance or higher construction costs.

The fall-back strategies should provide "substantial" alternatives to the preferred strategy. Small variations in the preferred strategy, such as a change in overlay thickness by 20 percent or a postponement of the overlay by only 1 yr, do not usually constitute substantial alternatives.

Fall-back strategies should be developed that

- Provide a substantial postponement of expenditures,
- Enable a systematic assessment of the consequences of not implementing the recommended preferred strategies on time, and
- Enable comparisons of these consequences among different sections.

Using a structured format, two types of fall-back strategies are developed: holding strategies and deferred strategies.

A holding strategy is designed to hold the pavement for at least 2 or 3 yr until the preferred strategy can be undertaken. For example, according to the action plan given in Table 2, the preferred strategy calls for a 90 mm overlay in 1989. The corresponding holding strategy recommends patching in 1988 and the same 90 mm overlay in 1991. The patching component of the holding strategy is intended to hold the pavement until 1991 and to postpone the major part of the expenditure from 1989 to 1991.

A deferred strategy assumes that it is necessary to defer all expenditures associated with the preferred strategy by at least 3 yr if the preferred strategy is a rehabilitation treatment. If the preferred strategy only recommends a maintenance treatment, the deferral may be just 1 or 2 yr. Returning to the example in Table 2, the deferred strategy assumes that no funds (other than those for routine maintenance) are available until 1992 and, based on this constraint, recommends deferring padding and resurfacing until that time. The deferred strategy should address the new situation in the most cost-effective manner. The expected pavement performance curves for all strategies listed in Table 2 are shown in Figure 1.

By default, all sections also have do-nothing strategies, which assume that no pavement preservation expenditures (other than those for routine maintenance) will be made during the 5-yr planning period. The consequences of do-nothing strategies can be judged by the ΔPCI_3 estimates.

Not all sections require the full palette of preservation strategies for the 5-yr planning period. For example,

- Many sections do not require any specific preservation treatment other than routine maintenance, which is not included on the action plans;
- Some preferred strategies, particularly those scheduled for the beginning of the planning period, are already approved and do not require any fall-back strategies; and

TABLE 1 PAVEMENT PERFORMANCE RECORD, SECTION 22, STRATFORD DISTRICT

PAVEMENT PERFORMANCE RECORD

DISTRICT: 3
HIGHWAY :8

LHRS	OFFSET	LENGTH	DIRECTION	FACILITY	CLASS	LANES	AADT	TRUCK %
15870	2.90	2.4	B	A	C	4	32600	10.8

FROM: FREEPORT DR.

TO : FAIRWAY RD. INTERCHANGE

OVERALL PAVEMENT PERFORMANCE HISTORY

YEAR	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95
AGE	16					21	22	23										
PCR	80					68	68	68										
PCI	78					71	66		66									
RCR	7.0					7.0	5.9	7.0	5.7									
DMI	22.8					40.0	40.0	38.5	38.5									

DETAILED PAVEMENT PERFORMANCE HISTORY

		1978		1983		1984		1985		1986		19		19	
SEV.=SEVERITY, DEN.=DENSITY		SEV.	DEN.	SEV.	DEN.	SEV.	DEN.	SEV.	DEN.	SEV.	DEN.	SEV.	DEN.	SEV.	DEN.
C. AGG. LOSS & RAV.															
FLUSHING															
RIPPLING AND SHOVING															
WHEEL TRACK RUTTING		2	5	3	5	3	5	3	5						
DISTORTION				3	2	3	2	3	1						
LONGITUDINAL	SING. & MULT.	2	2	2	4	2	4	2	4						
(WHEEL TRACK)	ALLIGATOR														
CENTRE LINE	SING. & MULT.	2	1	3	2	3	2	3	2						
	ALLIGATOR														
PAVEMENT EDGE	SING. & MULT.			2	2	2	2	2	2						
	ALLIGATOR														
TRANSVERSE	HALF,FULL & MULT.	2	5	2	5	2	5	2	5						
	ALLIGATOR														
MEANDER AND MIDLANE				2	1	2	1	2	1						
RANDOM															

SEVERITY OF DISTRESS CODES

1. VERY SLIGHT
2. SLIGHT
3. MODERATE
4. SEVERE
5. VERY SEVERE

DENSITY (EXTENT) CODES

1. <10%
2. 10-20
3. 20-50
4. 50-80
5. 80-100

MAJOR MAINTENANCE HISTORY

YEAR>	1986		
ITEM	I	II	III
TYPE(CODE)			
EXTENT %	0	0	0
COST	.0	.0	.0

SHOULDER CONDITION FOR GRAVEL SHOULDERS

YEAR>	1986	
	SEV.	DEN.

DISTRESS COMMENTS:

OTHER COMMENTS :

EXTENT OF MAINTENANCE TREATMENT IN 1986 (CODE)

	PAVEMENT	SHOULDER
MANUAL PATCHING	0	0
MACHINE PATCHING	0	0
SPRAY PATCHING	0	
ROUT & SEAL CRACKS	0	0
CHIP SEAL	0	0

DATE: AUGUST 26, 1987

TABLE 2 ACTION PLAN FACT SHEET, SECTION 22, STRATFORD DISTRICT

ACTION PLAN FACT SHEET

DISTRICT: 3
HIGHWAY : 8

LHRS	OFFSET	LENGTH	DIRECTION	FACILITY	CLASS	LANES	AADT	TRUCK %
15070	2.90	2.4	B	A	C	4	32600	10.8

FROM: FREEPORT DR.

TO : FAIRWAY RD. INTERCHANGE

REHABILITATIONS

STRUCTURE	1	2	3	4	5
CONTRACT NO.					
SURFACE TYPE (CODE)					
TOTAL SURFACE THICKNESS, MM					
ADDED OVERLAY THICKNESS, MM					
MILL / REMOVE THICKNESS, MM					
BASE TYPE (CODE)					
BASE THICKNESS, MM					

SUBBASE TYPE (CODE) :
SUBBASE THICKNESS, MM :
SUBGRADE TYPE (CODE) : 2

STRUCTURAL COMMENTS:
SECTION LIMITS MAY BE CHANGED.

HISTORY OF PAVEMENT PERFORMANCE

```
     100 +
         |
         |
      90 +
         |
         |
      80 +
P        |                        78
         |
C     70 +                                         71
         |                                            66    66
I        |
      60 +
         |
         |
      50 +
         |
         |
      40 +--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+--+-
   YEAR>  66    68    70    72    74    76    78    80    82    84    86    88    90    92    94   <YEAR
```

URBAN SECTION, % : 0

PAVEMENT WIDTH,M : 14.6

SHOULDER TYPE : G

LOAD RESTRICTION :

YEAR OF LAST SKID TEST :

OTHER DEFICIENCIES :

PAVEMENT CONDITION:	YEAR	PCI	RCR	DMI
	1986	66	5.7	38.5

ESTIMATED CHANGE IN PCI AFTER 3 YEARS > 6 <

PREFERRED, HOLDING AND DEFERRED STRATEGIES

	PART	DESCRIPTION	CODE	PROG YEAR	EXTENT %	COST/KM ($1000)	PCI AFTER	LIFE YEARS	S
PREFERRED STRATEGY	I	90MM HOT MIX RESURFACING	HM90	89	100	140	95	14	
	II								
HOLDING STRATEGY	I	MACHINE PATCHING (MAINLY E.B.L.)-25%	MP	88	25	30	66	14	
	II	90MM HOT MIX RESURFACING	HM90	91	100	140	95		
DEFERRED STRATEGY	I	90MM HOT MIX RESURFACING, 35-40% PADDING	HM90PAD40	92	100	200	95	14	
	II								

STRATEGY COMMENTS:

PREPARED BY:
DATE: AUGUST 26, 1987

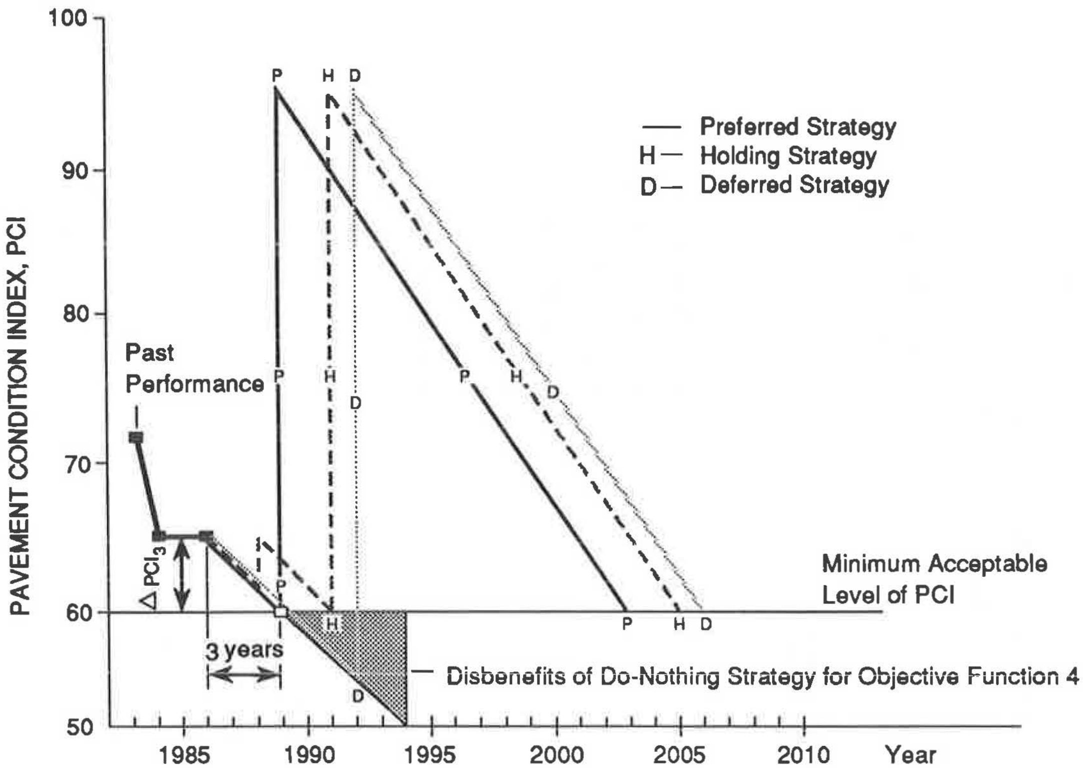

FIGURE 1 Performance curves for section 22, Stratford district.

- For some sections, no feasible holding strategies exist.

Each strategy must be fully described using the parameters listed in Table 2. These include type of treatment (Description in Table 2), its timing (Program Year), and its consequences (PCI After—PCI predicted immediately after the treatment is applied; and Life Years—predicted lifespan of the treatment in years). Predictions of ΔPCI_3, PCI After, and Life Years are based mainly on the engineering judgment of experienced regional staff who are familiar with local conditions. This situation occurs because of the unavailability or poor documentation of the many models required for predicting the performance of various pavement preservation treatments.

EVALUATION OF FUNDING REQUIREMENTS

The costs of all pavement preservation strategies can be summarized from individual action plans to obtain the total funding requirements for a district, region, or province; these costs can then be compared with available budgets. This process is illustrated using data for the Stratford district, which has 134 pavement management sections comprising about 1100 km of two-lane highways. Of the 134 sections, 75 required some pavement preservation funding during the 5-yr planning period. It must be emphasized that all data in this paper are provided for illustration only; any other interpretation of the data is incorrect.

Table 3 lists action plan strategies, in terms of their cost and timing, for the first five sections requiring funding in the Stratford district. This table also shows the results of linear programming, which will be discussed later. The funding requirements for all sections in the Stratford district are summarized in Table 4, which was obtained by retrieving data from the data base. The distribution of costs required to fund the preferred, holding, and deferred strategies is plotted in Figures 2a to 2c. It can be noted that the total cost of the preferred strategies ($26.73 million in Table 4) is not evenly distributed during the planning period. Also, $7.69 million worth of the preferred strategies do not have any fall-back strategies.

OPTIMIZATION ON NETWORK LEVEL

From the viewpoint of individual sections, the aggregated cost of all preferred strategies constitutes the most effective overall funding level. If this level exceeds the available yearly budget, either during one or more specific years of the planning period or during the entire planning period, not all preferred strategies can be implemented as recommended. Some must be replaced by holding, deferred, or do-nothing alternatives. Because there are usually multiple investment objectives and many sections involved, it is difficult to select the investment alternatives that would best use the available budget.

Any practical solution of this problem must take into account technical as well as societal investment objectives. For example, a technical investment objective may be to obtain the highest possible pavement performance, while a societal investment objective may be to achieve an equitable distribution of construction jobs in the province. An ideal situation

TABLE 3 DETAILED RESULTS OF LINEAR PROGRAMMING

Section Number	Strategy	Total Cost in \$1000's	Year	Decision Variable, Xij [a)]								
				Objective Function Used (yearly budget \$5,000,000)					Yearly Budget Constraint Using Objective Function 4 (in \$1000's)			
				1 Int.	1 Real	2	3	4	4000	3000	2000	1000
1	2	3	4	5	6	7	8	9	10	11	12	13
1	Preferred	367	89	1	1	1	1	1	1	1	1	0.2
	Holding											
	Deferred	464	92									0.8
	Do-Nothing											
2	Preferred	237	90				1	0.2	1	1	1	1
	Holding	392	88/92	1	1	1		0.8				
	Deferred	459	93									
	Do-Nothing											
3	Preferred	43	88				1					
	Holding											
	Deferred	211	93	1	1	1		1	1	1	1	1
	Do-Nothing											
4	Preferred	273	89				1		1			
	Holding	383	89/92	1	1	1		1				
	Deferred											
	Do-Nothing									1	1	1
5	Preferred	273	89				1					
	Holding	383	89/92	1	1	1		1	1			
	Deferred											
	Do-Nothing									1	1	1

a) Xij are defined by Equation 3. Zero values are not listed.
"1" means that the strategy will be done in its entirety."

TABLE 4 SUMMARY OF FUNDING REQUIREMENTS FOR STRATFORD DISTRICT

Strategy Type	Strategy Year 1987	1988	1989	1990	1991	1992	1993	1994	Total
All preferred strategies		5,458	10,767	5,914	1,330	3,147		114	26,730
Preferred strategies lacking fall-back strategies		2,820	2,190	526	830	1,210		114	7,690
Preferred strategies having fall-back strategies		2,638	8,577	5,387	500	1,937			19,039
All preferred strategies having holding strategies		1,533	7,359	4,861	500	1,937			16,190
All holding strategies	462	1,931	647	3,702	5,457	5,307	305	2,090	19,901
All preferred strategies having deferred strategies		2,638	6,153	4,333					13,124
All deferred strategies		57				6,131	11,908	1,783	19,879

NOTE: Figures are in thousands of dollars.

occurs when the selected investment alternatives fully satisfy both technical and societal objectives.

Linear Programming

Given a limited budget, the selection of investment strategies can be optimized using linear programming. A linear programming (LP) technique can maximize (or minimize) an aggregate consequence of individual actions (i.e., an objective function) given a set of limitations or constraints. In other words, preservation dollars can be allocated in a manner that yields the maximum benefit to the given pavement network (*5*). The LP problem was formulated as follows:

Maximize expression

$$\sum_{i=1}^{n} \sum_{j=1}^{m} X_{ij} B_{ij} \tag{1}$$

subject to

$$\sum_{i=1}^{n} X_{ij} \leq 1 \qquad j = 1 \text{ to } n \tag{2}$$

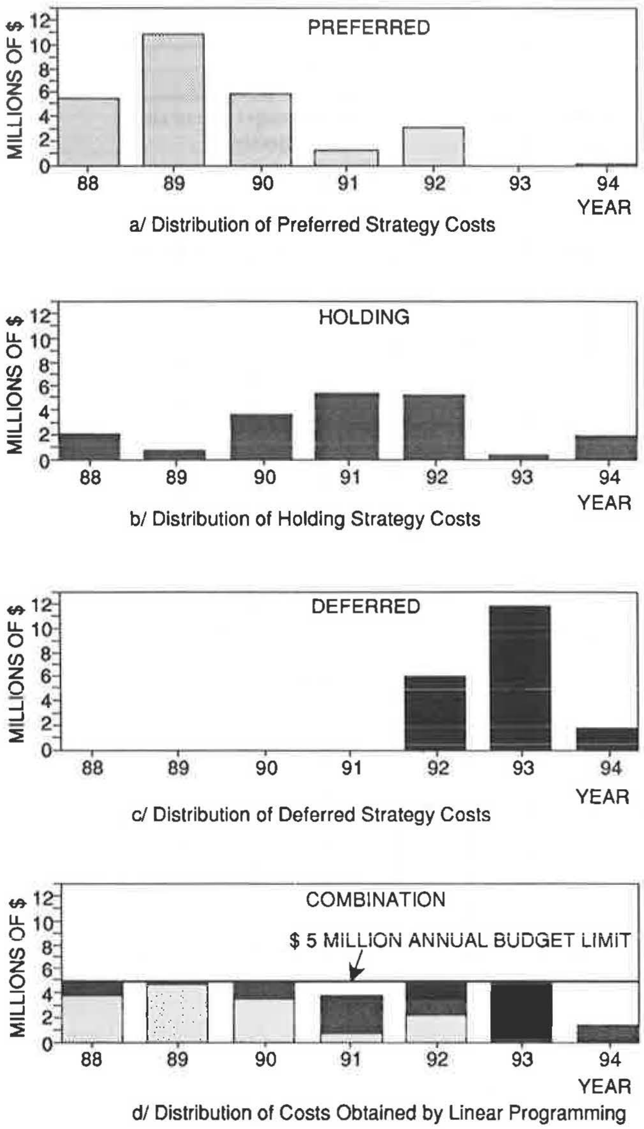

FIGURE 2 Cost composition for linear programming solution, objective function 4, Stratford district.

and

$$\sum_{i=1}^{n} \sum_{j=1}^{m} X_{ij} C_{ijt} \leq B_t$$

$$t = \text{each year within the planning period} \qquad (3)$$

where

B_{ij} = benefit associated with implementing strategy i for section j;
X_{ij} = a decision variable (for mixed integer programming model, X_{ij} = 1 if strategy i is selected and 0 otherwise);
C_{ijt} = cost of strategy i for section j in year t;
n = number of strategies for a given section (four maximum: preferred, holding, deferred, and do-nothing);
m = number of sections requiring funding during the 5-yr period (75 for the Stratford district); and
B_t = budget available in year t.

Equation 1 is called the objective function and represents the sum of benefits that can be obtained by implementing all selected strategies. Its goal is to maximize the value of the objective function subject to the constraints (Equations 2 and 3). Equation 2 ensures that only one strategy "unit" is selected per section, while Equation 3 makes sure available yearly budgets are not exceeded. The investment alternatives considered by LP are the strategies listed on the action plan fact sheets. They are practical, feasible alternatives with systematically evaluated costs and consequences.

To explore different ways of measuring pavement investment benefits and the consequences of different benefits on the LP solution, the benefits were measured in four different ways and the resulting LP solutions were obtained and compared. The following objective functions were evaluated using nonmonetary benefits.

Objective Function 1

The benefits used to calculate the value of objective function 1 (B_1) were expressed as the time in years at which pavement performance will be above the acceptable level (y), multiplied by the section length (L):

$$B_1 = \sum_{i=1}^{n} \sum_{j=1}^{m} y_{ij} L_j \qquad (4)$$

For example, considering data given in Table 2 and Figure 1, the benefit of the preferred strategy is the product of 14 yr (the strategy is expected to last from 1989 to 2003) and the length of 2.4 km. The benefit of the do-nothing alternative for the same section was calculated as the ratio of the section's ΔPCI_3 (six PCI units in Table 2) and the average ΔPCI_3 for all sections in the Stratford district (6.3), multiplied by 5 (years). Given a limited budget, this formulation maximizes the number of sections with an acceptable level of pavement performance.

Objective Function 2

The benefits used to calculate the value of objective function 2 (B_2) were similar to those of objective function 1 but were also factored to include the influence of traffic:

$$B_2 = \sum_{i=1}^{n} \sum_{j=1}^{m} y_{ij} L_j T_j \qquad (5)$$

where

T_j = log (AADT of cars + 3 AADT of trucks), and
AADT = annual average daily traffic.

This formulation increases benefits for sections with high traffic volumes, particularly truck volumes. Truck volumes were multiplied by three because user savings attributed to the improved pavement performance have been estimated to be roughly three times greater for trucks than for cars (*6*).

Objective Function 3

The benefits used for objective function 3 were the same as those of objective function 2 and were expressed as a benefit/

TABLE 5 EVALUATION OF OBJECTIVE FUNCTIONS FOR YEARLY BUDGET OF $5 MILLION

	Type of Objective Function				
	Integer Solution	Real Solution			
Evaluation Parameter	1	1	2	3	4
Total benefits	7,493.6[a]	7,520.2[a]	N/A[b]	N/A	N/A
Strategy					
Preferred					
Cost[c]	12,357.0	13,324.8	13,470.3	16,015.0	15,134.2
Number	39	38	38	51	45
Holding					
Cost	15,013.0	14,322.3	14,111.2	10,581.6	8,784.9
Number	30	31	32	21	20
Deferred					
Cost	2,633.0	2,356.0	2,329.2	1,071.4	6,110.0
Number	6	6	5	2	10
Do Nothing					
Number	0	0	0	1	0
Total					
Cost	30,003.0	30,003.1	29,910.7	27,668.0	30,029.1
Number	75	75	75	75	75
Number of split strategy solutions	0	5	5	4	4

[a]Nonmonetary benefits.
[b]Not applicable for comparison.
[c]Costs are in thousands of dollars.

cost ratio. The costs were the estimated strategy costs given on the action plan fact sheets. No penalty was included for do-nothing alternatives.

Objective Function 4

Objective Function 4 attempted to maximize user benefits (B_4) by using the area underneath the performance curve (PCI–time curves in Figure 1) rather than the length of the expected lifespan as in objective functions 1, 2, and 3:

$$B_4 = \sum_{i=1}^{n} \sum_{j=1}^{m} A_{ij} L_j T_j \tag{6}$$

where A_{ij} equals the area under the performance curve of strategy i for section j.

For simplicity, all pavement performance curves used to calculate B_4 were assumed to be straight lines, even though the performance curves for individual pavement sections in Ontario can also show an increasing or decreasing rate of pavement deterioration with time (*7*). The linear rate is a compromise; there is some evidence from the AASHO Road Test that the rate of pavement deterioration due to traffic loadings is linear when results are plotted against a roughness-dominated measure such as the PCI (*8*). It should also be noted that performance curves established for groups of similar pavements in Ontario are approximately linear (*9*).

The objective functions evaluated in this paper were based on technical considerations alone. While the LP model can optimize the value of only one objective function, it is possible to construct a single objective function that incorporates both technical and societal objectives. For example, the technically calculated benefits, such as objective function 4, can be adjusted according to their geographical area. However, the funds that can be saved by using optimization instead of ranking are substantial and should be considered seriously (*10*).

The linear programming solution was obtained by LP83/MIP83 microcomputer software (*11*), which can provide both real number linear programming (RNLP) and mixed integer linear programming (MILP) solutions. The MILP solution ensures that X_{ij} of Equation 1 is either 1 or 0, while the RNLP solution allows X_{ij} to be real numbers, which may result in "split" strategy solutions. For example, a split strategy solution may recommend implementing 20 percent of the preferred strategy and 80 percent of the do-nothing strategy (see section 1, column 13, in Table 3).

The differences between the RNLP and MILP solutions were evaluated for the 75 sections in the Stratford district using objective function 1. The computation time required for the MILP solution using an IBM-AT microcomputer was about 24 hr, while the time for the RNLP solution was only 5 min. Despite the large difference in the computation time, the solutions were quite similar in terms of total benefits (7493.6 versus 7520.2, as shown in Table 5) and in terms of the strategies selected for individual pavement sections. The number of split strategy solutions allowed by RNLP (five) represented only 7 percent of the 75 sections. Furthermore, it can be shown that the number of split strategy solutions cannot exceed the number of constraints defined by Equation 3 (that is, the number of years in the planning period). After considering the computation time savings and the intended use of the LP solution as a management decision support tool, RNLP was used for all subsequent analyses.

Sensitivity Analysis of the Objective Function

The example results of a sensitivity analysis of the four objective functions using data from the Stratford district are listed in Table 3, columns 5 to 9, and in Table 5. The analysis assumed a yearly budget of $5 million for 7 consecutive years. The total yearly cost of the preferred strategies during this period reached a maximum of $10.8 million in 1989 (see Table 4 and Figure 2a).

The linear programming solution keeps the total yearly cost

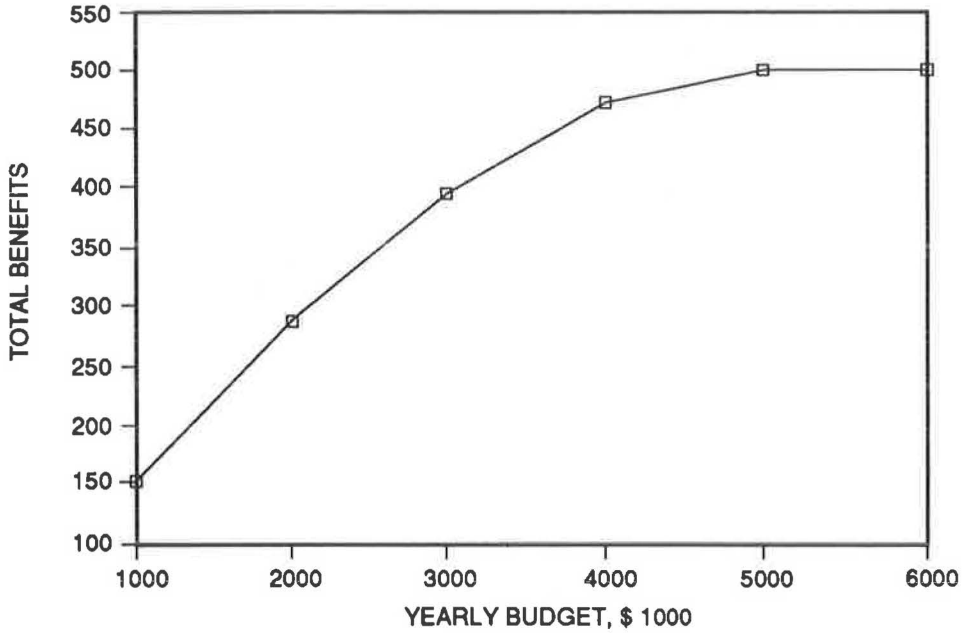

FIGURE 3 Influence of budget constraints on investment benefits.

of the selected strategies below the $5 million constraint. This was done by selecting the fall-back strategies (holding, deferred, and do-nothing strategies) instead of the preferred strategies for some sections, while maximizing the total benefit. The composition of the resulting LP solutions for objective functions 1 through 4 is given in Table 5. The composition for objective function 4 is also illustrated in Figure 2d. No do-nothing alternatives were selected for the relatively high yearly budget of $5 million.

The results of the sensitivity analyses given in Tables 3 and 5 indicate that the linear programming solution is sensitive to the formulation of the objective function. For example, the LP solution for objective function 2 recommends the implementation of holding strategies on 32 pavement sections, while the solution for objective function 4 recommends holding strategies for only 20 sections. Objective function 2 maximizes the total length of pavement sections above the minimum acceptable PCI level. Because holding strategies usually include patching, and patching can keep pavement performance just above the minimum acceptable level at relatively low cost, many holding strategies were selected for objective function 2. Objective function 4 takes into account that the benefit to road users varies with the level of PCI above the minimum acceptable level. Therefore, holding strategies become less attractive because of patching and low pavement serviceability. These results underline the need for careful and clear identification of pavement investment objectives, in other words, an unequivocal declaration of what the investments are supposed to achieve.

Sensitivity Analysis of the Yearly Budget Constraints

A sensitivity analysis of the effect of changing the yearly budget constraints was done by reducing the yearly budget from $5 million to $1 million in four equal steps. This analysis was conducted using data from the Stratford district for objective function 4 only. Its purpose was to determine the effect of budget reduction on strategy selection and on the value of the objective function.

Fall-back strategies were chosen during initial budget reductions, which allowed for the deferral of expenditures to later years. With further reductions, the budgets did not permit expenditures on an increasing number of sections, which resulted in an increasing number of do-nothing recommendations. While there were no do-nothing strategies for the $5 million budget, there were 7 for the $4 million budget and 55 for the $1 million budget.

The do-nothing strategies, like other alternatives, were selected to maximize total investment benefits. These strategies were characterized by negative benefits (disbenefits) since they allowed the PCI to drop below the minimum acceptable level (see Figure 1). The LP solution selected do-nothing strategies for those sections where the drop below the minimum acceptable level created the smallest number of disbenefits.

With the reduction in yearly budgets, the value of the corresponding objective function summarizing the total investment benefits was also reduced. Figure 3 shows that this relationship was not linear. As the budget was reduced, the LP solution first eliminated those strategies that provided the least amount of benefits.

FUNDING DECISIONS AND THEIR CONSEQUENCES

The action plan strategies clearly identify the most cost-effective funding requirements for individual pavement sections as well as the consequences of not providing the required funding on time. These project-level strategies are determined by individuals who are responsible for pavement design and familiar with local conditions.

On the network level, the funding requirements for indi-

TABLE 6 EXAMPLE OF AN OPTIMIZED PAVEMENT PRESERVATION PLAN

Section Number	Section Identification					Recommended Strategy			
	Hwy	LHRS	Offset (km)	Length	Dir.	Type	Description	Year	Cost x1000
1	4	12880	5.6	10.8	B	Preferred	Mill 35 mm," Padding 10 % & H.M. 50 mm (R)	1989	$367
2	4	12890	1.9	8.2	B	Preferred	Mill 35 mm & H.M. 50 mm (R)	1990	$237
3	4	12900	1.9	7.3	B	Deferred	Mill 25 mm, Padding 5 % & H.M. 50 mm (R)	1993	$211
4	6	13605	0	3.7	N	Preferred	Padding & H.M. 50+40 mm	1989	$273
5	6	13605	0	3.7	S	Holding	1. Mill 40 mm & H.M. 40 mm 2 Padding & H.M. 80 mm	1989 1992	$110 $273

*** NOTE:** (R) means Recycled Hot Mix

vidual sections can be summarized and prioritized to provide authoritative overall funding recommendations. For a given budget, the selection of specific sections (and treatments) to be funded can be optimized by linear programming. Consequences of any restrictive funding decisions can be readily identified in terms of section-specific consequences.

Table 6 lists recommended strategies for the first five sections of the Stratford district assuming a $4 million yearly budget and objective function 4. This is an example of an optimized pavement preservation plan that can be used as a basis for considering prior commitments and other constraints during the formulation of an actual pavement preservation program.

CONCLUSIONS AND RECOMMENDATIONS

• The prioritization framework described in this paper reflects the distributive nature of the decision-making process for pavement maintenance and rehabilitation planning and programming in the Ontario Ministry of Transportation. The concerns and inputs of the experienced field staff in the Ministry's five regions and 18 districts are accommodated within the framework. Also, funding and planning decisions made on the network level can be easily translated into project-specific actions and consequences.

• Action plans prepared for individual pavement sections provide sufficient information for prioritizing and optimizing pavement preservation strategies on both project and network levels. They enable an estimation of the overall health of the network at the end of the planning period.

• The aggregate cost of all preferred strategies, obtained for individual pavement sections by life-cycle economic analysis, constitutes the most effective overall funding level.

• Linear programming is a very useful management tool that can help allocate pavement investments to yield the maximum benefit for the entire pavement network.

• The way in which pavement investment benefits are measured must be carefully considered. Ideally, the benefits should reflect both technical and societal investment objectives. Objective function 4, which considers road user benefits, is recommended for addressing technical investment objectives.

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