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Contents

Foreword

Many of the 24 peer-reviewed papers in this volume were presented at two sessions sponsored by TRB Committee A2B01, Committee on Pavement Management Systems, during the 1994 Annual Meeting of the Transportation Research Board. Others were submitted and presented at previous years' meetings.

Collura et al. investigate the involvement of metropolitan planning organizations (MPOs) in pavement management activities and develop a framework to better formulate their roles. Andres and Collura discuss priority ranking of pavement management activities at the network level, focusing on problems peculiar to local communities. Gáspár presents the first Hungarian pavement management system based on Markov transition probability matrixes. Fwa et al. describe an application of a relatively new optimization technique for pavement management programming known as genetic algorithms. Ewing compares residential street design guidelines used in Britain and Australia with those generally in use in the United States. Ravirala and Grivas present a tool for developing an annual pavement program using nonpreemptive goal programming methodology. Sikow et al. study the effects of reduced highway investment on the pavements in the arctic climate of Lapland, Finland. Chen et al. describe the project-level pavement design and maintenance subsystems of the Urban Roadway Management System (URMS). Paterson proposes a "cracking indicator," which would provide a universal worldwide measure of cracking in pavements.

DeCabooter et al. describe the development of Wisconsin's geographical information system-based pavement management system since its inception in 1987, with emphasis on the decision support system. Rose and Bennett report on the economic benefits from using the accelerated loading facility for pavement research in Australia. Wang et al. discuss the revisions and improvements to the award-winning Arizona Network Optimization System using a newly developed linear optimizer. Khedr and El Dimeery present a pavement management system for provinces in developing countries and describe a successful implementation in Fayoum, Egypt. Hutchinson et al. compare the results of two pavement deterioration models with a view toward optimal strategy for Ontario pavements. Hall et al. conducted three analyses of the Interstate pavement network in Illinois to provide useful information toward forecasting pavement rehabilitation needs. George et al. present an overview of the development of the Mississippi Pavement Management Information System and the major products therefrom. Männistö and Tapio describe the coupling of the pavement and bridge management systems into a new Finnish National Road Administration Infrastructure Management System. Van Dam and Thurston discuss a multiattribute utility analysis method for comparing pavement design alternatives. Queiroz et al. discuss the relationship between national economic development and prosperity and paved road infrastructure. Attoh-Okine and Martinelli apply belief functions, otherwise known as the Dempster-Shafer theory of evidence, to pavement management system decision making. Dossey and Hudson use 20 years of historical condition survey data to develop distress prediction models for continuously reinforced concrete pavements in Texas. Shahin presents a procedure that is part of the Micro PAVER system developed by the U.S. Army Corps of Engineers to analyze the consequences of various budget scenarios on pavement condition and the backlog of maintenance and rehabilitation. Saeed et al. build on the principles of the Strategic Highway Research Program's Long-Term Pavement Performance program to develop a procedure for a state-level long-term pavement performance program for modeling of rigid pavement performance. Mijušković et al. use a controlled nonhomogeneous Markov process to describe road network deterioration in developing rules to define a pavement maintenance strategy.

As a related activity during 1994 TRB sponsored the Third International Conference on Managing Pavements, May 22–26, 1994, in San Antonio, Texas. Proceedings of that conference are available from TRB.

Roles of Metropolitan Planning Organizations in Pavement Management

JOHN COLLURA, EMMANUEL OFORI-DARKO, AND FREDERICK P. ORLOSKI

In recognition of the need for local pavement management and the issues surrounding the possible involvement of the metropolitan planning organization (MPO), the ways in which MPOs have participated in local pavement management activities were studied and a framework that could be used as a guide to identifying the appropriate role and set of responsibilities of MPOs in the conduct of local pavement management studies and projects was formulated. Case studies of the pavement management experiences of four regional planning agencies (RPAs) in Massachusetts are reviewed. These RPAs, which provide staff support to their MPOs, have participated in local pavement management studies and have attempted to integrate such efforts into the regional transportation planning process. The framework consists of eight elements covering the major issues and activities pertaining to pavement management and is intended to be used as a guide for MPOs in the conduct of local pavement management activities. In addition, the flexibility of the framework facilitates the incorporation of the results of pavement management studies into the urban transportation planning process and specifically into the transportation improvement program. A number of conclusions pertaining to the variety of roles MPOs could play in local pavement management are presented, and the need for MPOs to seek assistance from individuals who are not members of the MPO staff is described.

The promotion, development, and implementation of a proper pavement management system has been of interest to both metropolitan planning organizations (MPOs) and local governments (*1*). Typically the road network in any planning region may be identified as a component of a system under various jurisdictional controls, namely, federal, state, county, city, and town. As a result, requirements and funding responsibilities depend on who has jurisdictional control and several other factors, including traffic volumes, environmental conditions, and the original pavement structure. Therefore, it is important for all levels of government within a specific regional area to establish the proper cooperative effort and the required communication channels to maximize fully the benefits of pavement management.

The benefits to be derived from the use of a pavement management system (PMS) accrue to local and regional agencies. For example, local agencies would be able to maintain a data base for assessing the condition of the road networks within their communities and also to identify the competing infrastructure needs. In addition, the PMS would provide the local agency with an objective tool that would aid decision makers in scheduling roadway investments and maintenance actions, and for those local communities in which a PMS is in place, an added benefit would be the formulation of more cost-effective alternatives at the project level. Benefits realized by the MPOs include the ability to develop a comprehensive data base for their respective regions that would help address the regional transportation needs and also perhaps help state transportation agencies improve the state PMS data base. The PMS could be used to help in selection of projects for the transportation improvement program (TIP). At any point the regional pavement infrastructure needs could be generated from the PMS data base and the potential funding requirements could be determined; MPOs may then be in the position of helping local communities develop and assess alternative forms of funding for locally maintained roads. Finally, MPOs may also assist in the coordination of resources between local communities with similar needs. Given the nature of this cooperative effort, it is reasonable for the MPO to be directly involved in local pavement management. Further discussions of local and regional benefits associated with the use of a PMS have been presented by others (*2,3*).

This paper presents a framework to assist MPOs in the determination of their proper roles and responsibilities in pavement management. The framework essentially consists of elements (or activities) in which MPO involvement may be limited or extensive, with MPO involvement being determined by factors such as expertise of MPO staff, local roadway conditions, and available funding for improvements.

PAVEMENT MANAGEMENT SYSTEMS

Billions of dollars are invested in roadway infrastructure annually to ensure the mobility of people and goods. As a component of these investments the restoration of roadway infrastructure requires the continuous flow of resources to maintain and rehabilitate highway pavements for the purpose of protecting the required surface conditions and structural capabilities. In view of the problems of inflation, deteriorating road conditions, increasing traffic loading, and reductions in funding, the maintenance and rehabilitation process presents a complex management challenge (*4*). This task involves studying pavement networks and conditions, deciding on maintenance strategies, setting priorities, and making investment decisions, which together constitute the pavement management process.

In the past 20 years the concept of pavement management has become an active process at federal, state or provincial, regional, and local levels (*3,5,6*). This concept has become increasingly important in the highway community in the past 10 years (*7*). The pavement management concept continues to expand and is considered for use at all levels of government at varying levels of detail and sophistication. As described elsewhere (*8*) PMSs are primarily a set of analytical tools or methods that assist decision makers in finding optimum strategies for maintaining pavements in a serviceable condition over a given period of time. For some systems implementation is labor-intensive and time-consuming, whereas for others it is

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simple and the PMSs are easy to use. It has also been determined that data requirements can be extensive, and computer facilities may be required (*2*).

CASE STUDIES

This section summarizes case studies of the pavement management activities of four regional planning agencies (RPAs) in Massachusetts, all of which represent the staffs of the MPOs. The geographic locations of the four MPOs are shown in Figure 1. The regional areas served by these MPOs include rural and urban settings.

Tables 1 to 3 provide a comparison of various characteristics of the pavement management (PM) processes in each of the four RPAs, which include Pioneer Valley Planning Commission (PVPC), Old Colony Planning Council (OCPC), Southeastern Regional Planning and Economic Development District (SRPEDD), and the Metropolitan Area Planning Council (MAPC). The major findings of the case studies are as follows:

1. The four RPAs—PVPC, OCPC, SRPEDD, and MAPC—participated in local pavement management primarily to develop and implement a continuous and systematic method of optimizing scarce public funds available for local road maintenance, rehabilitation, and reconstruction.
2. RPA involvement in local pavement management in Massachusetts has been encouraged because of the availability of federal and state funding in the comprehensive, continuing, and cooperative (3C) transportation planning program. This involvement began in the early to middle 1980s.
3. Before the conduct of the present research the Massachusetts Department of Public Works (now the state Department of Highways) indicated that local PM activities may be proposed within the annual work program of each RPA. Therefore, if an RPA considered PM to be a priority it would include PM within the proposed work plan. At the time of the present research only the four RPAs listed had included major PM activities in their work plans and had conducted substantive PM work. In the past year several additional RPAs have included PM in their work programs. Given that PM activities are considered for inclusion in annual work plans, PM activities compete with other work plan activities such as transit, bicycle, air quality, and other work plan projects.
4. In general, the roles and levels of involvement of RPAs have varied from promotion, education, software development, and training to participation in the conduct of the individual activities within the respective PMS. The level of involvement in the pavement management programs consisted of policy planning and network-level analysis in a limited number of towns in each region.
5. Two potential PMS development strategies are observed. Whereas PVPC and MAPC developed their own software without major outside assistance, OCPC and SRPEDD used available software packages and modified them to satisfy the needs of the local communities. OCPC used software available from the San Francisco Bay's Metropolitan Transportation Commission, and SRPEDD selected the New Hampshire Rural Technical Assistance Program's software.
6. Local community participation increased steadily as a result of the promotional aspects currently present in some RPA areas. It should be emphasized that very little local pavement management activity has occurred in these regional areas apart from those efforts initiated by the respective RPAs.
7. The focus of these regional and local efforts has been on roads under local jurisdiction. For this and other reasons, integration of the various PMS results into the 3C transportation planning process has been absent. In addition, the absence of a defined and systematic approach makes it difficult for MPOs to integrate such pavement management efforts into the annual TIPs.
8. Almost no follow-up of local pavement management studies has occurred, and no steps have yet been taken to incorporate pavement management study results into the TIP. However, it should be noted that actions are being initiated by the RPAs to address the issues of follow-up and TIP programming.
9. Table 4 provides a summary of the highway programs that the four RPAs coordinate and presents the programs within which pavement management results might be programmed and integrated into the transportation planning process. The federal aid programs are standard programs for which all MPOs in the United States are eligible. In addition, there are non-federal aid programs unique to Massachusetts, including the Chapter 90 Program and the Public Works and Economic Development program.

FRAMEWORK

As presented in Figure 2 the framework consists of eight major elements. An element is an activity or group of activities with specific purposes. This framework is designed to provide flexibility for different pavement management models, systems, and procedures to be used in the major elements listed. Table 5 presents examples of such activities within each of the eight major elements.

Five important roles identified for the MPO in the conduct of local pavement management may be described as follows: an initiator, in which the MPO might give a presentation to a local public works committee of the costs and benefits of local pavement management, which would lead to the conduct of a network-level study carried out by the city or town; a facilitator, in which the MPO makes it easier for the local government to perform an activity, for example, the MPO might provide computer expertise by processing the distress data collected by local officials; a coordinator, a role in which the MPO brings local communities together to a joint activity such as bulk purchasing of materials or services; a trainer, in which the MPO is involved in providing instruction to local personnel, perhaps related to the conduct of a distress survey; and a doer, in which the MPO executes or performs a task such as actually carrying out the survey.

A detailed description of the elements in the framework follows. In addition, the extent of an MPO's involvement and the various roles are discussed.

Education and Promotion

The education and promotion element covers the promotional and educational aspects of the PM process. It initiates the PM process and constitutes an important aspect that fosters different levels of local community involvement. Results from the four case studies indicate greater participation in regions where MPOs embarked on educational and promotional activities.

MPOs involved in this element might be termed *initiators.* At the regional level MPOs may have a higher degree of involvement in this activity aimed at creating PM awareness within local communities and convincing communities to participate in the ongoing

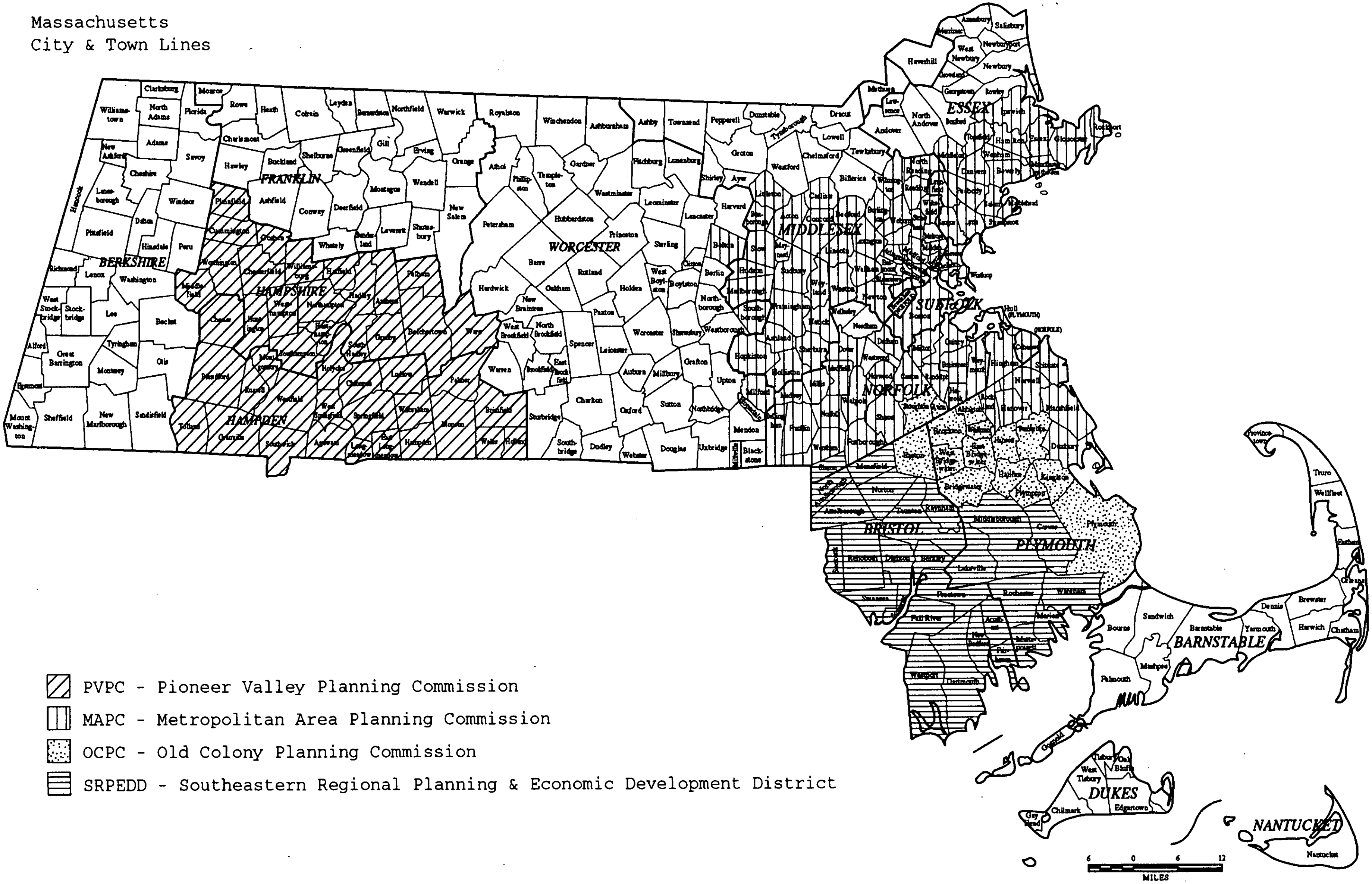


FIGURE 1 Geographic locations of case study areas.

TABLE 1 General Information on RPAs

AGENCY	URBANIZED AREA	**AREA POPULATION	CITY/* TOWN ROAD MILEAGE	DATE OF PM INVOLVEMENT
PVPC	Springfield-Chicopee-Holyoke urbanized area (43 cities and towns)	602,878	3,252.50	1984
OCPC	Brockton Urbanized area (15 cities and towns)	293,745	1,634.48	1987
SRPEDD	Fall River-New Bedford-Taunton urbanized area (28 cities and towns)	565,514	2,589.00	1984
MAPC	Boston metropolitan area (101 cities and towns)	2,922,934	9,520.00	1984

*Source: Massachusetts Dept. of Public Works, Road Inventory Program.
**Source: U.S. Census Bureau, 1990 Population.

process. This effort should present PM concepts to town and city managers, elected officials, selectmen, highway superintendents, and public works officials. Activities that are part of this element might include public presentations and the preparation of brochures on the concepts, contents, benefits, and extent of commitment associated with PM. Emphasis is often placed on the benefits achieved with minimal expenditure of resources. Expected results would be the adoption by communities of a PM program and personnel and funding commitments toward its development and implementation. MPOs may only have to deal with communities in their planning regions with a work activity already defined in the 3C planning work program. It should be noted that some RTAP centers and private consultants have already participated in promoting PM and in offering PM training workshops to local officials.

Policy Planning

The Policy planning element addresses issues pertaining to the formulation of local policies, goals, and objectives. When necessary and appropriate local officials such as the town highway superintendents may solicit assistance from the MPO. Key issues within this activity include assessing current and past maintenance policies, defining managerial and physical objectives, and investigating funding requirements to maintain an acceptable road network condition and to meet future needs. The local commitments necessary to ensure a continuous PM process are also identified. The extent of MPO involvement in this element may not be extremely extensive, and the MPO role might be that of facilitator.

Network-Level Analysis or Systems Planning

Of those PM efforts in which MPOs have been involved, the network-level analysis or systems planning element has been the one in which MPOs have been very active. A review of the literature indicated that the majority of the PMSs developed and implemented in local communities have been geared toward addressing network-level needs. A number of activities ranging from network section definition, data selection and collection, and manual or computerized data processing and analysis to priority setting, budgeting, the generation of reports, and implementation are contained within this element.

The network analysis might be executed by using one of the several computerized network-level PMSs available. The roles and responsibilities of the MPO and local agency within this element largely depend on the particular PMS to be adopted and the development and implementation goals.

Depending on the level of expertise within the respective local agency and MPO, local personnel might require training on the

TABLE 2 RPA PM Level of Operation and PMS Development

AGENCY	PM LEVEL OF OPERATION	PMS DEVELOPMENT	PMS CHARACTERISTICS			
			Data Collection		Data Analysis	
			Manual	Computerized	Manual	Computerized
PVPC	Network Level	In-house	X	X	X	X
OCPC	Network Level	Adopted MTC system		X		X
SRPEDD	Policy Planning	MAPC's PM forecasting model				X
	Network Level	Adopted NH RTAP RSMS	X	X		X
MAPC	Policy Planning	In-house				X
	Network Level	In-house	X		X	

PMS. This basic training may cover data collection and processing and computer hardware and software techniques. For the PMS adopted and implemented by local communities with MPO assistance, this training role may be carried out by the MPO, the state RTAP center, or a private consultant. In some communities MPOs may be involved in data collection and analysis at the initial stages of the pavement management program. When computerized techniques are used in the PMS, MPOs may provide equipment or coordinate sharing of hardware and software for those communities that cannot afford the initial capital costs. It should be noted that the extent of MPO involvement in this element may range from minimal to extensive, and roles may be as a facilitator, trainer, or doer.

Project-Level Analysis

The project-level analysis element usually involves more technical and engineering activities, including pavement design. Results of case studies show the absence of MPO involvement in this aspect of the PM program. Project-level analysis is usually well executed through professional engineering organizations. However, MPOs may be useful in coordinating the hiring or joint hiring of consulting services and the preparation of contract documents for those local communities that may need project-level assistance. This will enable communities with limited resources to combine such resources and also to control and monitor such consulting activities. MPOs may therefore be coordinators within this element, with a relatively minimal extent of involvement.

Programming

The programming element is primarily aimed at directing and integrating the final products of both the network- and project-level elements into the respective regionwide transportation planning program.

Final priority ranked projects for pavement rehabilitation, reconstruction, and maintenance selected through the network and project analyses may be integrated into the 3C planning process through their inclusion in the regionwide TIPs.

Pavement rehabilitation and reconstruction projects resulting from network- and project-level analyses on roads under the federal aid system may be listed in the regionwide multiyear element in the TIP if funding is expected or in the annual element if funding commitments have been made. Non-federal aid projects may also be listed in the respective sections of the TIPs to provide a comprehensive documentation of the various regional transportation needs and improvements for both capacity deficiency and surface condition.

An important issue in the programming element is the criterion or set of criteria to be used. Primarily, these criteria may depend on both funding and the pavement condition assessment. In most PMSs an index or set of indexes is established as a measure of a pavement segment's condition or the condition of individual sections within the network. This index or set of indexes is usually used as a basis for recommending treatments. Examples of such pavement condition measures include a pavement condition index and a pavement serviceability index. Other indexes include the ride comfort index,

TABLE 3 Local Community Participation and RPA Roles

AGENCY	LOCAL COMMUNITY PARTICIPANTS	DATE OF PM STUDIES	RPA ROLES AND RESPONSIBILITIES: Initiated PM Study	Data Collection	Data Analysis	Funding for PM Study	Report Preparation	Training	Report Presentations
PVPC	Westhampton Middlefield Williamsburg Goshen Chesterfield Worthington Pelham Agawam	1988 1988 1988 1989 1989 1989 1990 1990	X	X	X	X	X	X	
OCPC	Kingston	1987		X	X	X	X		
SRPEDD	Somerset Plainville Somerset Rochester Seekonk	1984 1986 1988 1990 1990		X	X	X	X	X	X
MAPC	Wenham Medfield	1986 1986		X	X		X	X	

the structural adequacy index, the surface distress index, and a composite pavement quality index (*9*).

The key inputs into the programming element may include pavement condition, which is addressed through the PMS; safety, which is addressed through the highway safety and improvement program; and capacity deficiency, which is addressed through the transportation system management. A benefit to be derived from this composite approach would be an improvement in the use of the scarce funds available for preserving the road's infrastructure. This approach would enable safety improvements, road widening, and pavement rehabilitation or reconstruction to be coordinated and perhaps combined and programmed together.

An example of an approach similar to that used previously for ranking deficient roads was developed and recommended for use by members of the Southeast Michigan Council on Regional Development in 1984. This approach, outlined in Figure 3, is composed of six basic steps and uses capacity and pavement condition as measures in identifying deficient roads. Capacity is defined in terms of both present and future levels of congestion, whereas pavement condition is defined by both surface and base deterioration. The primary aim of the ranking methodology is to enable the agencies involved to develop a realistic listing of deficient corridors to be programmed for project implementation. The congested roadways are grouped into one of two categories in terms of length (i.e., less than or greater than or equal to 2 mi). The congested road sections greater than or equal to 2 mi are further classified into high, medium, and low congestion. Each congested road less than 2 mi long together with the medium- and low-congested roadways, are referred to the county-level TSM committees for analysis, whereas roadways classified under high congestion are grouped into corridors for improvement under the region's transportation plan (*10*).

To carry out this element there is a need for effective communication between the MPO and local governments. Because this element is mainly a planning-related activity, the extent of MPO involvement will likely be high and, hence, the MPO role will be that of a doer.

Construction

This is the element in which the programmed projects are constructed. This element results from the projects selected during the network- or project-level elements. Included in construction are contract control, contract scheduling, construction inspection, and the main construction activities.

MPOs may have very little role in this element. However, depending on the existing MPO-community relationship, MPOs may help communities schedule contract activities and project construction. MPOs may assist in coordination of joint construction programming for communities within their regions undertaking similar construction projects and may encourage joint inspection control.

TABLE 4 Highway Programs Coordinated by RPAs

PROGRAM \ AGENCY	PVPC	OCPC	SRPEDD	MAPC
Federal-aid programs (FA)				
Interstate (const.)				X
Interstate (4R)	X	X	X	X
Interstate (transfer)			X	X
Urban Systems	X	X	X	X
Consolidated primary	X	X	X	X
Bridge R&R	X	X	X	X
Rural secondary	X	X	X	
Hazard Elimination	X	X	X	X
Rail / highway hazard crossing		X	X	
Other (VSPD)	X			X
State funded highway programs				
Non federal-aid (NFA)	X	X		X
PWED		X		X

PWED - Public Works and Economic Development
VSPD - Various Special Project Developments

The activities within the construction element include a maintenance element, which is needed to keep the existing and rehabilitated pavements in their acceptable conditions, managing the various maintenance activities, and maintaining an accurate maintenance record. Potential roles might include coordinating equipment sharing in communities undertaking similar maintenance jobs and assisting such communities in scheduling similar jobs.

Follow-Up

The follow-up element deals with monitoring local pavement management efforts to ensure that such efforts are being carried out with continuity, where appropriate. It also concerns issues regarding the use of new technology to improve the pavement management program when necessary and updating initial budget and pavement planning data inputs. Results of project implementation through the construction element would be used to update highway historical records.

The follow-up element might also include the dissemination of information about pavement management activities, perhaps through an MPO newsletter or the RTAP centers.

The extent of MPO involvement might be high, depending on the local community participation in the pavement management process and the size of the pavement management data base.

Research

The research element may include evaluation of the conduct and performance of the pavement management process within each respective region and identifying possible changes, if necessary. Efforts should be initiated to develop performance models, to evaluate the cost-effectiveness of maintenance and rehabilitation strategies, and to develop improvements to the overall local pavement management process. The extent of MPO involvement might be more extensive, depending on the role and commitment of local communities and the participation of other agencies, for example, RTAP centers and local communities.

Summary

Table 6 presents a summary of the possible roles and extent of MPO involvement in each element.

The possible extent of involvement ranges from low (2 to 4 person-days a month) to medium (4 to 8 person-days a month) to high (10 to 15 person-days a month). The results of the case studies and the literature review were used as a basis for developing these estimates of possible involvement.

SUMMARY AND CONCLUSIONS

Interest in the pavement management process has increased substantially at the local level in an effort to improve the overall condition of local roads with limited resources. However, the structures and institutional characteristics of local highway agencies bring about a complex set of managerial issues in attempting to organize a broader pavement management program that considers regional goals and objectives. These difficulties are due in part to differences in local road maintenance policies, resources, practices, and priorities.

Considering the nature of the regional road network and the benefits to be gained from a PMS, MPOs should play a greater role in the initiation, development, and implementation of local pavement management. As discussed previously, the institution of pavement management programs in local communities and the participation of MPOs in local pavement management would result in a number of benefits ranging from the judicious use of limited local resources to the improvement in both the local and regional road network conditions. The PMS data base at the local and regional levels would enhance and encourage efficient decision making, which would facilitate the development of appropriate road maintenance and

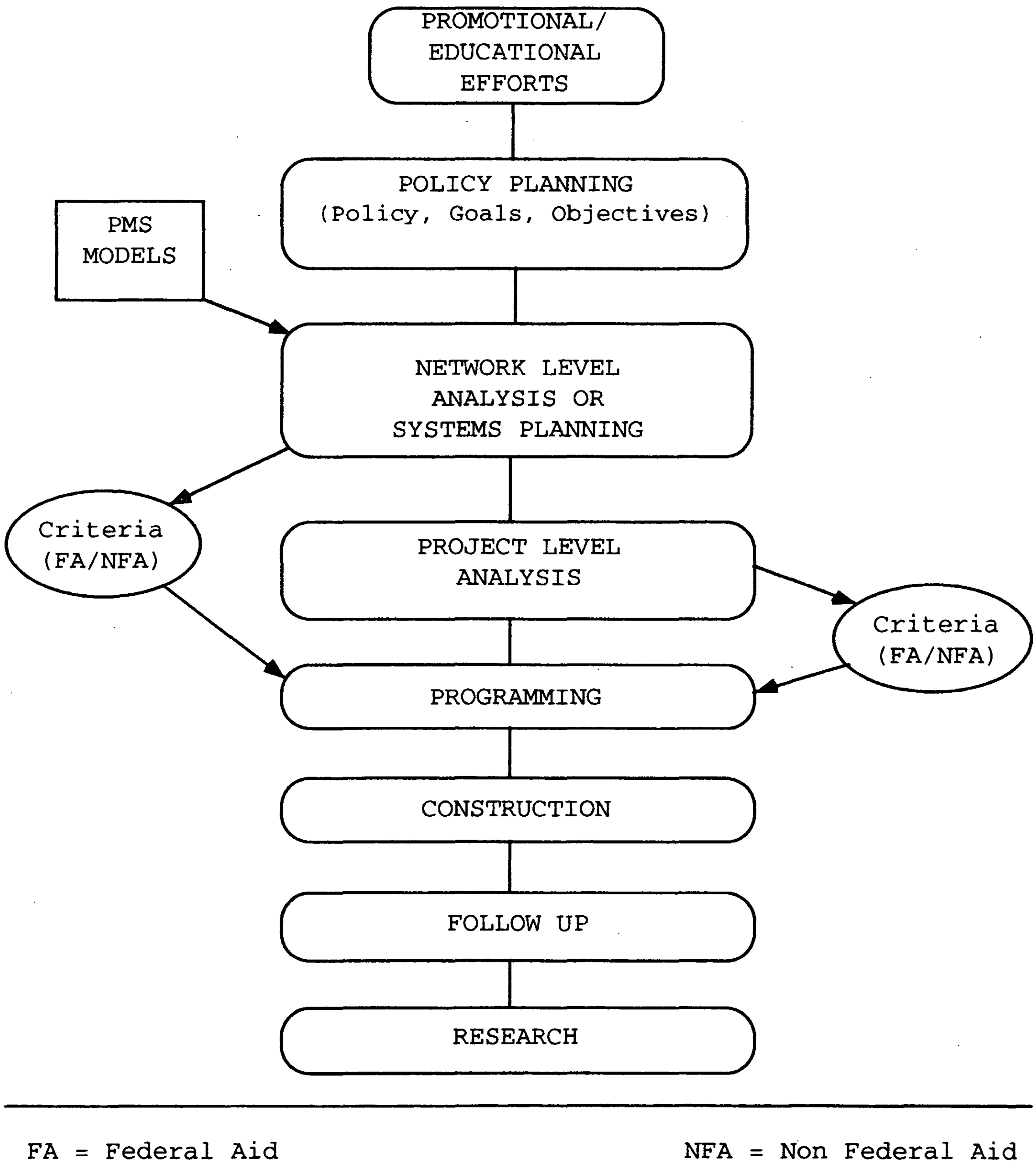


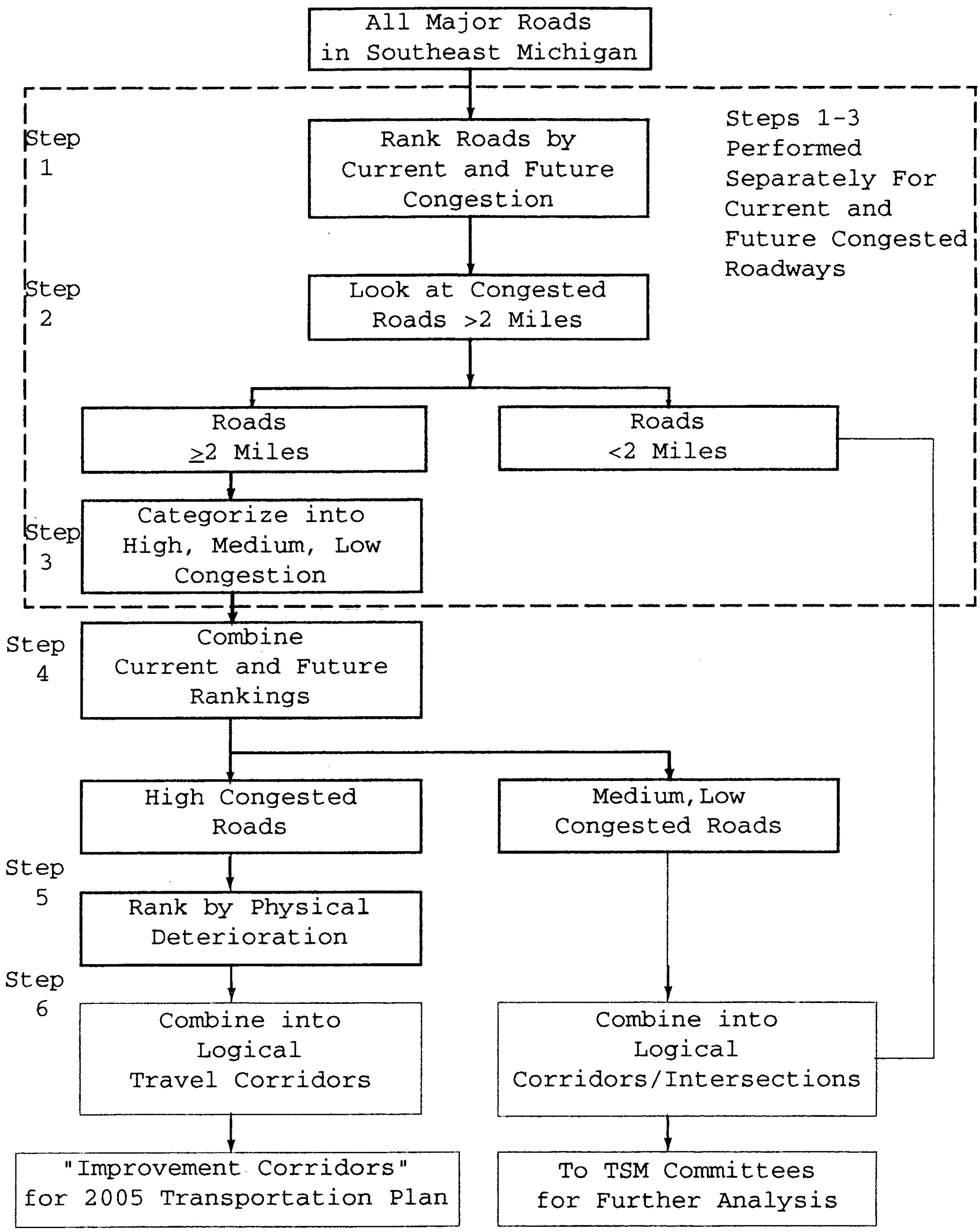
FIGURE 2 Framework for MPO involvement in PM.

TABLE 5 Framework Activities

MAJOR ELEMENTS	ACTIVITIES
Promotion	* public presentations on PM * design of PM brochures for cities and towns
Policy Planning	* past and current maintenance policies assessment * definition of managerial and physical objectives
Network Level Or Systems Planning	* network sections definition * data collection * data analysis * priority setting (network sections) * budgeting * implementation of results * training
Project Analysis	* coordination of pre-constructed activities * detailed engineering design * selection of best alternative * economic analysis * budgeting
Programming	* programming reconstruction & rehab. into TIP including fed-aid roads and non fed-aid roads * integration of network and project level in the development of local master plans
Construction	* contract scheduling & control * construction inspection * actual construction
Follow Up	* overall PM process monitoring * data base update * changes to overall process * information dissemination about PM activities through MPO newsletter or RTAP sources
Research	* PM process assessment * pavement performance evaluation * performance models development * cost effectiveness of maintenance and rehabilitation strategies evaluation

improvement policies. Local agencies would be able to work with the MPOs in investigating alternative funding sources in situations in which such needs arise, and longer lives would be achieved for roads before they require substantial rehabilitation or reconstruction. Perhaps this will address the needs of TIP and reduce the number of roads receiving federal and state aid that need to be programmed.

Results of this study highlight that pavement management efforts of those MPOs that have participated in local pavement management have been extensive. In addition, the contribution of RTAP centers in local pavement management has been quite significant. For example, the Baystate Roads Program has contributed toward the promotion of local pavement management through several workshops held in the commonwealth of Massachusetts and has worked closely with cities and towns in implementing such programs. The New Hampshire RTAP center has been instrumental in the development and testing of a personal computer-based pavement management software package. However, on the basis of the information in the literature and a limited survey conducted as part of the present project, only a small number of MPOs in the country have been involved in local pavement management, and furthermore, those MPOs that have been involved have not incorporated the results of such pavement management projects into the 3C process. It should be noted that local PM results in Massachusetts have not been incorporated into the 3C process, because it was not clear to the RPAs how it should be done. The results were documented in a report that was given to the local city or town officials, some of whom used the results for local programming and budgeting purposes. At present, one RPA (PVPC) is considering the inclusion of such results in its TIP and is formulating an approach similar to the one used by southeastern Michigan's COG. It is also noteworthy that a local PM workshop was held at the University of Massachusetts in which a hands-on session addressed the need to



Source: Southeast Michigan Area Council of Governments.

FIGURE 3 Corridor ranking process.

consider integrating results into the TIP. In that session the participants (six to seven RPAs were represented) were given fictitious results of PM studies and other transportation analysis projects, and they had to conduct an evaluation using an approach similar to the process used in southeastern Michigan, and then each of the groups had to present their findings, conclusions, and recommendations. It is believed that the development of a regional transportation plan that predominantly addresses highway capacity and safety needs would benefit from the inclusion of pavement condition needs.

The present study has suggested that the role of an MPO in a local pavement management program can be achieved through one or more of the eight major elements represented in the framework. However, the roles and responsibilities of the MPO in the eight elements may vary because of a number of factors such as local

TABLE 6 Possible Roles and Extent of MPO Involvement in Framework Elements

FRAMEWORK ELEMENTS		EXTENT OF INVOLVEMENT	ROLE
PROM./EDUC.	→	MEDIUM-HIGH	* INITIATOR
POLICY PLANNING	→	LOW-MEDIUM	* FACILITATOR
NETWORK LEVEL or SYSTEMS PLANNING	→	MEDIUM-HIGH	* FACILITATOR * TRAINER * DOER
PROJECT LEVEL	→	LOW	* COORDINATOR
PROGRAMMING	→	HIGH	* DOER
CONSTRUCTION	→	LOW	* COORDINATOR
FOLLOW UP	→	MEDIUM-HIGH	* DOER
RESEARCH	→	LOW-MEDIUM	* FACILITATOR * COORDINATOR * DOER

LOW - 2-4 person-days / month
MED. - 4-8 person-days / month
HIGH - 10-15 person-days / month

agency resources in terms of equipment and manpower, MPO and local technical capabilities, and the overall willingness of the MPO and local communities to make commitments to the pavement management program.

The possible use of technical support from outside the MPO and local agency needs to be considered. Some MPOs contracted with private consulting firms that have the experience in the selected framework elements that does not exist in the MPO; other MPOs have made use of the expertise of RTAP centers. It should also be emphasized that the required technical support for training and dissemination of information can be accomplished with support from various RTAP centers and state highway agencies.

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Network-Level Prioritization of Local Pavement Improvements in Small and Medium-Sized Communities

CORNELIUS W. ANDRES AND JOHN COLLURA

Theoretical and pragmatic problems surrounding network-level prioritization of pavement management activities for local communities are examined. Although the principles of pavement management are the same for all agencies, pavement management at the local level is generally somewhat different from that at the state level because of dissimilarities in resources and responsibilities. Instead of creating a new management system a public domain software package, developed by the Metropolitan Transportation Commission of Oakland, California, was used as a framework for the analysis. This is a comprehensive but low-cost system that can be run on a personal computer. Because much of the research on which this system is based has already been published, an overview of only some elements of the system is presented. The focus is on the use of the system for prioritization of treatment for a local community facing budget constraints. This was accomplished through the use of computer-modeled budget scenarios. This type of analysis proved to be a practical tool for conducting multiyear network-level prioritization at the local level. Analysis, however, is more complicated than just printing out a list. The principal finding was that there is a means of incorporating the link between priority assessment and roadway funding into the network-level prioritization process. This was accomplished by varying treatment selection as funding levels changed. Merely ordering a set of alternatives that were found to be optimal at the project level did not maintain a roadway network in acceptable condition when funding was constrained.

A primary purpose of a pavement management system (PMS) is to provide information so that roadway improvements can be priority ranked (*1*). Ideally, prioritization is a consistent and justifiable process. It should involve minimizing life cycle costs subject to minimum levels of serviceability and budget constraints. This is no simple task. Prioritization is a complicated process that requires sound engineering judgment and a good understanding of local conditions.

Bad prioritization decisions can lead to costly future problems. Current fiscal crises and rising roadway improvement costs have made prioritization decisions more important than ever. The pavement management literature, however, provides little prioritization guidance for use at the local level. This paper examines theoretical and pragmatic problems surrounding the prioritization process and includes a methodology for addressing these problems.

PROBLEM STATEMENT

It is not uncommon for a prioritized list of projects generated by a computerized PMS to bear little resemblance to the work that a community's qualified highway official thinks should be done first. Differences are typically most striking in times of severe fiscal crisis, the times when prioritization guidance is most needed. The cause of this deficiency seems to be that the prioritization procedures in many PMSs are simplistic and do not necessarily reflect the views or constraints of decision makers. At the local level, projects are typically ranked by measures such as pavement condition, rideability, or composite ratings that incorporate other factors such as traffic volume and accident history. Prioritization is then accomplished by procedures such as "best first," "worst first," or percentage-based approaches that divide resources between maintenance, rehabilitation, and reconstruction. These methods are based on common roadway management strategies. Little research, however, has been done to verify the actual efficiency of these strategies. Mathematical procedures exist that can be proven to optimize the allocation of resources. These procedures, however, are complicated and have not yet been applied at the local level.

Because the goal of this research was to develop prioritization guidance for use by local and regional agencies, there was a balancing of efficiency and effort. An efficient prioritization scheme is not useful if it requires unrealistic amounts of data, expertise, and time. Pavement management is a continuing activity. A prioritization procedure should therefore provide useful results, and implementation and updating of the procedure should be able to be done in a practical manner. If not, this element of a pavement management system may be of little use to public officials. Simplicity and practicality are therefore stressed in the final recommendations.

ROLE OF PRIORITY ASSESSMENT IN PAVEMENT MANAGEMENT

A primary step in the establishment of a local PMS is to determine a level of funding that is adequate to maintain roadways in acceptable condition at a minimal life cycle cost. When funding is constrained somewhat below this level in the short run (less than 5 years), as often happens in local communities, preventive maintenance, which is relatively inexpensive, becomes especially important, even if not optimal at the project level. When fiscal crises that reduce roadway funding arrive, however, local highway officials are forced to save what pavements they can and let the rest go. The costs in this situation are generally passed on, by default, to motorists in the form of higher user costs. For example, a community facing extreme fiscal constraints may be forced to patch and seal high-volume roadways instead of overlaying them and skip maintenance of residential roadways altogether. Over the long run such strategies probably cost more than proper maintenance and rehabil-

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itation. They also result in higher user costs because of pavement roughness. However, this may be the best option for maintaining a local roadway network if funding is just not available.

Prioritization procedures should reflect this scale of fiscal circumstances and be able to assist a community in spending limited funds in the best way possible. This is especially important in today's fiscal climate.

METHODOLOGY

This section describes methods used to prioritize roadway improvements and outlines the information needed before prioritization can take place.

Prioritization Indexes

Prioritization indexes are used to order needs. The simplest indexes are based on pavement distress or roughness. Composite indexes are formed when more than one serviceability indicator is combined into a single index. An example of a composite index is a pavement condition index (PCI), which combines measures of distress and roughness. Composite indexes may also incorporate variables such as traffic volume, drainage, and accident history. The next level of prioritization indexes is based on pavement performance. Performance is typically estimated through deterioration curves. The area under these curves can serve as a proxy for user benefits (*2*), which is a measure of effectiveness. This approach takes advantage of the deterioration curves that are already required by multiyear analysis procedures for condition projection. This is a higher-order approach that considers future as well as present pavement conditions. It should be noted, however, that the results are only as good as the original deterioration curves.

Any of these indexes may be cast in a cost-benefit framework to provide additional information to decision makers. In the case of a cost-effectiveness rating, this is accomplished by dividing effectiveness by cost of treatment. This cost-effectiveness rating must be weighted by traffic volume. This is necessary because treatment applied to low-volume roadways typically costs less per year than that applied to high-volume roadways. For example, a thin overlay on a residential street may last as long as a thick overlay on a principal collector but will obviously be less expensive per year of life. The weight is a means of ensuring that these lower-cost treatments for low-volume roadways are not necessarily ranked above the higher-cost treatments applied to heavily traveled roadways. It is a means of normalizing the rating to account for the higher number of users. Additional details on weighted effectiveness ratings can be found in research done by Andres (*3*).

Computer-Modeled Budget Scenarios

The most realistic means of assessing near-term (5-year) local roadway treatment priorities appears to be through the use of computer-modeled scenarios. This type of network simulation approach can be superior to mechanical spreadsheet-type approaches in the analysis of complicated problems (*4*). Modeled scenarios also allow preventive maintenance such as sealing to be considered, and penalties (stopgap maintenance such as pothole patching) can be assigned when treatment is delayed.

The Metropolitan Transportation Commission (MTC) of Oakland, California, has developed a pavement management system that uses this type of procedure to evaluate the impacts of alternative funding levels (*5*). The system uses a composite performance-based index to assess priorities. The index is formed by first determining an effectiveness ratio. This is accomplished by dividing "benefits" (areas under deterioration curves after treatment is assigned) by the cost per square yard of treatment. This effectiveness ratio is then weighted by functional class. This measure of the influence of traffic volume on priority assessment can be modified by the user. The details of weight selection are referenced in the user's guide (*5*).

During the scenario procedure percentages of the agency's budget are allocated to rehabilitation and preventive maintenance. The weighted effectiveness ratio is then used to prioritize road segments in each category. Stopgap maintenance can be assigned when treatment is delayed. Ideally, the determination of the split between preventive maintenance and rehabilitation would be derived through the use of an optimization procedure. Unfortunately, true optimization procedures are currently too complex for practical use at the local level. They also typically require unrealistic amounts of long-term data (20-plus years). After true optimization procedures are refined at the state level and better local data are accumulated, such procedures may prove useful to local communities. At present, however, such procedures would most likely be "black boxes" with limited usefulness. For the present analysis the split was determined through trial and error. The goal was to achieve the best overall network condition subject to conditions such as keeping agency forces gainfully employed, limits on contractor capabilities, and political considerations.

Budget scenarios proved to be an excellent format for prioritizing local roadway needs. "What if" experiments with budget levels and treatment selection can be rapidly simulated on a computer instead of on the actual roadway network. It is a powerful tool that allows highway officials to substantiate the results of their engineering judgment. This method of priority assessment is probably the most advanced procedure readily available for use at the local level. It is not as pleasing theoretically as true optimization, but the measure of benefits that are ranked (the areas under deterioration curves) is often the same as that used for true optimization. The effort described here therefore adopted the MTC system as a framework for an analysis of prioritization at the local level, but it is anticipated that the results of this analysis can be applied to other PMSs.

Prerequisite Tasks

This section briefly describes the steps that must be accomplished before prioritization can take place. It should be noted that these steps are similar for many PMSs.

Inventory

During the inventory phase the roadway network is broken into management sections. Data such as length, width, date of construction, pavement type, and functional classification are then obtained for each section.

Condition Survey

Pavement condition can be measured in several ways. Some PMSs use measures of rideability or roughness. This is usually done for high-speed roads such as Interstate highways. Other systems measure pavement deflection under loads. Deflection measurements of structural capacity are typically used for detailed project-level analyses. Measures of surface distresses, however, are best suited to network-level use on local roads.

The MTC system uses a PCI to measure pavement distress. The PCI is a scale with a range of 100 to 0. A pavement with a PCI of 100 is perfect. The distresses measured were alligator cracking, block cracking, distortions, longitudinal and transverse cracking, patch and utility cut patch, rutting and depression, and weathering and raveling.

The distresses are measured by a sampling approach. This consists of a detailed examination of at least 10 percent of the pavement in each roadway section. This approach is much less expensive than examining the entire segment. An extensive body of literature, documented in the MTC PMS user's guide (*5*), concludes that a sampling approach provides adequate data for a network-level analysis. During the field test sampling was found to give consistent results, especially for pavements in better than fair condition. If excessive variation is found between the conditions of samples on the same management segment the computer flags that segment for additional inspection.

The sampling procedure is more complicated and time-consuming than a "windshield" survey in which pavements are rated while the inspector drives at 5 to 10 mph. During a windshield survey pavements are usually rated qualitatively, that is, excellent, good, fair, and poor. This type of information, however, is not generally adequate for use with deterioration models. The information gathered by sampling is more detailed and accurate. These qualities lead to better estimates of future roadway conditions. Knowledge of the type, severity, and quantity of each distress also allows reasonable network-level estimates of maintenance expenses to be made.

Because manual condition inspection is time consuming, the entire roadway network is not reinspected yearly. The critical pavements to be inspected are those that are anticipated to cross decision thresholds. The MTC software therefore generates a reinspection schedule on the basis of anticipated deterioration.

Condition Projection

Deterioration curves are used to accomplish condition projection. These projections are used to identify when maintenance and rehabilitation will be required, determine future budget needs, and prioritize treatment when funding is constrained.

The MTC system assumes that pavement deterioration takes the form of a reverse S-shaped curve. This choice of form was based on research from Texas A&M University (*6*) and Cornell University (*7*), which used regression analysis to relate pavement condition to age. The reverse S-Shape seems theoretically sound. One would expect that pavements would deteriorate slowly at first and that deterioration would quicken over time. This increase in the rate of deterioration is especially likely in regions that experience abundant precipitation and hard frosts. In these regions a sharp drop off in condition would be expected as winter exacerbates existing defects. If a reverse S-shaped deterioration curve is inappropriate for a region or pavement type, such as overlays, the software could be modified.

The basic functional form of this relationship curve can be expressed as

$$\mathrm{PCI} = 100 - \left[\frac{R}{(\ln A - \ln \mathrm{age})^{1/B}} \right] \tag{1}$$

where

PCI = pavement condition index,
age = age of pavement,
ln = natural logarithm, and
R, A, B = regression coefficients.

The limit to the drop in later years is determined by the A coefficient. Examination of the PCI formula shows that it becomes undefined when age is equal to A. Because pavements rarely, if ever, reach a PCI of 0, the segment of the curve below the designated failure PCI should be disregarded.

Accurate pavement condition projections are dependent on the suitability of the deterioration curve for the pavement being modeled. The choice of a deterioration curve, however, is complicated by the extreme variability of pavement life. This variation is due to differences in factors such as native subgrades, road construction methods and materials, quality control, drainage, traffic loadings, and environmental factors.

This variation is handled in two ways. First, pavements are separated into the categories that were discussed in the data section (functional classification, pavement type). Pavements within these categories are assumed to deteriorate in a similar fashion. Therefore, they are assigned a family deterioration curve. These are typical curves for average pavements in each class.

Second, additional variation within these categories is accounted for by adjusting the family curves. The adjustment procedure acknowledges that there will be variation in pavement condition, but assumes that future deterioration will occur in a fashion similar to that category's family curve. There are two procedures used by the MTC package to adjust the deterioration curves. They are illustrated in Figure 1. The curve either is adjusted up or down or is shifted horizontally. The appropriate procedure will be determined by the observed condition and age of the particular road segment.

If the observed point falls within the confidence limits shown in Figure 1, the deterioration curve is adjusted up or down. The MTC user's guide calls this procedure "adjusting the curve." It is a percentage-based procedure that entails multiplying the deduct value by an adjustment factor. The following equation is used to calculate the adjustment factor:

$$\mathrm{ADJ\ FAC} = \left(\frac{100 - \mathrm{PCI}}{100 - \mathrm{PCI}_f} \right) \tag{2}$$

where

ADJ FAC = adjustment factor,
PCI = observed PCI, and
PCI_f = family PCI.

The adjusted PCI is represented by the following equation:

$$\mathrm{ADJ\ PCI} = \mathrm{ADJ\ FAC} \ast \left(\frac{R}{(\ln A - \ln \mathrm{age})} \right)^{1/B}$$

The upper and lower confidence limits can be thought of as the best and worst pavement deterioration scenarios. When a specific segment falls in between these ranges its deterioration is interpolated (*8*). The default value of the confidence limits is plus or minus 50 percent of the drop in the PCI. This value for the confidence limits was deemed "reasonable" in the MTC user's guide. It produced good results for this analysis but can be modified if necessary. Eventually, after more deterioration data are gathered for the family of pavements, these limits might be tightened.

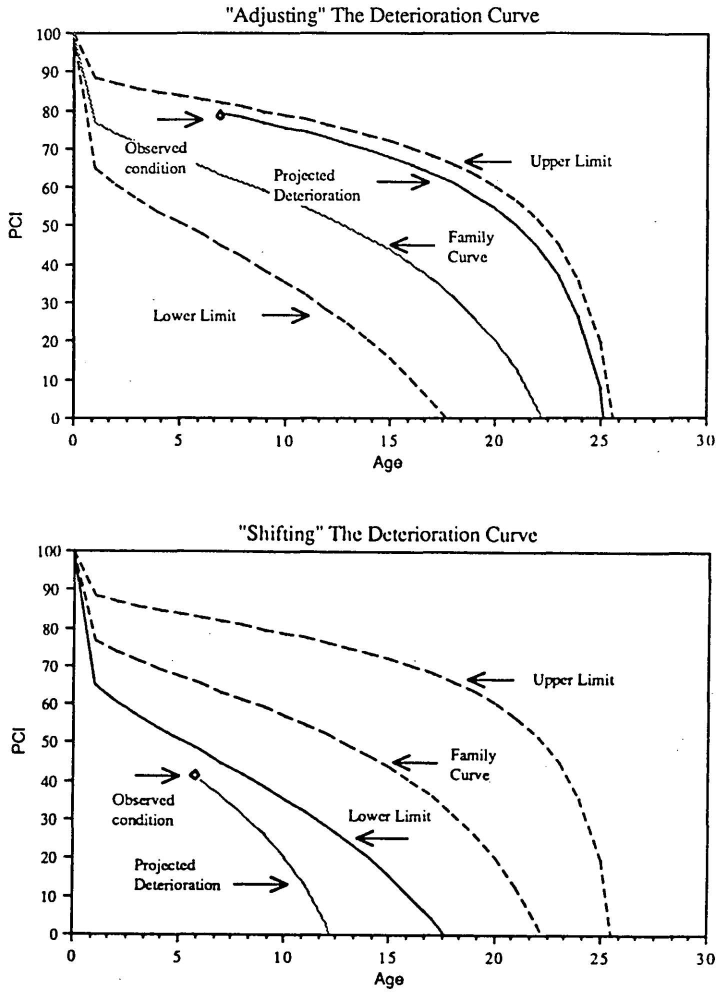

FIGURE 1 Adjustments to family deterioration curves.

If the observed point falls outside these limits the curve is simply shifted horizontally. This procedure is called "shifting the curve." The reason for having two procedures is that adjusting the curve can result in an unrealistic condition projection if the observed point is outside the confidence limits. This could also result in the inability to calculate the area under the deterioration curve. This procedure is equivalent to assuming that the date of construction is wrong or that maintenance or rehabilitation was not recorded.

Rehabilitation and maintenance will affect PCI. Rehabilitation, such as an overlay, will raise the PCI of a segment to 100. Maintenance, such as crack sealing, will turn medium- and high-severity cracks to low-severity cracks. A new PCI is then calculated. The effects of rehabilitation on PCI were calculated through regression analysis of a large sample of actual projects. These default coefficients can be modified by the user, but they appeared suitable for use in the study area.

Treatment Selection

To develop multiyear work plans and examine the effects of various funding levels, future pavement maintenance and rehabilitation actions are modeled. This is accomplished through the use of a set of decision criteria that are triggered by pavement condition. At specified thresholds user-defined treatment is assigned. Treatment is based on pavement type, functional classification, and condition. At this point it should again be stressed that this is network-level analysis. It is only meant to predict probable maintenance for typical pavements to anticipate budget needs. The actual maintenance to be done on specific pavement segments is determined from detailed project-level analyses.

ANALYSIS

This section details the prioritization of local roadway improvements for the town of Eastham, Massachusetts. This network-level analysis was conducted as a pilot project by the Cape Cod Commission, a metropolitan planning organization (MPO). The project was sponsored by the Massachusetts Highway Department and FHWA. This type of technical assistance is continuing at the Cape Cod Commission as well as at other MPOs across the country. These local pavement management efforts by MPOs are supported by the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA).

Budget Needs

Before prioritization can take place budget needs must be determined. This was accomplished through the application of the treatment selection procedure discussed above. Application of this algorithm produces a list of treatments anticipated to minimize the life cycle cost of each management section. The budget needs module of the MTC PMS produces a list of selected treatments and the weighted effectiveness of each selection in the present year. The budget needs analysis was conducted in current dollars. The MTC program, however, does allow the use of a discount rate.

The weighted effectiveness ratings are used to prioritize roadway improvements when funding is constrained. This is accomplished through the use of computer-modeled scenarios. In the future the MTC plans to develop a procedure to calculate current weighted cost-effectiveness for all potential projects in each analysis year rather than just calculating them all in the beginning (Sachs, unpublished data). This improvement will be possible because of the phenomenal increase in microcomputer capability in recent years.

The budget needs analysis indicated that the sample community should spend just over \$1 million during the next 5 years to maintain its roadway system adequately. The unconstrained budget needs analysis, however, did tend to "front load" needs. It indicated that the town should spend more in the early years. This is due to having to catch up with maintenance and the fact that repairs become more expensive with time. The budget needs analysis procedure produced the following results:

1991	\$311,159
1992	\$314,336
1993	\$276,699
1994	\$99,538
1995	\$2,416
	\$1,004,148

The unconstrained analysis is an initial cut at determining maintenance and rehabilitation requirements. But fiscal constraints, which tend to favor uniform yearly appropriations, need to be considered. This initial budget option was infeasible for the sample community. The typical skew of an unconstrained budget toward the present, however, suggests the possibility of a road bond issue for long-term improvements as a means of minimizing roadway expenditures.

Budget Scenarios

Budget scenarios are modeled over a 5-year period. Two methods of specifying yearly budgets are available. In the first method an initial budget and optional budget increase factor is selected. The second method entails specifying individual yearly budgets. This allows the analysis of nonuniform allocations such as bonding. In either case a split between preventive maintenance and rehabilitation is specified.

During scenario analysis the computer selects projects on the basis of the weighted effectiveness index until the rehabilitation allotment is exhausted. If rehabilitation expenses are less than the budgeted amount the remainder is allocated to preventive maintenance for that same year. Those sections identified for rehabilitation but not selected are deferred until the next year. Stopgap maintenance costs, such as emergency patching, can be assigned as a penalty for delay. Stopgap repair expenses are subtracted from the preventive maintenance allotment. Management sections identified for preventive maintenance are then selected on the basis of the same weighted effectiveness rating. This process is repeated yearly.

The end results of the scenario procedure are detailed, and summary reports that document assigned treatment, surplus and deferred expenses, and network conditions are prepared. Scenarios can be rerun until, in the highway official's judgment, the best strategies and funding levels are determined. Budget scenarios may be run on any subset of management sections.

Although an infinite number of budget options is possible, only four funding levels are presented. Prioritization was accomplished through somewhat different means at each of these funding levels. These funding levels are a concise format for presenting budget options to decision makers. The presentation of too many options confuses decision makers. Although not presented in this analysis, the development of a road bond option would be straightforward.

Zero Funding

The impacts of zero funding of the roadway maintenance and rehabilitation budget were calculated as a benchmark. This, however, was actually done in the sample community the year that the present study was conducted. The analysis of zero funding required no prioritization decisions. The impacts of this funding level over the next 5 years are illustrated in Figure 2. It can be seen that this strategy will result in the average condition of the sample community's roadway network decreasing from very good (PCI = 75) to good (PCI = 61). Although this average condition may not appear to be too bad, a closer examination reveals that almost 7 percent of the town's roadway network would be in very poor or failed condition (PCI <25) within 5 years under the zero funding scenario. The town would also find that it would have no choice but to incur repair

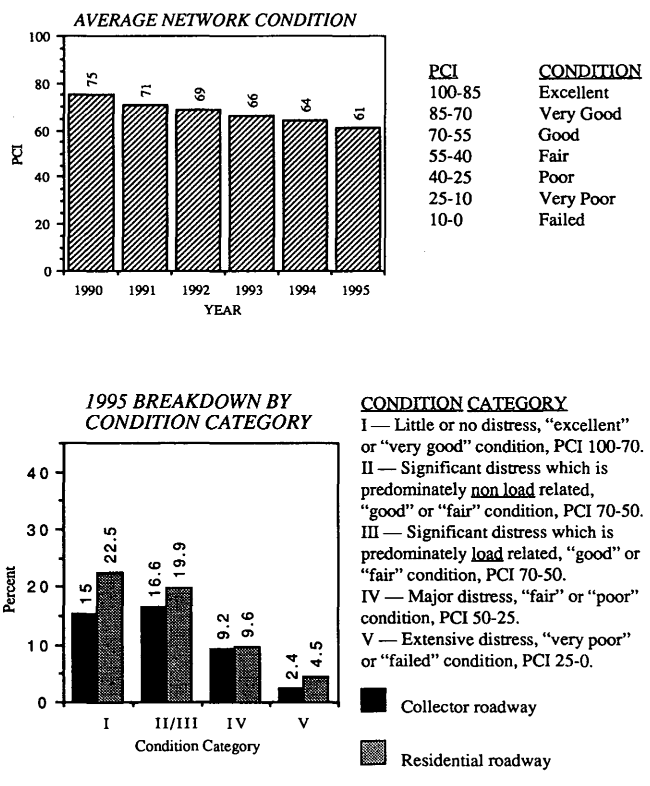

FIGURE 2 Projected roadway conditions with zero funding.

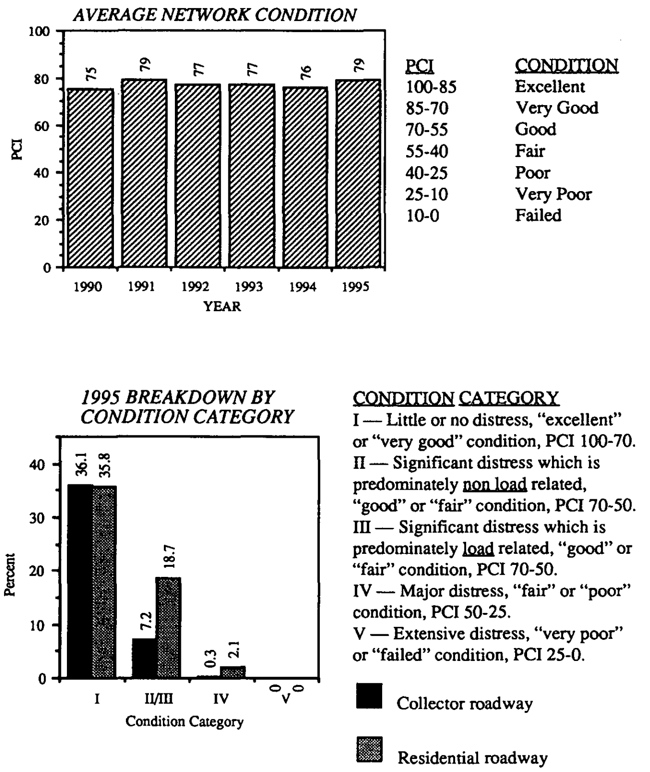

FIGURE 3 Projected roadway conditions with $200,000 annual budget.

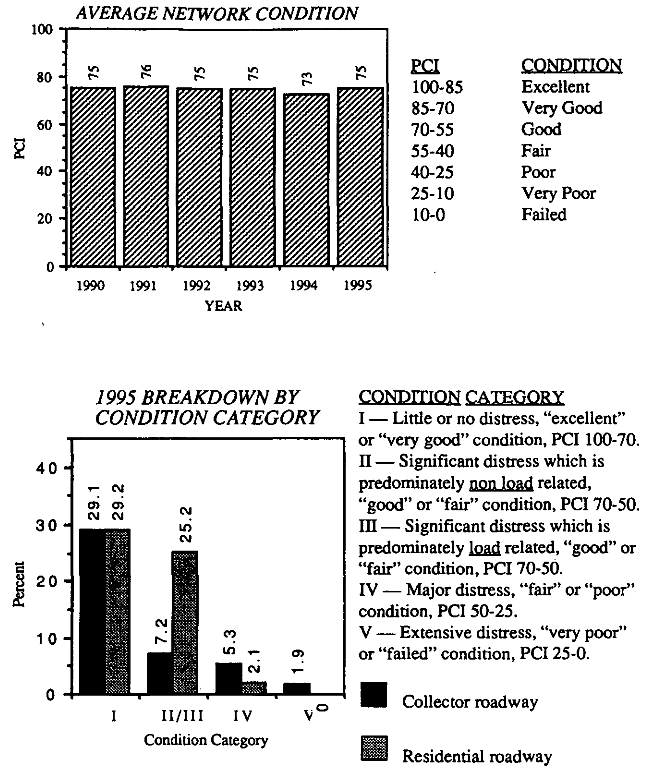

FIGURE 4 Projected roadway conditions with $100,000 annual budget and original treatment selection.

expenses much larger than would have been necessary if it had budgeted timely maintenance.

Full Funding

A uniform maintenance and rehabilitation budget was favored by decision makers in the pilot community. After repeated application of the budget scenario procedure, a $200,000 maintenance and rehabilitation budget was developed as a full funding option. A yearly maintenance and rehabilitation appropriation of this amount would allow for preventive maintenance and annual capital improvements such as upgrading sections of collector roadways from surface treatment to hot mix and the resurfacing of some residential roadways. Figure 3 shows that this level of funding would raise the average condition of the sample community's roadway network. It would also prevent any roadway segment from deteriorating to a poor or failed condition.

Moderate Budget Cuts

This budget level was presented as a compromise. It would allow preventive maintenance such as crack sealing and sand sealing. It is not sufficient, however, to allow for annual capital improvements. The impacts of this level of funding are presented in Figure 4. The end result would be a fairly uniform average network condition, but a percentage of the collector streets would deteriorate into poor or failed condition. The collector roadways were not assigned treatment because the rehabilitation costs for a section of this type of roadway were generally greater than a whole year's rehabilitation budget. No matter what the weighting it is impossible to assign rehabilitation if the total required for these improvements is greater than the whole budget.

Because of the higher user costs resulting from defects on collector roadways, most highway officials would do their best to avoid a situation in which these roadways are failing. The first option examined to prevent this situation was to shift funds from preventive maintenance to rehabilitation. The high effectiveness ratings and low costs for preventive maintenance activities, however, lead to the conclusion that this was an unwise strategy. It creates a cycle of dealing with problems at the point where they are most expensive to fix.

The next option was to assign treatments with lower initial costs. These were treatments that were excluded from consideration initially because of higher life cycle costs. Examples of these types of treatment are wedging or chip sealing a roadway in poor condition instead of reconstruction. Veteran highway officials indicate that they know that such treatment costs more over the long run, but at times they cannot afford to do anything else. These types of treatments can be necessary to prevent the loss of infrastructure investments such as roadway bases. Although they do not make sense at the project level, they can, at times, be the best strategies at the network level. This is especially true at the local level, where user costs because of repeated treatment, such as chip seals, are not as high as user costs on heavily traveled state highways. Local communities typically go through cycles in which funding is constrained. These types of holding strategies can allow a community's roadway investment to be preserved until funding constraints are relaxed.

Modifications to treatment selection were accomplished by first examining detailed budget needs reports to determine what expenses could be eliminated or postponed. That review determined that rehabilitation of residential asphalt concrete roadways was a major expense that would probably be delayed. It was impossible to avoid these selections by modifying the weight assigned to functional classification because by doing so many low-cost but effective selections, such as heavy sand seals for residential surface-treated roadways, would be eliminated. It was therefore necessary to modify the treatment selection procedure. Because none of these pavements was near failure a decision was made to assign "do nothing" instead of rehabilitation. The probable long-term effect of this decision will be higher life cycle costs for these pavements.

The next modifications to treatment selection were changes to the maintenance and rehabilitation strategies for collector roadways. First, maintenance overlays for roadways in "good" condition were eliminated. Second, a staged rehabilitation approach that was successfully used in the past was adopted. This strategy consisted of leveling and placing a 2-in. lift of dense binder, which also served as a wearing surface.

Unfortunately, the consequences of actions such as these, which are contrary to pavement management philosophy, cannot be adequately illustrated with a 5-year analysis. It should be stressed that these strategies are only buying time. Eventually a significantly higher annual funding level or a road bond will be required. The results of these changes in treatment selection are illustrated in Figure 5. It can be seen that although there are higher percentages of pavements in the lower PCI ranges, roadway failures and the associated dramatic increases in user costs are avoided. Again, this is accomplished at the expense of higher future costs for some management segments that are currently in acceptable condition.

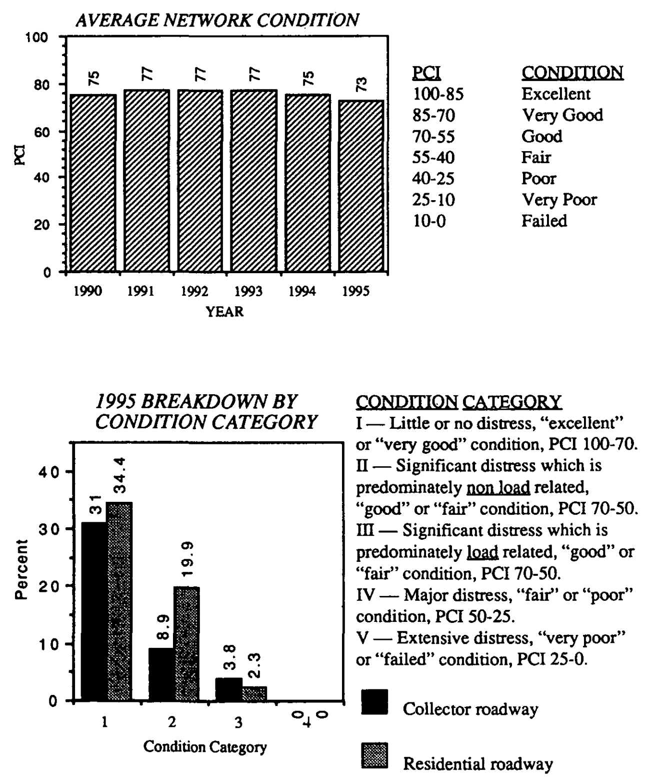

FIGURE 5 Projected roadway conditions with $100,000 annual budget and modified treatment selection.

Calculation of life cycle costs followed by trial-and-error modifications to treatment selection and use of the budget scenario procedure will be required to determine a community's least painful alternatives. User costs such as traffic delays, user inconvenience, and higher operating costs should be considered. These costs, however, are much less significant for low-volume, low-speed local roadways than for state highways. Emphasis should be placed on determining the least-cost holding strategies that minimize increases in future funding requirements. It should be stressed again that this requires extensive engineering judgment. The computer will not make decisions for the highway official, but it is a tremendous tool for examining the consequences of particular strategies. It facilitates the development of a defendable 5-year plan.

Severe Budget Cuts

Prioritization under severe budget cuts is difficult to determine through the use of budget scenario models. In this situation the most effective treatment for the network is generally preventive maintenance. It was found, however, that it took more time to get the model to come up with accurate estimates of preventive maintenance needs than it did simply to use current preventive maintenance budgets and the information contained in the data base to develop a bare-bones preventive maintenance budget.

In the case of the sample community it was determined that the town should, at a minimum, fully fund preventive maintenance actions to preserve its substantial roadway investment. Preventive maintenance includes the sand seal program, the drainage program, and the adoption of a crack sealing program. Collector roadways should be given priority. The following yearly appropriations were recommended:

Sand sealing	$50,000
Crack sealing	$10,000
Drainage	$15,000
	$75,000

This funding strategy will not prevent some roadways from failing, but it will hold most roadways in acceptable condition until adequate funding is available. Additional funding for emergency maintenance, however, will probably be required in the future. Motorists will also pay for deferred maintenance through higher user costs.

RESULTS

This paper documents the results of using the MTC PMS, a personal computer-based software package in the public domain, to conduct a network-level analysis of a local community's roadway system. Prioritization was accomplished through the use of computer-modeled scenarios that employed weighted cost-effectiveness as a prioritization index. The key result is the identification of the prioritization linkage between funding level and treatment selection.

Without budgetary constraints the goal of local pavement management is to provide acceptable pavement at a minimum life cycle cost. Prioritization is not an issue. Network-level funding is determined through the use of a treatment selection algorithm that identifies treatment and timing to achieve these objectives. In times of constrained funding, however, only a limited number of these identified treatments can be selected. Prioritization research has concentrated on selecting the best mix of identified treatments, with the objective being to choose an optimum combination of projects. This is illustrated in Figure 6. Little attention, however, has been paid to verifying that the items on the list of potential treatments that are being ranked are indeed the best choices for maintaining a roadway network at a given funding level.

To maintain a roadway network in the best possible condition with constrained funding, local highway officials are often forced to make decisions that are, at the project level, suboptiminal. For example, a roadway segment that would ideally receive a thick overlay might, instead, receive a chip seal that will last only 3 years. This may seem to be a waste at the project level. But at the network level it may be possible to treat 10 times the amount of pavement with this strategy. Instead of having one excellent roadway and nine pavement failures the community would have maintained acceptable surface conditions, saved the roadway bases, and bought time in which to accomplish the required work. Holding strategies such as this one, however, are often not in the subset of treatments that are being prioritized. Treatment selection must therefore be modified at lower funding levels to include alternatives that may be optimal only at the network level. If project-level decisions are allowed to routinely override network-level suggestions, a PMS is not serving its best use.

An alternative that must be considered when funding is constrained is to delay treatment. This can be accomplished by assigning "do nothing" as treatment. For example, a residential reconstruction job may have to wait until it is projected to cross the next threshold before treatment is assigned. This option would not be considered by a prioritization procedure if the linkage between treatment selection and prioritization was not examined.

Ideally, modifications in treatment selection would be based on a network optimization model. At present, however, engineering judgment, life cycle cost analysis, and repeated application of the budget scenario procedure will have to suffice at the local level.

In times of severe fiscal crises the goal of local pavement management is to save those pavements that can be saved. This triage is best accomplished through an emphasis on preventive maintenance on selected pavements. Examination of weighted effectiveness ratios for preventive maintenance confirms this. Extensive manipulations of computer scenarios are generally not necessary to assign preventive maintenance or determine funding levels under these circumstances.

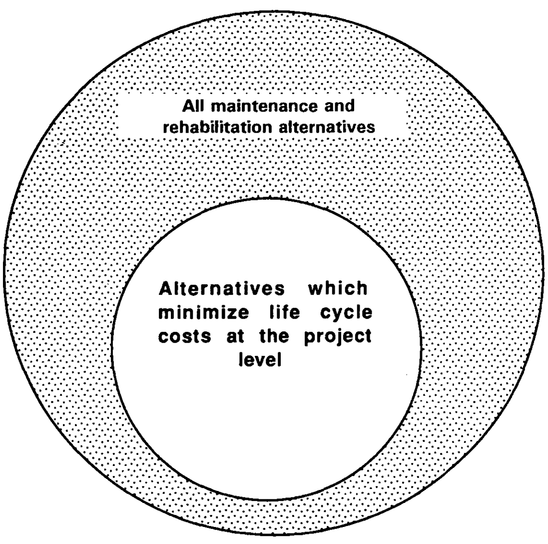

FIGURE 6 Sets of alternatives.

SUMMARY AND CONCLUSIONS

The central element of this paper was a network-level analysis of a local community's roadway network. The analysis was conducted with computer-modeled scenarios that employed a weighted cost-effectiveness ratio as a prioritization index. The areas under pavement deterioration curves served as proxies for user benefits.

This type of network simulation procedure proved to be a practical framework for conducting network-level prioritization of pavement improvements at the local level. The procedure had to be modified, however, to reflect local conditions. This customization requires an investment of time and funding to produce valid results. When there is a commitment to maintaining a PMS, this is a well-spent effort. If a community cannot make such a commitment, it should adopt and maintain a simpler system. Commitment to a good record-keeping system will make it relatively easy to graduate to a higher-order PMS in the future.

The principal finding was a means of incorporating the link between priority assessment and funding level into the network prioritization process. This was accomplished by varying the treatment selection process as funding levels changed.

It was found that at full funding the goal of pavement management was to assign those actions that were optimal at the project level. These are the actions that minimize life cycle costs. Prioritization is not an issue. When funding was constrained, as is often the case at the local level, the goal of network-level prioritization was to maintain a roadway network in the best possible condition at a minimum long-run cost. This involved some decisions that, at the project level, appeared to be suboptimal. If, however, a prioritization procedure is ordering only the subset of actions that minimize life cycle costs at the project level, no amount of mathematics will produce an optimum network-level strategy. Changes must therefore be made in the treatment selection procedure as funding levels ebb.

The greatest strides in network-level prioritization can be made by determining more accurate estimates of the lives and costs of pavements and pavement maintenance treatments in localized situations (*9*). This entails a commitment to keeping records. Eventually good record keeping will allow efficient pavement maintenance strategies to be developed through better performance prediction, life cycle cost analysis, and optimization techniques. When funding is constrained, computer-modeled scenarios will offer a practical means of assessing the future impacts of alternate strategies, establishing priorities, and convincing local decision makers to support adequate funding levels before roadway investments are compromised.

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Compilation of First Hungarian Network-Level Pavement Management System

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The first Hungarian network-level pavement management system relies on Markov transition probability matrices. A combined condition parameter is applied taking into consideration the bearing capacity, the unevenness, and the surface quality scores. The matrix variables are pavement type, traffic volume, and intervention variants. The system can be used to calculate the funds needed for highways at various condition levels, for the regional distribution of given amounts of money at a minimum cost to the national economy, and for the determination of the economic and technical consequences of subsequent modifications in funds distribution. Several trial runs have proved the practicability of the system.

All over the world the financial means available for highways lag more and more behind the actual needs. Although a growing share of these funds is used for maintenance and preservation tasks, the financial means even for these tasks decrease continually in several countries; among others, this has been the case in Hungary. This naturally influences the actual national roads policy, and the aim can only be at slowing down the general deterioration.

That is why the optimal distribution and allocation of rather limited financial means have become even more important than before. In recent years a significant development in the actual method of the allocation of highway funds between counties (highway directorates) has been seen. The former simple procedure that relied on normative values that were functions of road length, pavement type, and to a small extent traffic size were gradually substituted by methods that used information about the actual pavement condition. Recently, the need has emerged to transform this funds allocation so that it is as objective and reliable as possible. The objective preconditions of this development are the availability of the necessary data (insufficient quantity and accuracy) and of the appropriate computer technical background necessary for data retrieval and processing and the elaboration of an allocation model.

Thus, the development of the first Hungarian network-level pavement management system (PMS) has been performed in light of the circumstances described.

SCOPE OF TASK AND NETWORK-LEVEL AND PROJECT-LEVEL PMSs

The research presented here is aimed at the compilation of a mathematical model of the first Hungarian network-level PMS (*1*), more precisely, the establishment of the first version of the Hungarian network-level PMS.

The model relies on, for example, the available highway network, traffic, pavement structure, and cost information. The main technical-economic factors influencing the model were realistically considered, and at the same time, the limitations of available information were also taken into account. Compared with the preceding methods, the establishment of an optimization system can be considered a significant step forward, although several areas that need to be developed in the future can already be pointed out.

The model has the following main functions: determination of the necessary amount of funds required to ensure a given pavement condition in the future and a reliable regional allocation with certain limitations of the available financial means. When establishing the model the eventual controversial requirements of a high level of scientific accuracy and easy practicability were also considered and had a direct effect on the complexity and the size of the mathematical model and on the actual approximations applied.

In 1988 an expert team investigated the preconditions for the development of the first Hungarian PMS. It was concluded that to work out the first working version of the project-level PMS in Hungary several years of intensive research activities were still needed, whereas the elaboration of the network-level PMS appeared to be realistic in 1 to 2 years.

The network-level system should be established before the project-level one not only because the distribution of the financial means between various regions (counties) precedes even logically the optimal ranking of actual condition-improving interventions from a technical-economic point of view—that is, the elaboration of a project-level PMS—but also because

- Deficiencies and eventual limited reliability of the existing relevant data do not hinder the elaboration of the network-level PMS as much as they do the project-level PMS because in the former case mean values and only partly homogeneous data sets can be used,
- For the creation of the network-level PMS there is an existing method that can serve as a starting point for the new system, and
- The first version of the project-level PMS can be properly operated only if major organizational changes relating to several institutions (e.g., highway directorates and design and construction firms) are made, whereas this time-consuming and complex series of measures is not needed for the compilation of the network-level variant.

Thus, the Hungarian Institute for Transport Sciences (KTI) elaborated the mathematical model for the first Hungarian network-level PMS in 1989 and 1990. The system that was developed deals only with the maintenance-operation funds (*2*). The computer and mathematical aspects are presented elsewhere (*1*).

SELECTION OF MODEL

Preliminary Investigations

Development of the network-level model requires various preliminary investigations.

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The decision about the pavement types to be applied in the system was considered one of the first tasks. For this purpose the distribution of the national highway network area was investigated according to pavement type.

For the model to be established three main intervention alternatives (routine maintenance, surface dressing, and asphalt overlay) were chosen. Note that the maximal number of parameters applied was strongly limited by the need for the matrix on which the model relies to be manageable.

Among the traffic parameters, average annual daily traffic (AADT) and daily number of 100-kN axle loads (N) were taken into consideration. Although the actual heavy axle load has a direct connection to the loss of bearing capacity of the pavement structure, the parameter AADT was preferred partly because it is more widespread than the other parameter and partly because some other pavement deterioration forms—not only the loss of bearing capacity—should be considered in this complex investigation.

Selected Methodology

For the solution of the task outlined here, a methodology that took into account both the possibilities and the constraints had to be chosen.

The following existing possibilities for use in the establishment of the model are outlined:

- Available road data mass,
- Former fund distribution possibilities,
- Knowledge of similar foreign systems,
- Set of mathematical means for the treatment of this problem, and
- Goal-oriented expert team, including experienced highway engineers and mathematicians.

The following were objective difficulties and constraints in the establishment of the model:

- Limited time available for the elaboration of the model,
- Only a part of the huge mass of information on the countrywide sufficiency rating initiated in 1979 was available for data processing before the compilation of the model,
- Some available data are not sufficiently accurate because, for example, the existing data base does not contain the consequences of the recent changes in the numbers of kilometers of highways,
- Use of the time series is disadvantageously influenced because the condition parameters were often evaluated at various time points by different methods, and in some cases, the results of these procedures have no correlation between each other, and
- No domestic relationships are available between pavement condition and vehicle operating costs that would definitely help in the determination of well-founded optimum criteria.

Taking into account all of these aspects, the one that uses Markov-type transition probability matrices was chosen among several methods published in the literature. One reason was that it is clear and it does not need a longer time series as a precondition. For practicability, only limited numbers of condition variations, pavement types, traffic sizes, and intervention variants were taken for the establishment of the model.

MARKOV-TYPE TRANSISTION PROBABILITY MATRIX

The Markov-type transition probability matrix—in the case of a certain pavement type, traffic volume, and intervention strategy—supplies in the model the distribution of the probabilities of a given condition variant transitioning to another condition variant or of its remaining in the same condition variant during a certain period (e.g., 1 year).

Matrix Variables

The following are the matrix variables: pavement type, traffic size, and intervention variant.

Asphalt concrete and asphalt macadam pavements were chosen as pavement types. (In the first group all pavements were of the rolled asphalt type, whereas in the second group coated chippings and mixed and penetration asphalt macadams were included.) The rest of the highway network—rigid and unpaved sections of very limited lengths—has deterioration characteristics different from those of the selected flexible pavement groups.

For the characterization of low, medium, and high traffic, the following classes were chosen for the present study: 0 to 3,000 pcu/day, 3,001 to 8,000 pcu/day, and more than 8,001 pcu/day, respectively. The following three intervention variants were preferred: routine maintenance, surface dressing, and asphalt overlay. (Note that several foreign PMSs also apply the "do-nothing" variant. It was decided, however, to apply only the "routine maintenance" variant to the Hungarian System, even in the case of the slightest-intervention variant, when the necessary routine maintenance activities must be performed after the initiation of the first cracks and potholes. The possibility that the pavement would be left alone without any maintenance was unacceptable.

Taking into account the aforementioned facts, theoretically $2 \times 3 \times 3 = 18$ matrices should be made; two of them (surface dressing above 8,000 pcu/day for both pavement types), however, were excluded for technological reasons. So, the aim was the elaboration of 16 matrices.

Determination of Condition Variants in Matrix

The rows and the columns of Markov transition probability matrices are formed by pavement condition variants. On one hand the condition scores supplied by the countrywide highway suitability surveys are used for this calculation, for which sufficient data were available, with eventual time series, on the other hand, being considered of basic importance from the viewpoint of the deterioration process. The following pavement condition parameters were therefore selected:

- Pavement structure-bearing capacity score,
- Longitudinal unevenness (roughness) score, and
- Pavement surface quality score.

The ranking of these condition parameters into five quality classes appeared to provide sufficient accuracy for the intervention decisions.

For the longitudinal unevenness the following quality levels were applied:

With a Bump Integrator	*By Visual Evaluation*
1. good; maximum, 150 cm/km	1. good
2. sufficient; 151–225 cm/km	
3. fair; 226–275 cm/km	3. fair
4. insufficient; 276–350 cm/km	
5. unbearable; minimum 351 cm/km	5. poor

For the sake of the uniformity, a three-grade evaluation was selected for the longitudinal unevenness of the pavement. So, theoretically, $5 \times 5 \times 3 = 75$ condition variants would be available. To solve the problem mathematically, however, the number of condition variants had to be reduced.

The relatively rare condition variants (maximum 10 km in the whole network) were not considered separately but were united with the similar (sufficiently widespread) condition variants. Following this procedure, the 41 condition variants presented in Table 1 were considered in the model.

Calculation of Matrix Elements

Each element of the matrix—that is, the decimal probability of the transition of a certain condition variant to another one in 1 year in case of a given pavement type, traffic volume, and intervention strategy—was calculated on the basis of the results obtained by processing actual domestic data or, when they were lacking, by interpolation.

The available highway network and pavement structural data set was processed by the following method. First, the changing condition variants in 1984 and 1989 were determined for some 2,500 road sections of various lengths on which no overlay or surface dressing was applied during the investigation period. This process, which took into consideration the pavement and traffic categories mentioned, supplied the distribution of condition variants (in percent) after 5 years. For example, in the case of asphalt concrete pave-

TABLE 1 Condition Variant Groups of Markov-Type Transition Matrices

Number	Condition variants
1.	111
2.	112
3.	113+114+115
4.	131+132+151
5.	133+152
6.	134+135
7.	153+154+155
8.	211
9.	212
10.	213
11.	231+251
12.	232+252
13.	233+214
14.	234+215+235+253+254+255
15.	311
16.	312+331
17.	313+314
18.	332+351
19.	333+352+353
20.	334+315+335
21.	355+354
22.	411

(continued on next page)

TABLE 1 *(continued)*

Number	Condition variants
23.	412
24.	423+414
25.	432+431
26.	433
27.	434+415+435
28.	452+451
29.	453
30.	454
31.	455
32.	511
33.	512
34.	513
35.	514+515
36.	532+531+551
37.	533+552
38.	534+535
39.	553
40.	554
41.	555

Legend:

135 condition variant of a pavement with bearing capacity score 1 + pavement unevenness score 3 + surface quality score 5

ments with AADTs of 3,000 to 8,000 pcu/day and with condition variant 111, 89 percent remained in the same category, 6 percent deteriorated to Category 112, and 5 percent deteriorated to Category 211 after 5 years. These changes in percentages were divided by 5 to relate them to 1 year. The calculation was made for each variant if a minimum of 5 km of total length was available. After dividing by 100 and rounding off, these percentage distributions became the matrix elements. When no actual data were available interpolation (or sometimes extrapolation) was performed. In cases of applications of surface dressings and asphalt overlays the condition scores before and after the intervention were compared to obtain information about the typical change in condition.

A row-vector situated under the matrix is connected with it. Every element of this vector indicates the unit cost for 1 m^2 for the given intervention type performed on the section that has a condition variant specified above the appropriate column of the matrix. This unit cost is identical for a given variant in the whole country.

Interpretation of Matrix

Any of the 16 matrices has a size of 41 × 41. According to the condition variants presented in Figure 1, using their numbering, the matrix has the following structure. (The horizontal axis indicates the condition variants in the first year—the basic situation—whereas the vertical axis indicates the expected condition variant distribution in the second year.) For example, the symbol a_{23} in the matrix means the probability of the transition of a pavement with Condi-

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1	.60																				
2		.55																			
3		.10	.45																		
4				.70																	
5					.65																
6						.70															
7							.60														
8	.40							.75													
9		.35							.60												
10			.20							.55											
11				.20							.80										
12				.10							.10	.75									
13			.35		.35								.55								
14						.30	.40							.80							
15								.25							.50						
16									.40		.10					.75					
17										.45							.55				
18																		.70			
19												.25	.45						.65		
20														.20						.90	
21																					.85
22															.50						
23																					
24																	.45				
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Unit cost, HUF/m^2	.0	3,0	6,0	2,0	3,0	8,0	8,0	1,0	3,5	5,0	1,5	3,0	6,0	7,5	2,0	2,5	4,6	2,6	9,5	12,0	9,5

	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41
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27					.005	.90														
28							.90													
29				.005			.10	.85												
30									.70											
31										.75										
32	.20										.80									
33	.15	.50									.20	.90								
34			.20									.10	.90							
35													.10	.90						
36				.15										.10	.90					
37															.10	.65				
38						.10											.90			
39								.15										.90		
40									.30							.35	.10	.10	.85	
41										.25									.15	1.0
Unit cost, HUF/m^2	1,5	3,0	6,0	2,5	5,0	12,0	2,6	4,5	8,0	15,0	2,0	3,0	5,5	13,5	2,5	4,0	13,0	6,0	11,5	15,0

FIGURE 1 Markov-type transition probability matrices of asphalt concrete pavements with minimum traffic volume of 8,001 pcu/day for routine maintenance only.

tion variant 2 to Condition variant 3 in the next year (at the pavement type, the traffic volume, and the intervention strategy of the matrix in question). If the probability is 20 percent the value for a_{23} is 0.20.

The sum of each column in Figure 1 is 1.0 because 100 percent of the sections characterized by any condition variant will be in one of the condition variants in the next year.

After only routine maintenance the matrix elements can be found along or under the diagonal, according to the general deterioration. At the same time, surface dressing, especially asphalt overlay, pushes the elements that are different from 0 into the upper part of the matrix as a consequence of the general improvement in condition. (Some elements can be found along the diagonal for this case as well.)

Figure 1 presents, as an example, a matrix used for asphalt concrete pavements.

Unit Costs of Interventions

The unit costs of various interventions (routine maintenance only, surface dressing, asphalt overlay), which can be found as row vectors under the matrices, constitute an important part of the model.

Two questions arise in relation to this problem: What is the actual series value of intervention costs belonging to a given matrix and its 41 starting condition variants? Should costs unified countrywide or regional costs be used?

For the first question, it is evident that the interventions needed to repair the various condition variants have different costs:

- In the case of routine maintenance activities (it includes fixing potholes, crack filling, fine patching, and repairs on a large area) it can gradually increase from 0 to 10 t/km,
- In the case of surface dressing a double dressing is considered if the original condition is relatively poor (between condition variants of about 25 and 41), and
- Thickness of new overlay increases from a minimum of 3 cm up to 10 cm together with the condition deterioration.

Three main reasons explained the decision to apply unified average unit costs countrywide:

- Regional unit costs are influenced by several parameters that should not be included in the system,
- Realistic regional unit costs would be disadvantageous in the funds distribution for the highway directorates that maintain their networks economically, and
- Use of several unit costs would complicate considerably the "funds needed determination" part of the model.

APPLICATION AREAS

The network-level PMS model can be used primarily for the solution of three tasks

- Determination of the necessary funds needed to ensure a given condition level at a certain optimum criteria,
- Regional and functional distribution of a certain amount of money under constraints and given optimum criteria, and
- Evaluation of the technical and economic effects of subsequent funds distribution modifications.

DETERMINATION OF NECESSARY FUNDS

Basic Principles

The maintenance funds needed can also be determined with the help of 16 Markov-type transition probability matrices and the connected unit costs of the intervention.

Evidently, the actual funds needed relate to a desired condition level. In practice it usually means one of the following:

- Shares of some "good" condition variants are minimized,
- Shares of some "poor" condition variants are maximized,
- Former condition distribution is also required in the future, or
- Various constraints are selected for certain pavement types and traffic alternatives.

In general, the shares of condition variants can be maximized, minimized, fixed, or not regulated at all.

Some Trial Runs

The practicabilities of the principles mentioned were investigated by performing several trial runs and evaluating their results.

In a trial run the following constraints were assumed as future conditions: the areas of sections of poor condition variants—6, 14, 20, 21, 27–41 (Table 1)—should not decrease after the intervention. The areas of the rest of the condition variants were not limited at all. Because of the relatively few condition constraints the total funds needed were only 2.08×10^9 Hungarian forints (HUFt), of which 610 million HUFt was for routine maintenance, 646 million HUFt was for surface dressing, and 826 million HUFt was for asphalt overlay (US$1 = 100 HUFt). If the shares of areas undergoing an intervention are considered, the following results can be obtained: new asphalt overlay on 1.2 percent surface dressing on 11.0 percent, and routine maintenance (mainly patching) on 87.8 percent.

Another trial run was then performed when the influence of the increase in constraints on the funds needed and the actual funds needed was investigated. In this case, besides the constraints mentioned before (upper limitation of poor condition variants), it was also specified that the area of good condition variants—variants 1, 2, 4, and 8 according to Table 1—should not decrease.

The following area resulted: 111.7 million m^2 (73.0 percent) of routine maintenance, 8.2 million m^2 (5.4 percent) of surface dressing, and 32.8 million m^2 (21.5 percent) of asphalt overlay, (total of 152.7 million m^2).

The following cost resulted: 422 million HUFt (2.8 percent) for routine maintenance, 264 million HUFt (1.7 percent for surface dressing, and 14,410 million HUFt (95.5 percent) for asphalt overlay, (total of 15,096 million HUFt).

When evaluating these results it is striking that the attempt to preserve the sections in almost perfect condition required a high extra cost. The former 2,000 million HUFt increased here by 650 percent. It is interesting to observe that the share of asphalt overlay grew considerably. In the first version only 1.2 percent of the total area needed an overlay, whereas it grew to 21.6 percent after the increase in constraints.

Another trial run was carried out to investigate the shares of various intervention techniques (maintenance strategies) when a funds need determination for several years was performed by using the same overall condition requirements. Figure 2 shows that

- Funds needed each year increase slightly, with some fluctuation,
- Financial means used for routine maintenance gradually decrease during the 6-year period,
- More and more funds are destined for surface dressings, and
- Overlay needs grow rapidly until the fourth year, and then a little decrease can be observed.

FUNDS DISTRIBUTION

Basic Principles

In practice certain financial means are frequently divided for various purposes and regions. In this case the minimization of a value proportional to the vehicle operating costs is considered the objective function, whereas the traffic and pavement type constraints are also taken into account. It is correct to ask why the sum of the vehicle operating costs and the intervention costs—which is approximately the related national economic expenditures—is not maximized. At present this aim cannot be attained because information about the actual vehicle operating costs is not available and these absolute data would be needed to accomplish the summation with the absolute values of intervention costs. As long as only relative values connected with the vehicle operating costs (*3*) are used because of the lack of more accurate data, only the minimization of one of these parameters can be selected as an objective function. For this purpose the vehicle operating costs are chosen as being more significant on a national economic level.

Before the optimization the calculation already mentioned should be done according to which of the shares of necessary interventions on asphalt concrete and asphalt macadam pavements—separately for surface dressings and asphalt overlays—is given as a preliminary constraint.

The first step of the distribution of funds is the countrywide distribution of available financial means according to intervention categories, pavement types, condition variants, and traffic sizes.

After this optimization is done from the point of view of traffic operating costs, the regional distribution follows. This time no more weighting is needed; the distribution is made simply according to the area shares of sections with given characteristics (AADT, pavement type, condition variant) in various counties.

The selected objective function is the minimization of the following sum:

$$\sum_{i=1} A_i \cdot \mathrm{AADT}_i^a \cdot H_i, \qquad (1)$$

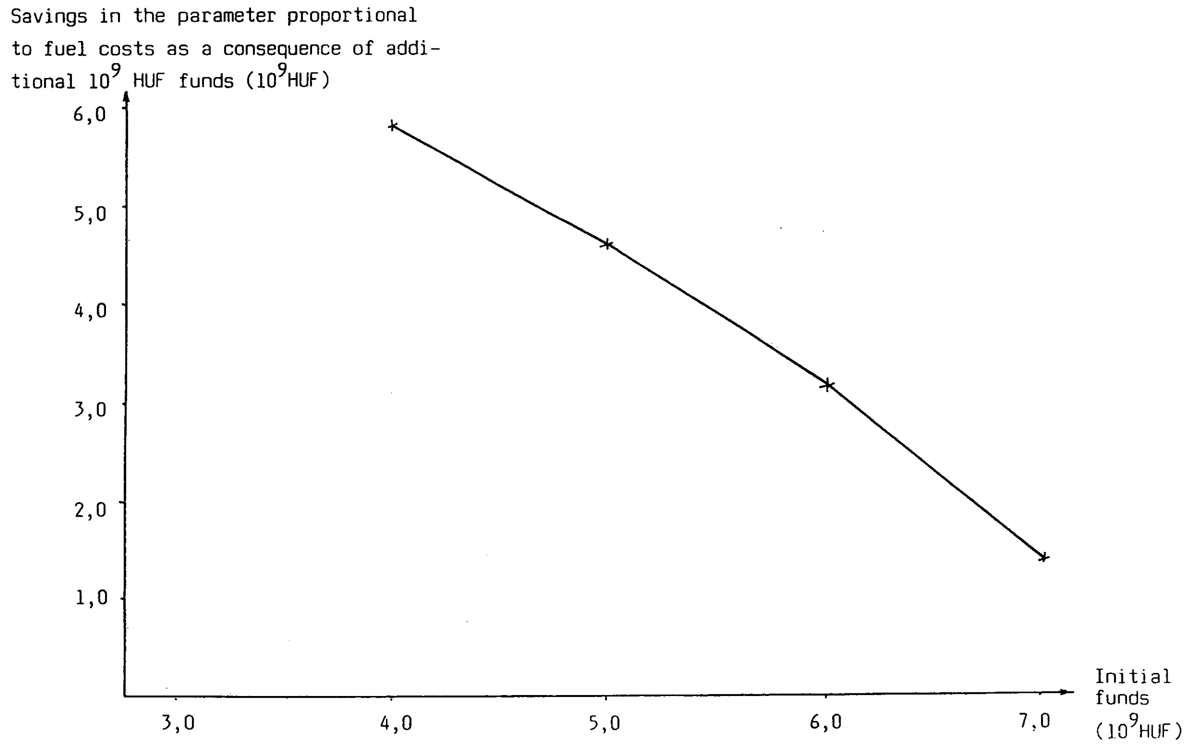

FIGURE 2 Expenditures for various intervention techniques required to ensure a given condition for 6 years.

where

A_i = specific vehicle operating cost parameter as a function of ith condition variant and the relative share of the heavy traffic (Figure 3).

$AADT_i^a$ = AADT weighed by the road area of ith condition variant (pcu/day), and

H_i = total length of sections in ith condition variant.

This sum of products can also be calculated before the intervention, so the effects of various condition-improving intervention strategies on vehicle operating costs can be evaluated (i.e., increase? decrease? to what extent?).

For Figure 3 the classification of 41 condition variants into the five groups shown in Figure 3 is needed. The three condition note variations were put into the following classes:

Pavement condition and type					In case of "n"			
					0,10	0,15	0,20	0,25
very good	good	fair	poor	very poor	fuel cost factor			
AB					1,00	1,00	1,00	1,00
AM	AB				1,05	1,04	1,04	1,04
	AM	AB			1,08	1,06	1,06	1,04
		AM	AB		1,21	1,19	1,16	1,14
			AM	AB	1,26	1,24	1,21	1,19
				AM	1,40	1,37	1,35	1,32

Legend: n - the ratio of the heavy (min 30 kN) axle load vehicles and all vehicles on the section

AB - asphalt concrete

AM - asphalt macadam

FIGURE 3 Extra fuel cost factors for roads with various pavement types and heavy traffic ratios.

- *Very good condition* if the sum of three condition notes is a maximum of 6,
- *Good condition* if the sum of three condition notes is between 7 and 9 and none of them is note 5,
- *Medium condition* if the sum of three condition notes is between 10 and 12 and none of them is note 5,
- *Poor condition* if only one of the condition notes is 5, and
- *Very poor condition* if two or three of the condition notes are 5.

The value n, the ratio of vehicles with heavy axle loads and all motor vehicles, is calculated by putting into the numerator the sum of the numbers of camions, trailers, buses, and heavy trucks.

Taking into account the aforementioned facts, the product $A_i \cdot \text{AADT}_i^a \cdot L_i$ is calculated for each condition-pavement type-traffic variant. These products are summarized for every variant to obtain the parameter K_K of the initial condition of the network, which is proportional to the vehicle operating costs.

The areas of the various condition-pavement type-traffic variants change after the distribution of the available funds because a slight percentage of the network receives an overlay and a higher share receives a surface dressing, whereas routine maintenance only is carried out on the majority of the total area.

For a new condition distribution, the parameter K_i, which is proportional to the actual vehicle operating costs, can be calculated by following the same principles. (The first element of the product is unchanged, the second one can be considered constant, whereas the third one usually changes. As a consequence the total sum of products will also be different.)

As a part of this computerized model the optimal variant with the lowest K_i value (*1*) can be determined by using linear programming techniques.

The K_i value of the optimal variant can exceed the former K_K value, proving that the available financial means are not sufficient for the preservation of the original condition level. If K_i is below K_K, then a more favorable situation than the former one can be attained.

Afterward the regional (county) funds allocation requires only a simple proportioning in which the funds for various condition-pavement type-traffic variants are divided among the counties according to the shares of the total area of their highway sections among the entire national area with given parameters.

Experiences from Some Trial Runs

Because the value of the funds (7.0×10^9 HUFt) assumed in the first trial run was greater than the presently realistic level, for the additional variants the funds assumed were gradually reduced; that is, 6×10^9 HUFt, then 4×10^9 HUFt, and finally, 3×10^9 HUFt were distributed.

The main direction of the investigation was to determine how the actual funds level influences the shares of three intervention types. Figure 4 indicates changes in the shares used for routine maintenance, surface dressing, and new asphalt overlay in the allocation according to this model as a function of available funds.

The following main results were obtained:

- In the case of the allocation of 3.0×10^9 HUFt, only one-third of the financial means was used for asphalt overlays; the highest share was spent for surface dressings;
- With an increase in funds, the financial means allocated to asphalt overlay grew considerably, whereas the shares for other two intervention types evidently decreased; and
- Among the areas of various intervention types, percent changes that were not as high could be observed since the unit costs of routine maintenance and surface dressing gradually decreased accordingly, because—together with the increase in total funds—asphalt overlay was applied on the worst sections that had earlier received only patching or surface dressing.

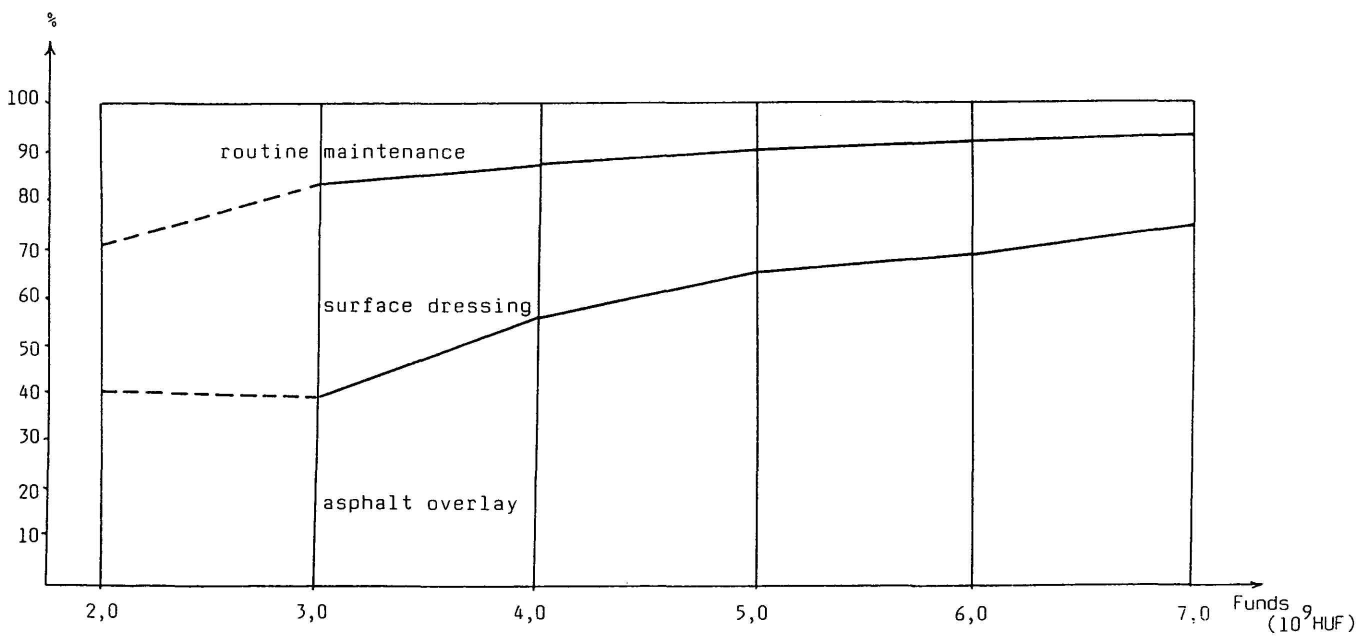

FIGURE 4 Relationship between funds and shares for various intervention types.

Figure 4 analyzes how the funds, which were increased by 1.0×10^9 HUFt, influenced the vehicle operating costs (or the parameter proportional to them). There was a definite tendency for the "savings" (reduced fuel costs) to be smaller and smaller as the total funds grew. This statement is not surprising because the extra funds allowed not only the very poor but also the less bad sections to be repaired. In the latter case a lower fuel cost reduction can evidently be attained by the interventions.

EVALUATION OF CONSEQUENCES OF SUBSEQUENT MODIFICATION IN FUNDS DISTRIBUTION

Frequently, (and presumably in the future as well) the optimum funds distribution is not implemented. The reasons, among others, can be consideration of local aspects, the need to concentrate financial means, and the necessity for an internal regrouping of money. It is appropriate to evaluate the technical and economic consequences of such modifications.

The technical consequence is the resulting condition distribution of the network concerned. It can easily be obtained by using the appropriate Markov transition probability matrices and forecasting the conditions in the following year according to the changed intervention spectrum.

The economic consequences can be evaluated by calculating total vehicle operating costs (or the parameter proportional to them). Determination of this sum of products, after the intervention alternative has been changed, makes it possible to estimate the losses in national economic costs as a result of the new decisions. (An improvement cannot be attained because the optimum variant was originally developed.)

CONCLUSIONS

The significance of the first Hungarian network-level PMS model can be summarized as follows:

- Determination and the distribution of maintenance-operation funds were carried out by considering several influencing factors,
- At the optimization, data on roadways not only in poor condition but also those of all conditions were taken into account,
- Distribution of funds is done by excluding the local subjective parameters,
- As a last step some other aspects can be applied in the system, and
- The system can readily be developed further.

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Optimal Programming by Genetic Algorithms for Pavement Management

T. F. FWA, W. T. CHAN, AND C. Y. TAN

Optimal programming of pavement activities is a desired element in pavement management systems. The complexity and scale of the problem, however, have prevented the widespread use of such analytical tools in practice. An application of a relatively new optimization technique, known as the *genetic algorithms,* to the pavement programming problem is described. This technique does not require any information on the differentiability, convexity, or other auxiliary properties of the problem parameters. On the basis of the mechanics of natural selection it has a robust search algorithm that makes it an attractive technique for pavement programming at the network level. Through an example application to a network-level pavement programming problem, the considerations involved in genetic representation of the problem and generation of new solutions (known as *offspring*) are presented. The convergence characteristics of the genetic algorithm solutions are also analyzed. Finally, the applicability of the technique to the general pavement management problem is discussed.

A primary objective of pavement management at the network level is to program pavement investments and schedule pavement activities to achieve optimal results of pavement network performance. Many optimization techniques have been developed since the mid-1970s to provide the necessary analytical tools to assist highway agencies in making such management decisions. These techniques include dynamic programming (*1*), optimal control theory (*2*), integer programming (*3*), linear programming (*4*), and nonlinear programming and heuristic methods (*5*). Because of the complexity of the pavement programming problem at the network level, different techniques are suitable under different circumstances.

This paper illustrates the application of a general purpose problem-solving and optimization technique, known as the *genetic algorithms,* (GAs) to the pavement management problem. The genetic algorithms are formulated loosely on the basis of the principles of Darwinian evolution (*6,7*). The general operating principles of genetic algorithms are presented first in this paper; this is followed by an application example that solves a network-level pavement programming problem.

OPERATING PRINCIPLES OF GAs

Theoretical Basis of GAs

GAs are robust search techniques formulated on the basis of the mechanics of natural selection and natural genetics. It was in the 1980s that genetic algorithm applications started to spread across a broad range of disciplines, including function optimizers (*8*), pattern recognition (*9,10*), computer-aided operation control (*11*), and robot kinetics (*12*).

GAs are different from traditional optimization techniques in a few important aspects. First, GAs work with a coding of the parameters instead of the parameters themselves. The choice of the parameter representation is important because GAs work on a coded version of the problem to be solved and not on the problem directly. Second, GAs operate by manipulating a pool of feasible solutions instead of one single solution in the search of good solutions. Working with a pool of solutions enables GAs to identify and explore properties that good solutions have in common. Third, GAs employ probabilistic transition rules to move from one pool of solutions to another. This introduces perturbations to move out of local optima. Last, GAs rely only on objective function evaluations. They do not require any information on differentiability, convexity, or other auxiliary properties. GAs are thus easy to use and implement for a wide variety of optimization problems.

Mechanics of GA Solution Process

For a given problem with a specified objective function, the problem-solving process of GAs begins with the identification of problem parameters and the genetic representation (i.e., coding) of these parameters. The search process of GAs for solutions that best satisfy the objective function involves generating an initial random pool of feasible solutions to form a parent solution pool and obtaining new solutions and forming new parent pools through an iterative process of copying, exchanging, and modifying parts of the genetic representations in a fashion similar to that of natural genetic evolution.

Each solution in the parent pool is evaluated by means of the objective function. The fitness value of each solution, as given by its objective function value, is used to determine its probable contribution in the generation of new solutions, known as *offspring*. The next parent pool is then formed by selecting the fittest offspring on the basis of their fitnesses (i.e., their objective function values). The entire process is repeated until a predetermined stopping criterion is reached on the basis of either the number of iterations or the magnitude of improvement in the solutions. Figure 1 presents a flow chart that summarizes this solution process.

EXAMPLE PROBLEM

Problem Description

The application of GAs to pavement management is illustrated in this paper by solving an integer-programming optimization problem

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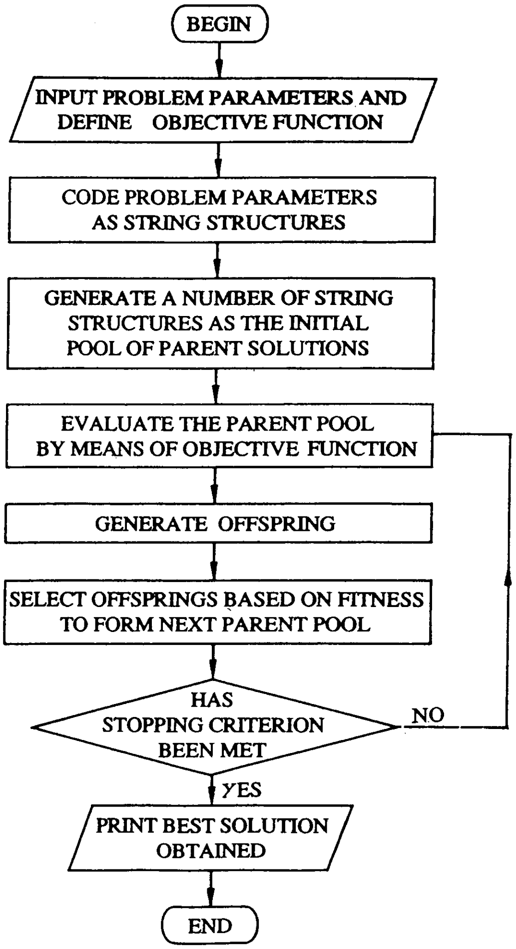

FIGURE 1 Solution process of GAs.

analyzed by Fwa et al. (*3*). On the basis of the framework of pavement management practice in Indiana, Fwa et al. solved for an optimal pavement repair program at the network level for a given rehabilitation schedule and subject to six forms of resources and operation constraints. Mathematically, the problem can be expressed as follows:

$$\text{Maximize} \sum_{i=1}^{N_1} \sum_{j=1}^{N_2} \sum_{k=1}^{N_3} W_{ijk} F_{ijk} \tag{1}$$

with W_{ijk} as integers for $i = 1, 2, \ldots N_1, j = 1, 2, \ldots N_2, k = 1, 2, \ldots N_3$, subject to the following constraints:

1. Production requirements:

$$0 \leq W_{ijk} \leq \frac{T_{ijk}\gamma_{ijk}}{U_{ijk}} \qquad i = 1, 2, \ldots N_1, j = 1, 2, \ldots N_2, \quad k = 1, 2, \ldots N_3, \tag{2}$$

2. Budget constraint

$$\sum_{i=1}^{N_1} \sum_{j=1}^{N_2} \sum_{k=1}^{N_3} W_{ijk} U_{ijk} C_{ijk} \leq B \tag{3a}$$

$$\sum_{i=1}^{N_1} \sum_{k=1}^{N_3} W_{ijk} U_{ijk} C_{ijk} \leq b_j \qquad j = 1, 2, \ldots N_2 \tag{3b}$$

3. Manpower availability

$$\sum_{i=1}^{N_1} \sum_{j=1}^{N_2} \sum_{k=1}^{N_3} W_{ijk} h_{jg} \leq H_g \qquad g = 1, 2, \ldots, G \tag{4}$$

4. Equipment availability

$$\sum_{i=1}^{N_1} \sum_{j=1}^{N_2} \sum_{k=1}^{N_3} W_{ijk} q_{jr} \leq Q_r \qquad r = 1, 2, \ldots, R \tag{5}$$

5. Material availability

$$\sum_{i=1}^{N_1} \sum_{j=1}^{N_2} \sum_{k=1}^{N_3} W_{ijk} m_{js} \leq M_s \qquad s = 1, 2, \ldots, S \tag{6}$$

6. Rehabilitation constraints

$$\gamma_{ijk} = \frac{D - d_{ijk}}{D} \qquad i = 1, 2, \ldots N_1, j = 1, 2, \ldots N_2, \quad k = 1, 2, \ldots N_3, \tag{7}$$

where all variables are defined in Table 1. The practical meaning and rationale for each of the above equations are found in the technical paper by Fwa et al. (*3*). Fwa et al. solved this problem by the integer programming technique with the branch and bound algorithm of the multipurpose optimization scheme (MPOS) (*13*).

Input Data

The problem solved by Fwa et al. (*3*) considered four pavement defects and three levels of maintenance-need urgency. Table 2(*a*) gives the production rate and unit cost data for each combination of pavement defect and urgency level. The requirements for four manpower types and six equipment types are listed in Table 2(*b*). Recorded in Table 2(*c*) are the pavement repair priority weighting factors, which are functions of highway class, pavement repair activity type, and its need-urgency level.

The estimates of the amount of work required for each type of repair in terms of workdays are found in Table 3(*a*). The input for rehabilitation constraint factors are given in Table 3(*b*). A zero value of γ_{ijk} represents a case in which there is a complete interference from rehabilitation work, whereas a γ_{ijk} value of unity indicates no interference from rehabilitation. Other necessary input information, namely, the analysis period, budget allocation, manpower availability, and equipment availability, are found in Table 3(*c*).

APPLICATION OF GAs TO EXAMPLE PROBLEM

Main Features of Problem

Two main features of the problem affect the choice of solution techniques. First, the decision variables W_{ijk} are integers that restrict the methods to those that can handle integer variables. The other feature of the problem is what is commonly known as the *combinatorial explosion* of the feasible solution space. In the current problem, there are four highway types (i = 4), four pavement repair activities (j = 4), and three need-urgency levels (k = 3). There are altogether 48 (4 × 4 × 3) decision variables (W_{ijk}). In the extreme case in which each decision variable is allowed to take up any integer value from 0 to 45 workdays, the total number of possible solutions is equal to 46^{48}, or 6.4×10^{79}. Even if one were to assume a case in

TABLE 1 Definitions of Variables in Equations 1–7

Variable	Definition
W_{ijk}	equivalent workload units in number of workdays of pavement repair activity j of need urgency level k performed on highway i
F_{ijk}	priority weighting factor for pavement repair activity j of need urgency level k on highway i
N_1	total number of highways considered
N_2	total number of pavement repair activities considered
N_3	total number of need urgency levels considered
T_{ijk}	total workload of pavement repair needs expressed in work measurement units (see Table 2) for pavement repair activity j of need urgency level k on highway i
γ_{ijk}	rehabilitation constraint factor for pavement repair activity j of need urgency level k, $0 \le \gamma_{ijk} \le 1$
U_{ijk}	work productivity for pavement repair activity j of need urgency level k on highway i
C_{ijk}	cost per production unit of pavement repair activity j of need urgency level k on highway i
B	total budget amount allocated for the analysis period considered
b_j	budgeted fund for pavement repair activity j
h_{jg}	number of mandays of work crew type g required for each unit of pavement repair activity j
H_g	total available number of mandays of work crew type g
G	total number of work crew type
q_{jr}	number of equipment days of equipment type r required for each production day of pavement repair activity j
Q_r	total available number of equipment days of equipment type r
R	total number of equipment types
m_{js}	quantity of material type s required for each production day of pavement repair activity j
M_s	total available quantity of material type s
S	total number of material types
d_{ijk}	number of working days before a scheduled rehabilitation during which no pavement repair activity j of need urgency level k would be performed on highway i
D	total number of working days in analysis period

which each decision variable could only take up a value from 0 to 5 workdays, the total number of possible solutions of 6^{48}, or 2.2×10^{37}, would still require a modern supercomputer many years to enumerate all possible solutions.

Genetic Representation of Problem

In GAs a solution to a problem is represented by a string structure similar to the chromosomes in natural evolution. As shown in Figure 2 the chromosomal representation of a solution is known as a *genotype,* which consists of a string of genes. The value of each gene is called its *allele.*

Each genotype is a solution in the structure of the solution space represented by the genetic representation chosen. For example, Figure 3 shows a genotype that represents a solution with the values of W_{ijk} indicated therein. Because each W_{ijk} can assume any integer value from 0 to 45, the representation is different from the traditional binary representation used in GA applications.

GA Operations

After the genetic representation is determined, an initial parent pool of solutions can be randomly generated. A pool of 80 solutions was selected for the present example. The following three GA operations were then executed in sequence repeatedly: (a) generation of offspring, (b) the formation of of a new parent pool, and (c) performance evaluation and convergence assessment of the parent pool solutions.

TABLE 2 Production and Resource Requirements Data

(a) Production rate and unit cost data

Need-Urgency Level k	Production Rate U_{ijk}				Unit Cost C_{ijk}			
	Shallow Patching j = 1 (kg mix per day)	Deep Patching j = 2 (kg mix per day)	Premix Leveling j = 3 (kg mix per day)	Crack Sealing j = 4 (km per day)	Shallow Patching j = 1 ($ per kg mix)	Deep Patching j = 2 ($ per kg mix)	Premix Leveling j = 3 ($ per kg mix)	Crack Sealing j = 4 ($ per km)
High (k=1)	6,537.6	17,978.4	10,896.0	10.1	0.0938	0.0852	0.0403	81.37
Medium (k=2)	3,813.6	9,443.2	80,448.8	13.5	0.1311	0.1333	0.0420	70.19
Low (k=3)	2,542.4	6,174.4	49,940.0	16.4	0.1751	0.1817	0.0467	63.98

(b) Manpower and equipment requirements

Repair Activity	Manpower Requirement h_{jg} (Man-days/Production Day)				Equipment Requirement q_{jr} (Equipment-days/Production Day)					
	g = 1	g = 2	g = 3	g = 4	r = 1	r = 2	r = 3	r = 4	r = 5	r = 6
j = 1	0	2	4	0	1	0	1	0	0	0
j = 2	1	1	5	1	1	1	0	0	0	1
j = 3	1	3	5	2	3	1	1	1	0	1
j = 4	1	2	2	4	2	1	0	1	0	0

Note: Manpower types 1 to 4 represent supervisors, drivers, laborers, and equipment operators, respectively; equipment types 1 to 6 represent dump trucks, pickup trucks, crew cabs, distributors, loaders, and rollers, respectively.

(c) Pavement Repair Priority Weighting Factors, F_{ijk}

Highway Class	Need-Urgency Level k	Shallow Patching j = 1	Deep Patching j = 2	Premix Leveling j = 3	Crack Sealing j = 4
Urban Interstate (g = 1)	k = 1 (High)	90	100	70	50
	k = 2 (Medium)	63	90	63	45
	k = 3 (Low)	54	60	42	30
Urban Arterial (g = 2)	k = 1 (High)	72	80	56	40
	k = 2 (Medium)	54	70	49	35
	k = 3 (Low)	45	50	35	25
Rural Interstate (g = 3)	k = 1 (High)	76.5	85	59.5	42.5
	k = 2 (Medium)	58.5	75	52.5	37.5
	k = 3 (Low)	40.5	45	31.5	22.5
Rural Primary (g = 4)	k = 1 (High)	70.5	65	45.5	32.5
	k = 2 (Medium)	36	40	28	20
	k = 3 (Low)	18	20	14	10

Generation of Offspring

The crossover operation and mutation operation are the two most commonly used GA operations for generating offspring from parent solutions. Figure 4 illustrates the mechanism of a simple crossover operation and that of a simple mutation operation. Figure 4(*a*) shows a simple crossover operation with a single cross point on two genotypes. A common point is randomly chosen, and the two parts of each genotype are swapped to create two offspring. Each offspring therefore consists of a part of each parent. In the case of a simple mutation operation on the binary genotype shown in Figure 4(*b*), a random number is generated for each allele and an allele is mutated if the random number generated is less than a predetermined number.

TABLE 3 Repair Needs Data and Constraint Information

(a) Repair work requirements in workdays, T_{ijk} / U_{ijk}

Highway Class	Need-Urgency Level k	Shallow Patching j = 1	Deep Patching j = 2	Premix Leveling j = 3	Crack Sealing j = 4
Urban Interstate (g = 1)	k = 1 (High)	4	6	8	2
	k = 2 (Medium)	6	4	2	3
	k = 3 (Low)	3	25	13	18
Urban Arterial (g = 2)	k = 1 (High)	2	6	9	2
	k = 2 (Medium)	2	10	8	8
	k = 3 (Low)	4	20	15	15
Rural Interstate (g = 3)	k = 1 (High)	5	8	6	5
	k = 2 (Medium)	5	2	10	10
	k = 3 (Low)	5	15	15	10
Rural Primary (g = 4)	k = 1 (High)	3	4	8	4
	k = 2 (Medium)	4	16	12	20
	k = 3 (Low)	15	15	18	15

(b) Rehabilitation constraint factors, γ_{ijk}

Highway Class	Need-Urgency Level k	Shallow Patching j = 1	Deep Patching j = 2	Premix Leveling j = 3	Crack Sealing j = 4
Urban Interstate (g = 1)	k = 1 (High)	0.82	0.83	1.00	0.80
	k = 2 (Medium)	0.70	0.90	0.90	1.00
	k = 3 (Low)	1.00	1.00	1.00	1.00
Urban Arterial (g = 2)	k = 1 (High)	0.93	1.00	1.00	0.92
	k = 2 (Medium)	0.84	1.00	1.00	0.96
	k = 3 (Low)	0.81	1.00	1.00	0.90
Rural Interstate (g = 3)	k = 1 (High)	0.92	1.00	1.00	0.83
	k = 2 (Medium)	0.78	1.00	1.00	0.91
	k = 3 (Low)	0.80	1.00	1.00	0.96
Rural Primary (g = 4)	k = 1 (High)	1.00	1.00	1.00	1.00
	k = 2 (Medium)	1.00	1.00	1.00	1.00
	k = 3 (Low)	1.00	1.00	1.00	1.00

(c) Resource constraints and other input information

Parameter	Value
Analysis Period	D = 45 working days
Budget Allocation	b_1 = \$18,000 b_2 = \$20,000 b_3 = 13,000 b_4 = 9,000
Manpower Availability	H_1 = 90 mandays H_2 = 135 mandays H_3 = 270 mandays H_4 = 90 mandays
Equipment Availability	Q_1 = 135 equipment days Q_2 = 45 equipment days Q_3 = 45 equipment days Q_4 = 45 equipment days Q_5 = 45 equipment days Q_6 = 45 equipment days

In the present example problem the simple crossover and mutation operations were found to be ineffective because they consistently led to more than 80 percent invalid offspring. This was because the problem was highly constrained. Various approaches have been used to handle constrained problems in GAs. One approach is to apply a penalty weightage to the objective function value of invalid offspring (*11*). Other approaches use what are known as *decoder* or *repair algorithms* to avoid creating invalid offspring (*14,15*).

The present study adopted an approach that made use of specialized GA operations to handle the constraints. These specialized operations were the single arithmetic crossover and

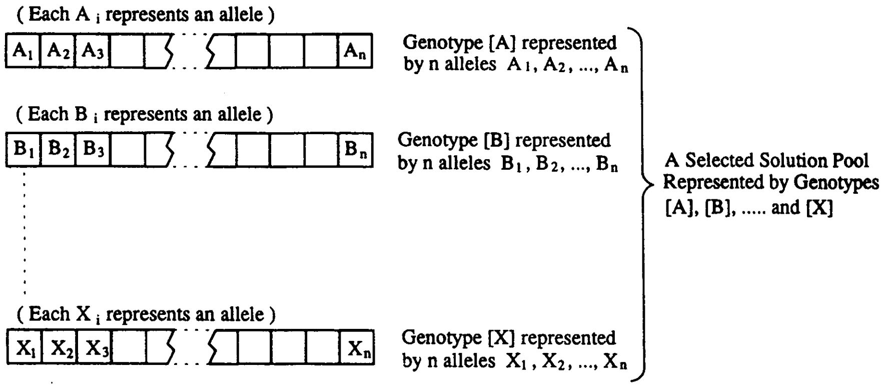

FIGURE 2 GA representation of knowledge.

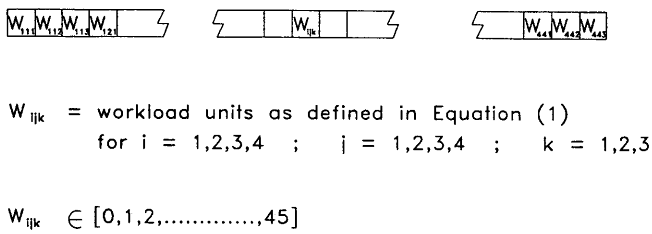

FIGURE 3 Integer coding for genetic representation of example problem.

the nonuniform mutation (*14*). The relative probabilities of applying the crossover and mutation operations were selected as 0.8 and 0.2, respectively. These two operations function as follows. When a single arithmetic crossover is operated on two parent genotypes $<X_1, X_2, \ldots, X_n>$ and $<Y_1, Y_2, \ldots, Y_n>$, the resulting offspring are $<X_1, X_2, \ldots, X_k', \ldots, X_n>$ and $<Y_1, Y_2, \ldots, Y_k', \ldots, Y_n>$, where $k \in [1, n]$, $X_k' = q\, Y_k + (1 - q)\, X_k$, $Y_k' = q\, X_k + (1 - q)\, Y_k$, and q is a random decimal value between 0 and 1.

When a nonuniform mutation operation is executed on a parent genotype $<X_1, X_2, \ldots, X_n>$ and X_k is the gene to be mutated, the resulting offspring would be $<X_1, X_2, \ldots, X_k', \ldots, X_n>$, where X_k' has one of the following values: $(0.2\, q\, X_{max})$ when $X_k = X_{max}$, $(1 - 0.2q)\, X_{max}$ when $X_k = 0$, or $(X_k + pqz)$ when $0 < X_k < X_{max}$. X_{max} is the maximum permissible value of X_k, q is a random decimal value between 0 and 1, p is either $+1$ or -1 with equal probability decided randomly by the program, and z is minimum $(X_k, X_{max} - X_k)$.

Formation of New Parent Pool

In the offspring generation phase 160 offspring were generated from the 80 solutions in the parent pool. The next step was to select 80 solutions from the offspring pool to form the next parent pool. Each offspring was first evaluated by means of the objective function to arrive at the so-called fitness value of the offspring. The top 80 genotypes in terms of fitness were then selected to form the new parent pool. This process ensures that the GA search is always directed toward solutions that return better values of fitness (i.e., the values of objective function) as the solution process proceeds.

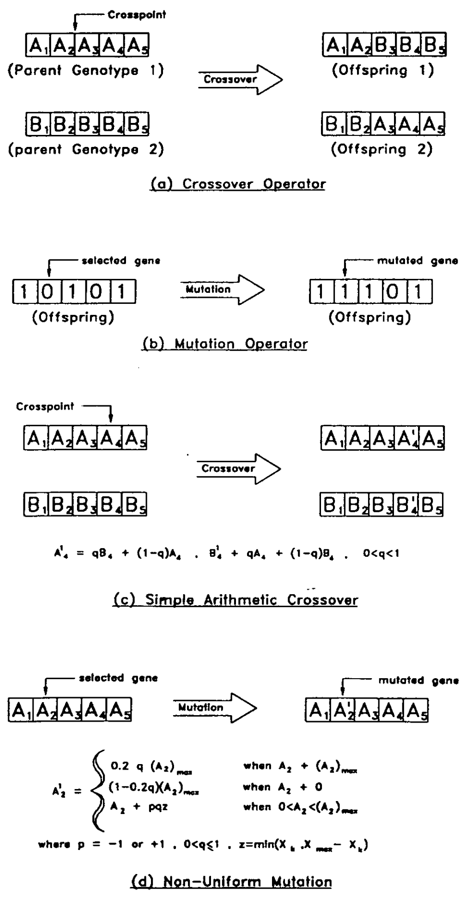

FIGURE 4 Examples of genetic operations.

Performance Evaluation of Solutions

There are four possible measures for assessing the performance of GA solutions. The first measure, known as the *online performance,* is the running average of the value of the objective function of all valid genotypes that have been generated. The second measure, known as the *offline parent-pool performance,* records the average of the objective function values of the genotypes of each parent pool selected. The third measure, known as the *offline offspring-pool performance,* records the average of the objective function values of the genotypes of each pool of offsprings generated. Finally, the *best solution* is the value of the objective function of the best genotype that has been generated. An example is given in Figure 5, which plots the four performance measures against the number of generations on the basis of the results of one of the GA solution runs obtained in the study. The best solution criterion is typically used to compare the performance of GAs, whereas the online performance and the offline performance are often used to monitor the convergence of GA solutions.

Convergence of GA Solutions

The typical trend of convergence of GA solutions is clearly displayed in Figure 5. When the curves of best solution and offline parent-pool performance are considered, it can seen that the GA solutions converged after the 28th generation, although the best solution was achieved at the 23rd generation. The average of parent-pool solutions lagged the best solution in terms of convergence, as expected. The fluctuations of the offline offspring-pool performance curve, after convergence had been achieved by the best-solution and the offline parent-pool performance curves, indicate the on-going GA mechanism of search for possible improvements. In comparison with these three performance measures, the online performance does not appear to provide as good an indication of the trend of convergence.

Figure 6 shows the best solution performance curves for four solution runs of the GA program for the example problem. The number of generations at which the best solution was reached varied from the 14th generation in Solution Run 1 to the 26th generation in Solution Run 3. The offline parent-pool performance curves in Figure 7(a) indicate that convergence of the parent-pool solutions occurred at the 19th generation for Solution Run 1 and at the 30th generation for Solution Runs 3 and 4. Figure 7(b) plots the offline offspring-pool performance for the four GA runs. All four curves exhibit the postconvergence fluctuations typical of the response of offspring-pool solutions described in the preceding paragraph.

COMPARISON OF INTEGER PROGRAMMING AND GAs

GA Solutions Versus Integer-Programming Solution

Table 4 presents the GA solutions obtained in the study together with the integer-programming solution produced by Fwa et al. (*3*). Table 4(a) shows that all the four GA runs could produce good solutions with objective function values comparable to those achieved by the integer programming solution. The improvements over the integer programming solution were 0.57, 0.21, 4.93, and 3.95 percent for GA Solution Runs 1, 2, 3, and 4, respectively. Table 4(b) shows the output values of decision variables W_{ijk} for all five solutions. For easy comparison Table 5 summarizes the results by highway class and pavement repair activity type in terms of the decision variables W_{ijk}. These answers in W_{ijk} can be converted into workload measurement units by multiplying the production rates given in Table 2(a).

It is apparent from the results in Tables 4 and 5 that the choices of pavement repair activities were different among the five solutions, although the concentration of activities in all of the solutions

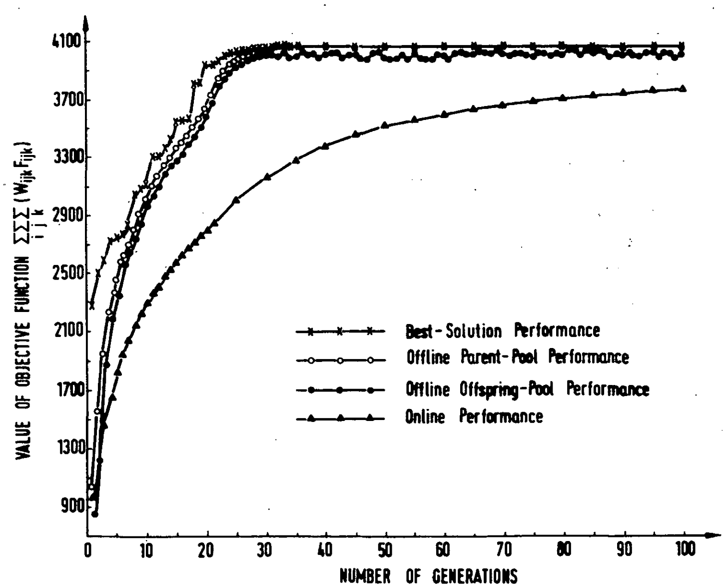

FIGURE 5 Performance evaluation of GA solutions.

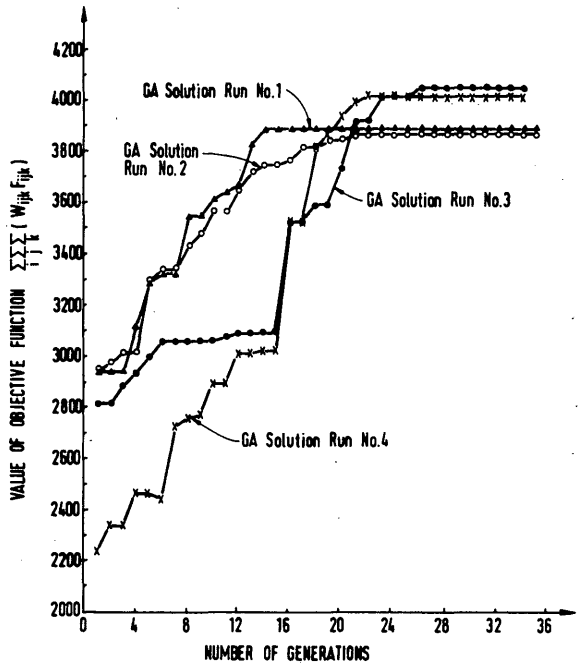

FIGURE 6 Best-solution performance curves for four GA solution runs.

was heavily influenced by the priority weighting factors [see Table 2(*c*)]. This is clearly depicted in Figure 8, in which the results of Table 5 are presented graphically. In general, as shown in Table 2(*c*), urban Interstate ($i = 1$) received the highest priority, whereas rural primary received the lowest; shallow and deep patching ($j = 1$ and $j = 2$, respectively) also had higher priorities than the other two repair activities. These priority patterns are generally reflected in all five solutions. It should be noted, however, that the need to maximize the objective function had the ultimate impact on the solutions. For example, on the whole more workdays of shallow patching ($j = 1$) than deep patching ($j = 2$) were assigned, even though the latter had higher priority.

Comments on GA Application to Pavement Management Problems

GAs have the inherent property of being able to process a large number of similar string structures. Holland (*6*) gave the name of *intrinsic parallelism* to this property, which is related to the notion of schema. A schema can be defined as a pattern-matching device or a similarity template that describes a subset of strings with similarities at certain string positions. For example, a binary code schema 111*xx* describes a subset of four strings (11100, 11101, 11110, 11101). In each generation GA operations process approximately a total of n^3 schemata (*6*), where n is the size of the parent pool. This property gives GAs great computational leverage.

With the stochastic generation of offspring in their search for new solutions, GAs represent a global search process. However, they are not simply random procedures because the offspring are generated from a parent pool of solutions that have been selected on the basis of their fitness. GAs are therefore able to efficiently exploit past information to explore new regions of the decision space with a high probability of finding solutions with improved fitness.

The choice of genetic operators in offspring generation is an important phase in GAs that can be used to provide meaningful ways of combining genetic information from different genotypes in the parent pool. Care should also be taken that the selected genetic operations do not lead to the creation of excessive numbers of invalid offspring or contribute to premature convergence. For example, the crossover operation alone would not be sufficient because *lost alleles* cannot be recovered, hence leading to premature convergence (*16*). A lost allele occurs when the entire parent pool or all offspring solutions have the same value for a particular gene. This problem is overcome by the use of the mutation operation that helps to maintain the genetic diversity and keep the gene pool well-stocked. It is significant that the mutation operation allows the search to reach new areas of the parameter space.

It is observed from the example application (e.g. Table 4 and Figures 6 and 7) that there were differences among the solutions obtained from different GA runs because of the stochastic nature of the technique. This is a genetic phenomenon known as *genetic drift* (*11*). Genetic drift is common and expected in applications to combinatorial optimization problems, and the differences are usually small. It should be noted that it is not possible to perform an ex-

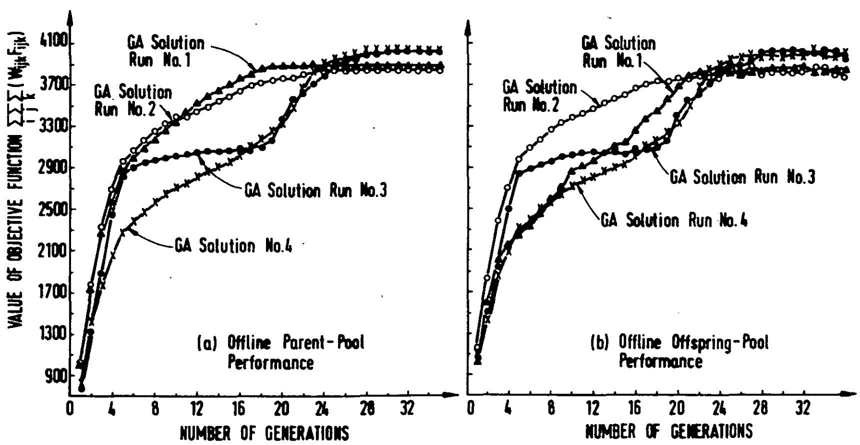

FIGURE 7 Performance curves for offline parent-pool solutions and offline offspring-pool solutions.

TABLE 4 Solutions of Example Problem by Integer Programming and GAs

(a) Values of Objective Function of Solutions

Method of Analysis	Integer Programming	GA Solution Run No. 1	GA Solution Run No. 2	GA Solution Run No. 3	GA Solution Run No. 4
Value of Objective Function	3865.5	3887.5	3873.6	4056.0	4018.0
Percent Improvement	--	0.57%	0.21%	4.93%	3.95%

(b) Values of Decision Variables W_{ijk}

W_{ijk}	IP	GA1	GA2	GA3	GA4	W_{ijk}	IP	GA1	GA2	GA3	GA4
W_{111}	3	2	2	2	3	W_{311}	4	3	4	4	4
W_{112}	4	3	4	4	3	W_{312}	3	2	3	3	3
W_{113}	3	2	3	3	3	W_{313}	4	3	4	4	4
W_{121}	4	4	4	4	4	W_{321}	0	0	3	0	0
W_{122}	3	2	2	3	3	W_{322}	2	2	2	2	2
W_{123}	0	0	0	0	0	W_{323}	0	0	0	0	0
W_{131}	0	1	0	6	2	W_{331}	0	5	5	0	5
W_{132}	1	1	1	0	0	W_{332}	0	0	0	0	0
W_{133}	4	1	3	4	4	W_{333}	0	0	0	0	0
W_{141}	1	1	1	0	0	W_{341}	4	3	3	4	4
W_{142}	3	2	3	3	3	W_{342}	1	1	1	0	3
W_{143}	0	0	0	0	0	W_{343}	0	0	0	0	0
W_{211}	1	1	1	0	0	W_{411}	3	2	2	3	3
W_{212}	1	1	1	0	0	W_{412}	4	3	3	4	4
W_{213}	3	2	3	3	3	W_{413}	1	1	1	0	0
W_{221}	0	0	0	0	0	W_{421}	0	0	0	0	0
W_{222}	6	5	3	6	6	W_{422}	0	0	0	0	0
W_{223}	0	2	0	0	0	W_{423}	0	0	0	0	0
W_{231}	0	7	0	0	0	W_{431}	0	0	0	0	0
W_{232}	0	0	0	0	0	W_{432}	0	0	0	0	0
W_{233}	0	0	0	0	0	W_{433}	0	0	0	0	0
W_{241}	1	1	1	1	0	W_{441}	0	0	0	2	0
W_{242}	0	2	1	0	0	W_{422}	0	0	0	0	0
W_{243}	0	0	0	0	0	W_{423}	0	0	0	0	0

Note: IP = integer-programming solution
GA1 = genetic-algorithm solution No. 1
GA2 = genetic-algorithm solution No. 2
GA3 = genetic-algorithm solution No. 3
GA4 = genetic-algorithm solution No. 4

TABLE 5 Summary of Workloads (W_{ijk}) by Highway Class and Repair Activity for Different Solutions

Highway Class	Shallow Patching (j = 1)	Deep Patching (j = 2)	Premix Leveling (j = 3)	Crack Sealing (j = 4)
i = 1	(10, 7, 9, 10, 9)	(7, 6, 6, 7, 9)	(5, 3, 4, 10, 9)	(4, 3, 4, 3, 3)
i = 2	(5, 4, 4, 3, 3)	(6, 5, 3, 6, 6)	(0, 7, 0, 0, 0)	(1, 3, 2, 1, 0)
i = 3	(11, 8, 11, 11, 11)	(2, 2, 5, 2, 2)	(0, 5, 5, 0, 5)	(5, 4, 4, 4, 7)
i = 4	(8, 6, 6, 4, 7)	(0, 0, 0, 0, 0)	(0, 0, 0, 0, 0)	(0, 0, 0, 0, 0)

Note: (1) Highway class i = 1, 2, 3 and 4 represent urban interstate, urban arterial, rural interstate, and rural primary respectively.
(2) The set of 5 numbers in parentheses represents five solutions in the following order: integer programming, GA run 1, GA run 2, GA run 3 and GA run 4.

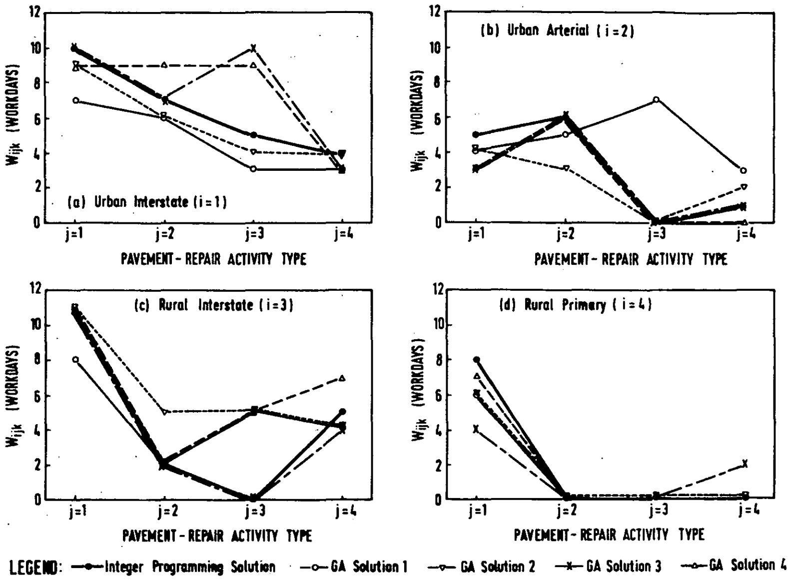

FIGURE 8 Comparison of solutions by integer programming and GAs.

haustive search for such problems, and what is important is the ability to reliably obtain a good and acceptable solution within a practical time frame. For the example problem, each GA solution run took less than 6 hr on a PC386 executing at a clock speed of 33 MHz.

Traditional methods of optimization most often are structurally rigid, with system models and improvement algorithms usually fixed in form. GAs do not have these major shortcomings because they require only payoff information defined by the objective function. This presents yet another attractive aspect of GAs in that it is relatively easy to modify the objective function to suit the user's requirements without affecting the efficiency of the GA search. For example, in pavement management at the network level the objective function could be to maximize production, as was specified in the example problem, to minimize the present worth of pavement expenditures over the analysis period, to maximize the use of allocated budgets, or to minimize the fluctuations of yearly demand for pavement expenditures.

CONCLUSIONS

This paper demonstrates the application of GAs to pavement management problems by solving a network-level pavement repair programming problem. The combinatorial explosion problem associated with a typical network-level pavement management programming analysis makes GAs an attractive technique for highway engineers and planners. The global search ability and flexibility and ease with which GAs can handle different objective functions facilitate comparison of the relative impacts of different strategies.

Genetic representation and the choice of GA operators are two major elements in the GA formulation of the problem analyzed. The considerations involved in the selection of both were illustrated in the paper through the example application of GA to a network-level pavement programming problem. In comparison with the solution obtained by an integer programming method, the example application showed that consistently good solutions can be achieved by GAs within a practical computation time on a personal computer.

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Residential Street Design: Do the British and Australians Know Something Americans Do Not?

REID EWING

American, British, and Australian street design guidelines governing geometrics, sidewalk warrants, intersection treatments, network design, and traffic-calming measures are compared. British and Australian guidelines provide for narrower pavement surfaces, sharper horizontal curves to control speeds, roundabouts and T-intersections, more efficient networks, and a wide array of traffic-calming devices. Americans have fallen behind the British and Australians in the conception of residential street functions and approaches to traffic management. British and Australian design guidelines appear to offer the best of the contemporary and the neotraditional, with European traffic calming thrown in for good measure.

In his classic *Livable Streets,* Appleyard calls streets the "most important part of our urban environment" (*1,* p. 243). It may sound like hyperbole, but just think about the effect on motorists, pedestrians, and residents of narrow, winding tree-lined streets versus wide gun-barrel designs. It almost does not matter what abuts the two road types in the way of structures, front yards, and driveways. The former will be more inviting to people and more calming to traffic.

Appleyard goes on to say:

> [W]e should raise our sights for the moment. What could a residential street—a street on which our children are brought up, adults live, and old people spend their last days—what could such a street be like? What are the rights of streetdwellers? (*1*)

In Florida the search for answers to these questions has led us to the design practices of Britain and Australia.

CURRENT DEBATE

Contemporary American street design has been much maligned recently, particularly by neotraditional planners (*2,3*). The geometrics of local streets, it is said, convert them into minifreeways. As a result of overdesign motorists travel too fast for public safety, walking and biking are discouraged, infrastructure and associated housing costs are inflated, land and energy are wasted, storm water runoff is increased, and a sense of community is lost.

The sparse network of branching streets, so common in the suburbs today, is said to force travelers up and down the local-collector-arterial hierarchy regardless of where they are going, lengthening trips and concentrating traffic at a few intersections on the collector and arterial road systems. A sparse network discourages walking trips, makes access difficult for emergency vehicles, and as much as doubles distances traveled by service vehicles.

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The curvature of streets in the suburbs, when topography does not demand it, is criticized as disorienting, unsafe because of limited sight distances, and counterproductive to the goal of getting people out of their automobiles. The slow speeds at which pedestrians move make direct routes preferable.

These criticisms have registered with the traffic engineering profession. ASCE, National Association of Home Builders (NAHB), and the Urban Land Institute (ULI) sound almost neotraditional at times in their design manual, *Residential Streets* (*4*). "Public officials and professional associations have often promulgated standards that, although reasonable for major thoroughfares, are inappropriate for local residential streets" (*4,* p.17). ITE has established a technical committee charged with developing new guidelines for traffic engineering in neotraditional neighborhoods.

Although fundamental change may be coming to the United States, it is not here yet. Americans have fallen behind the British and Australians in the conception of residential street functions and approaches to traffic management.

GUIDANCE FROM ABROAD

One hears from time to time that the Europeans, British, and Australians manage traffic better than Americans do. Much is made of Dutch *woonerf* designs (shared streets), Danish *stillevej* designs (quiet roads), German areawide traffic restraint, British environmental traffic management, and Australian local area traffic management. There is also growing interest in the United States in British and Australian roundabout design.

Thus, for the insights it might provide, this paper undertakes a comparison of American, British, and Australian residential street design guidelines. This is part of a larger effort to formulate community design guidelines for Florida.

The British and Australians use design vehicles similar to those used in the United States, and the Australians in particular are almost as automobile-dependent as Americans are (Figure 1). The following comparison therefore illustrates basic differences in street design philosophy and professional judgment as opposed to differences in street conditions.

Representing American design practice are

- *A Policy on Geometric Design of Highways and Streets* (by AASHTO),
- *Guidelines for Residential Subdivision Street Design* (by ITE), and
- *Residential Streets* (co-published by ASCE, NAHB, and ULI).

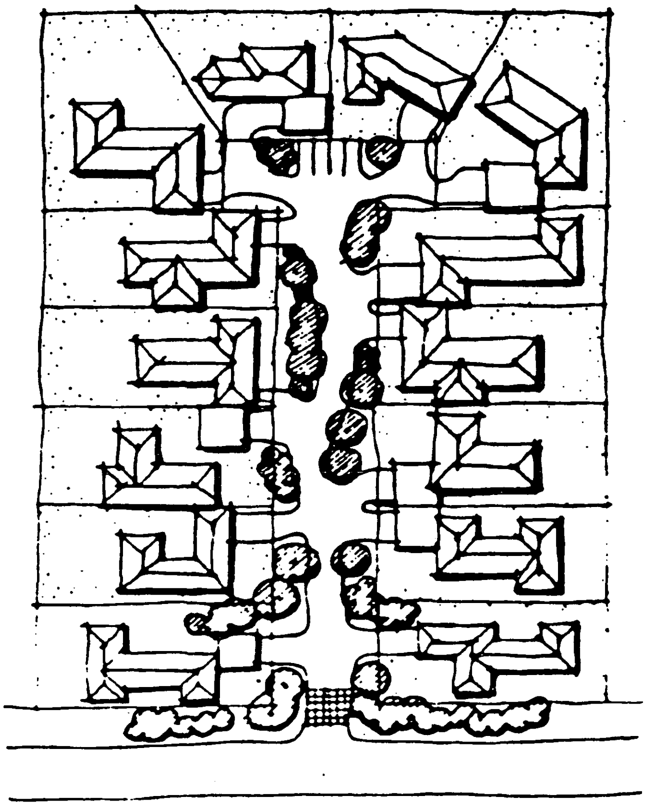

FIGURE 1 Australian subdivision with American-like density (Source: *Australian Model Code for Residential Development,* p. 30).

For purposes of comparison, two British manuals are used.

- *Residential Roads and Footpaths—Layout Considerations, Design Bulletin 32* (prepared jointly by the Department of the Environment and Department of Transport), and
- *Roads and Traffic in Urban Areas* (by the Institution of Highways and Transportation with the Department of Transport).

The first British manual provides guidelines for residential access roads (roughly equivalent to local roads in the American functional hierarchy). This is the second edition of *Design Bulletin 32,* updated in 1992 to reflect the discovery of European traffic-calming measures. The other manual, *Roads and Traffic in Urban Areas,* offers guidelines for roads at all levels in the British functional hierarchy, but most important for the present purposes are the guidelines for distributor roads (roughly equivalent to U.S. collectors).

Australian practice is harder to capture in a single set of guidelines because of differences among the Australian states. The *Australian Model Code for Residential Development,* developed under the auspices of the Commonwealth's Department of Health, Housing and Community Services, is taken to be most representative. The model code has been adopted, with some modifications, by the states of Victoria and Tasmania and is similar to South Australia's *Guidelines for Planning and Road Design for New Residential Sub-Divisions.* The central government hopes that core elements will eventually be adopted by all states after ongoing revisions are completed. The model code and supporting materials are contained in

- *Australian Model Code for Residential Development* (prepared by the Model Code Task Force of the Green Street Joint Venture),
- *AMCORD URBAN—Guidelines for Urban Housing,* Vol. 1. *Planning and Implementation Approaches,* and
- *AMCORD URBAN—Guidelines for Urban Housing,* Vol. 2. *Draft Code for Urban Housing.*

DIFFERENT VIEWS OF STREET FUNCTIONS

To help illustrate differences in design philosophy, consider the functions of local roads, collectors, and arterials as depicted by AASHTO. In the well-known hierarchy local roads mostly provide access to land, whereas arterials mostly provide mobility for through traffic. Collectors fall functionally halfway between (Figure 2).

In practice street systems in most suburban communities function more as illustrated in Figure 3. Much of the local street system consists of cul-de-sacs and loops that afford only land access, not mobility for through traffic. On the other hand many arterials are so

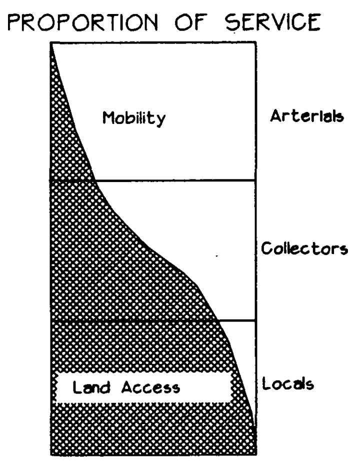

FIGURE 2 American road hierarchy (in theory).

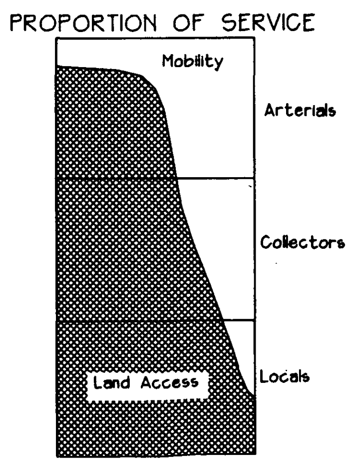

FIGURE 3 American road hierarchy (in practice).

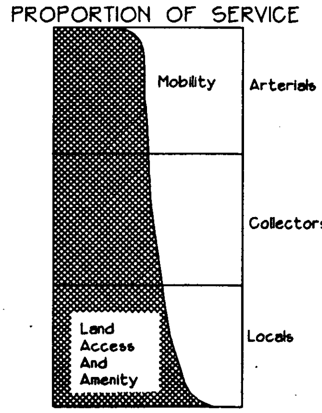

FIGURE 4 Neotraditional road hierarchy.

cluttered with driveways along commercial strips that they function more like collectors or even local roads. Freeways, of course, are the exception.

Neotraditional Road Hierarchy

Neotraditional planners tend to blur functional distinctions among local roads, collectors, and arterials in what one practitioner has called a *reduced* or *nonexistent hierarchy* of streets (*5*). They favor a return to a gridded street system, not an endless gridiron of parallel streets crossing at right angles but instead an "interrupted grid" of mostly straight streets terminating at T-intersections, Y-intersections, traffic circles, and town squares. They are adamant that local roads should carry some of the through traffic.

Neotraditional planners also emphasize the *social* and *amenity* functions of roads. The access function, acknowledged in the standard road hierarchy, relates to roads as channels of movement (albeit movement to or from an area rather than through it). In contrast, the social and amenity functions of roads relate to streets as public places and open spaces where people can commune, engage in people-watching, and the like. Given these views, a neotraditional road hierarchy would look like that in Figure 4.

British and Australian Road Hierarchy

From their writings and design manuals the British and Australians appear to embrace neither the contemporary American road hierarchy nor the neotraditional road network. Like the neotraditionalists, they acknowledge functions of local streets other than land access. Australians distinguish between access-service functions and social-amenity functions (*6*,p.1; *7*,pp.11–13; *8*,pp.136–137). They leave no doubt about which set of functions they consider more important, noting that people spend 90 percent of their time on the street "staying and playing" and only 10 percent "coming and going" (*7*,pp.13–14; *8*,pp.137–138).

However, unlike the neotraditionalists the British and Australians strive to keep through traffic out of neighborhoods. Indeed, they may differentiate the functions of arterials, collectors, and local roads even more than American traffic engineers do, leaning toward a two-class hierarchy in which roads either afford mobility or access, but ideally not both. Quoting Brindle of the Australian Road Research Board:

> Networks that avoid traffic/access ambiguity conform to the so-called "two-class" (or "separate functions") model, where roads are depicted as either traffic routes or local streets. The "two class" concept underlies British and Scandinavian practice. . . . In new street and road networks, the "intermediate" street should be avoided to the maximum degree possible.(*9*)

Brindle's two-class hierarchy is embraced to a degree by the Australian model code, which distinguishes between the mobility function of *roads* and the land access function of *streets*. One Australian engineer has redrawn the functional hierarchy as shown in Figure 5 (*10*).

INTERNATIONAL COMPARISONS

It is not in words that American, British, and Australian design manuals differ notably. The American manuals (even ITE's recommended practice, the most conservative of the three) pay homage to notions of livability and economy in residential street design. They call for a minimum of paved surface area and avoidance of excessive speeds.

Instead it is in deeds (that is, the specific guidelines set forth) that the manuals differ. What follows is an international comparison of geometrics, sidewalk warrants, intersection treatments, network design, and traffic-calming measures. Space permits discussion only of the high points.

More details are provided in Tables 1 through 3. For the sake of comparison British and Australian street dimensions have been converted from meters into feet and from kilometers per hour into miles per hour.

Geometrics: Local Streets

Design speeds are about the same for American, British, and Australian local and access roads (excluding Australian "access places"). Yet minimum pavement widths and maximum curve radii are so much greater for American than British or Australian streets

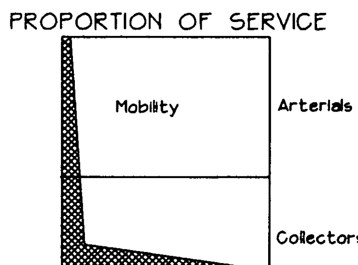

FIGURE 5 Australian-British hierarchy.

TABLE 1 Design Guidelines for Local and Access Roads

	British Design Guide 32	Australian Model Code	American AASHTO	American ITE	American ASCE/NAHB/ ULI
Design Speed	30 mph (major access roads) 20 mph (minor access roads) below 20 mph (shared surface streets)	24.8 mph (major access streets) 18.6 mph (minor access streets) 9.3 mph (access places)	20-30 mph	30 mph (level) 25 mph (rolling) 20 mph (hilly)	20 mph (access street and subcollector)
Pavement Width	12.0-18.0' (9.8' with passing bays)	18.0-21.3' (major access streets) 16.4-18.0 (minor access streets) 11.5-16.4' (access places)	26' standard (less when ROW is severely limited)	22-28' (low density) 28-34' (medium density) 36' (high density)	22-24' (access street) 26' (subcollector)
Minimum Curve Radius	32.8-98.4'	no minimums specified - maximum radius specified for traffic calming at each design speed (e.g., 98' curve to slow to traffic to 18.6 mph)	100' (as large as possible)	300' (level) 175' (rolling) 110' (hilly) (50' when street makes right angle turn)	100-150' (access streets) 150-300' (subcollector)
Curb (Corner) Radius	13.1-19.7' (depending on road width and volumes)	13.1'	15' (minimum of 25' is desirable)	20'	15-20'
Sidewalks	normally on both sides	not required on access places at least one side of access streets	at least one side	only at medium and high densities	not required on access streets at least one side of subcollectors
Minimum Sidewalk Width	4.4-6.6'	3.9'	4'	4-6'	4'

that one suspects that design speeds, in practice, are not all that similar (particularly when British and Australian traffic-calming measures are factored in).

Wider American streets result not from wider individual lanes but from a three-lane cross section, an unobstructed traffic lane, and parking on both sides (Figure 6). Americans assume the worst case (parked cars across from each other), which leaves Americans with very wide, high-speed cross sections for the common case (light traffic and no parked cars).

In contrast, the British and Australians allow one- and two-lane cross sections on local roads and deal with the worst case by requiring adequate off-street parking for residents (as Americans do almost always), banning parking on one or both sides (as Americans do sometimes), and providing frequent parking or passing bays on the narrowest streets (Figure 7).

As for curve radii, Americans strictly limit centerline curvature to extend sight distances. AASHTO's policy, for example, requires a minimum radius of 100 ft but recommends "as large a radius curve as feasible." British and Australians, on the other hand, use sharp curvature to slow down traffic to design speeds. Sight distances may be limited on such curves, but so are travel speeds.

One respect in which British and Australian guidelines do not differ much from American guidelines is in minimum curb (corner) radii at intersections. Large curb radii are not pedestrian friendly be-

TABLE 2 Design Guidelines for Collectors and Distributors

	British Roads and Traffic	Australian Model Code	American AASHTO	American ITE	American ASCE/NAHB/ULI
Design Speed	37.5-43.8 mph (30-40 mph speed limits)	31 mph (collectors) 37.2 mph (trunk collectors)	30 mph or higher	35 mph (level) 30 mph (rolling) 25 mph (hilly)	25-35 mph
Pavement Width	20.0-32.8' (2 lane) 40.4-47.9' (4 lane)	21.3-24.6' (collectors) 32.8' plus a median (trunk collectors)	20-44' (if practical, build four lanes and use the extra two for parking until needed)	24-36' (low and medium densities) 40' (high densities)	36'
Minimum Curve Radius	197-295'	no minimums specified - maximum radius of 197' specified for traffic calming at a design speed of 31 mph		350' (level) 250' (rolling) 150' (hilly)	300-500'
Curb Radius	32.8'		25-30' (where feasible)	25-30'	25-30'
Sidewalks	both sides	both sides of collectors both sides of trunk collectors when part of pedestrian network	both sides of roads used for access to schools, etc.	both sides	both sides
Minimum Sidewalk Width	5.9-6.6' (wider where larger flows)	3.9'	4'	4-6'	4'

cause they add to crossing distances and allow motorists to negotiate turns at high speeds. The British and Australian radii are larger than might be expected, given the pedestrian orientations of their other guidelines. They reflect a desire to avoid any encroachment of turning vehicles into opposing lanes.

Geometrics: Collectors

Unlike local roads, American, British, and Australian designs are similar for collectors. Apparently, the three countries have a common perception of collectors' function in the road hierarchy. They are perceived as channels of movement instead of extensions of the residential environment. (The Australians classify collectors as *residential streets* instead of *traffic routes,* implying an access function. However, the Australian design guidelines for collectors make them more like arterials than access streets.) The one respect in which the British and Australian guidelines differ significantly from the American guidelines is in their acceptance of relatively tight horizontal curves (Figure 8). As with local streets, curves are used on British and Australian collectors to enforce design speeds.

Sidewalk Warrants

Pedestrians appear better accommodated by the British and Australians than the Americans. It is not a matter of differing warrants for sidewalks. American manuals require sidewalks on higher-volume streets, and British and Australian manuals make exceptions to sidewalk requirements on lower-volume streets.

Rather, the difference among the countries is this: when sidewalks are not required, the British and Australians take extraordinary measures to slow down traffic. Both countries have incorporated *shared surface* street designs into their guidelines (Figure 9). These are streets with design speeds below 20 mph and special pavements, gateways, islands, and other measures to enforce the low design speeds. These streets differ from Dutch *woonerven* only in the avoidance of the "obstacle course" effect associated with *woonerf* designs.

Intersection Treatments

Ourston (*11*) contrasts British and American intersection designs and traffic controls. Americans usually opt for crossroads and traf-

TABLE 3 Other Design Guidelines

	British Design Guide 32	Australian Model Code	American AASHTO	American ITE	American ASCE/NAHB/ ULI
Intersection Treatments	Ts or roundabouts (crossroads only with raised junctions)	Ts or roundabouts (uncontrolled crossroads should be avoided)		T-intersections (crossroads also acceptable)	T-intersections (4-way intersections and rotaries also acceptable)
Network Designs	see narrative	see narrative		curvilinear designs/ interconnections as direct as possible	linear or curvilinear designs/short distances to collectors
Traffic Calming Devices	raised junctions chicanes speed tables narrowings gateways islands bends	chicanes bends islands narrowings humps thresholds roundabouts		curves	curves islands

fic signals or stop signs, whereas the British favor roundabouts or T-intersections with yield signs. The result, according to Ourston, is constant stop-and-go driving in the United States, whereas traffic in Britain keeps "moving safely with few stops and little sacrifice of land" (*11*).

Consistent with this characterization, the British and Australian manuals call for T-intersections or roundabouts within residential areas (Figure 10). In contrast, the American manuals, with one exception, fail to even acknowledge roundabouts. And although two American manuals recommend T-intersections for safety reasons, they still find crossroads acceptable under all circumstances.

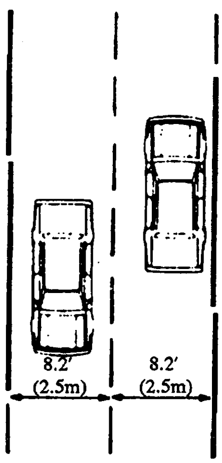

FIGURE 7 Access street in Australia (Source: *Australian Model Code for Residential Development*, p. 50).

Network Design

As a subject, road network design has slipped through the cracks between planning and engineering. Yet network design can have a profound effect on traffic congestion, vehicle miles traveled, accident rates, and fuel consumption (*12–16*).

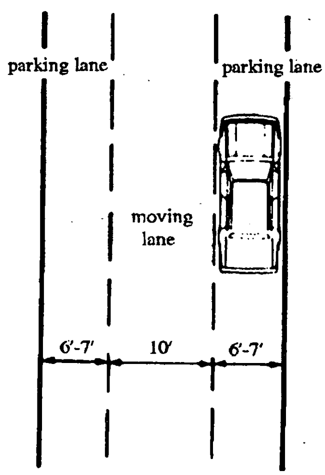

FIGURE 6 Access street in United States (*4*, p. 38).

Because of the general neglect of the subject, the design manuals provide only limited guidance regarding network design. British *Design Bulletin 32* is an exception.

Whereas the first edition of *Design Bulletin 32* (released in 1977) promoted a tree-like hierarchy of roads (relying on cul-de-sacs to avoid through traffic), the 1992 edition promotes what Noble, the principal author, calls a *hierarchical network* of traffic-calmed streets. The introduction of traffic calming gives traffic engineers the ability to design more street connections into the local network, while still discouraging through traffic and moderating impacts of local traffic.

Through traffic should be kept off residential streets, but not primarily (as in the United States) through the design of dead-end

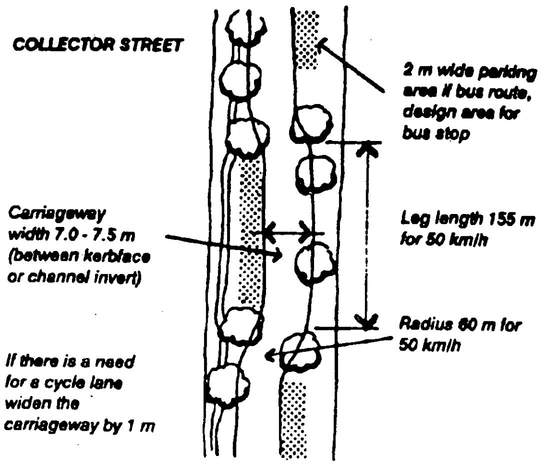

FIGURE 8 Horizontal curves on Australian collector (Source: *Australian Model Code for Residential Development,* p. 58).

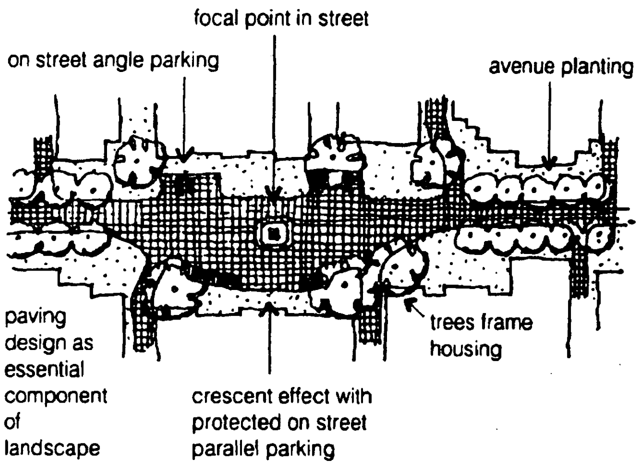

FIGURE 9 Shared surface street in Australia (Source: *AMCORD URBAN, Guidelines for Urban Housing, Draft Code for Urban Housing,* p. 2–45).

streets. Instead, the British favor circuitous through streets or loop roads (Figure 11). Cul-de-sacs are reserved for very small housing clusters. Compared with cul-de-sacs, through streets and loops are said to reduce the nuisance of reversing and turning, distribute vehicles more evenly across the network, and halve the distances traveled by service vehicles.

This is one area where the British and Australians part company. Although a cadre of Australian traffic engineers is calling for more permeability (connectivity) in local road networks and the model development code of one state—Victoria—embraces the idea, the weight of professional opinion still favors branching networks that exclude through traffic from residential streets. As a concession to network efficiency, the Australian model code limits to 1 min the amount of time required to reach a collector from any residential address. In other words, drivers must move up and down the hierarchy to get anywhere, but local streets are short enough and routes are direct enough to keep the access trip tolerable (Figure 12).

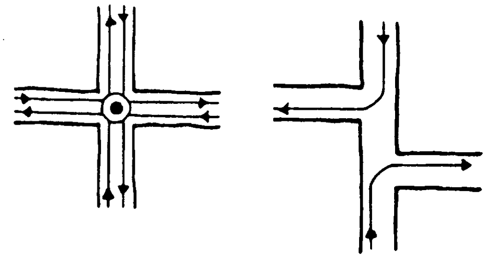

FIGURE 10 Preferred British intersection treatments (Source: *Residential Roads and Footpaths, Layout Considerations, Design Bulletin 32,* p. 45).

Traffic-Calming Measures

One of the imponderables of life in America is why engineers design roads for one speed and then promptly post much lower speed limits. When drivers exceed the speed limit, going speeds that are safe for given road widths, curvatures, and sight distances, one should not be surprised.

When the British and Australians set low speed limits, they mean it. In Britain, for example, the Department of Transport will approve low (20 mph) speed limits on residential streets only when drivers are alerted to the fact and engineering measures are taken to enforce the speed limit. Speed limits on local streets must be self-enforcing, there being minimal police presence on low-volume residential streets.

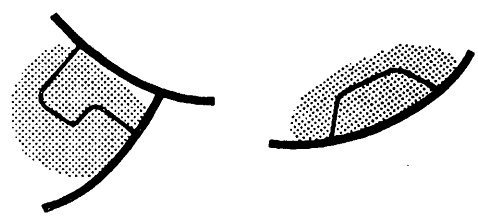

FIGURE 11 British road layouts favored over cul-de-sacs (Source: *Roads and Traffic in Urban Areas,* p. 261).

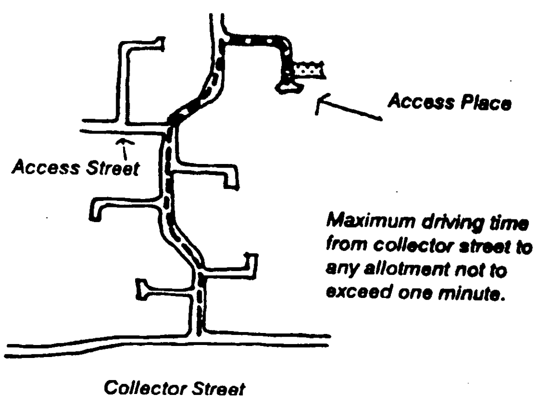

FIGURE 12 Maximum driving time out of a subdivision (Source: *Australian Model Code for Residential Development,* p. 48).

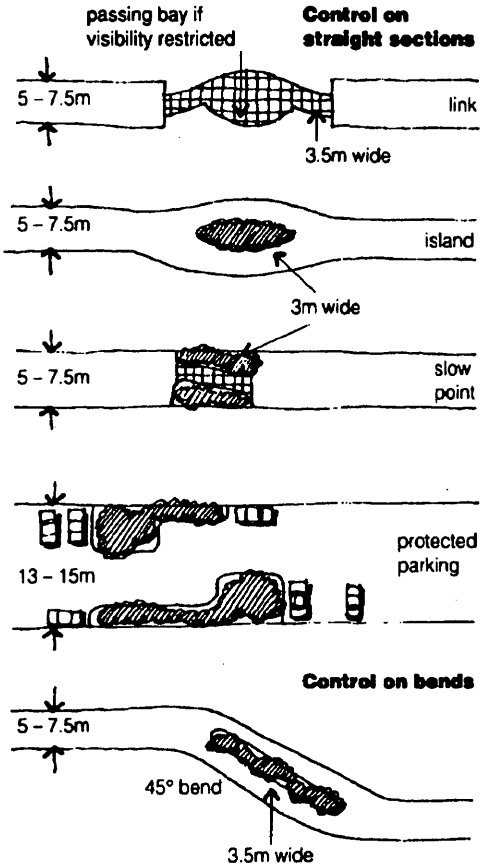

FIGURE 13 Australian traffic-calming devices (Source: *AMCORD URBAN, Guidelines for Urban Housing, Draft Code for Urban Housing,* p. 2-39).

Americans may use horizontal curvature to slow traffic or perhaps place an island at the entrance to a subdivision to create a gateway effect. But the British and Australians control speeds holistically, through European-like traffic calming. They use road network design, road geometrics, pavement texture and materials, edge treatments, roadside development, landscaping, and traffic-calming devices to create a protected environment (*17*). A host of traffic-calming devices is recommended in the design manuals of these countries (Table 3 and Figure 13).

CONCLUSION

This paper has compared American, British, and Australian geometrics, sidewalk warrants, intersection treatments, network designs, and traffic-calming measures. British and Australian guidelines provide for narrower pavement surfaces, sharper horizontal curves to control speeds, roundabouts and T-intersections, more efficient networks, and a wide array of traffic-calming devices.

Americans have fallen behind the British and Australians in the conception of residential street functions and approaches to traffic management. British and Australian design guidelines appear to offer the best of the contemporary and the neotraditional, including European traffic calming.

ACKNOWLEDGMENTS

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Nonpreemptive Goal Programming Methodology for Developing Annual Pavement Program

VENKATESH RAVIRALA AND DIMITRI A. GRIVAS

A nonpreemptive goal programming methodology for developing an annual pavement program is presented. It facilitates decision making on the basis of multiple objectives involved in the decentralized management of a pavement network. The methodology involves three major steps: (a) identification of objective functions that encompass needs at various management levels, (b) assessment of the "importance" of each objective, and (c) formulation of a goal programming model. The methodology is demonstrated by developing an annual pavement program on the basis of data specific to the New York State Thruway Authority. Objective functions with numerical goals are defined on the basis of cost factors, condition evaluation measures, and organizational requirements. Because not all goals can be achieved simultaneously, consideration of management priorities leads to the need to introduce penalties for exceeding or falling short of specified goals. Further use of the defined penalties and their relative importance enables the development of a goal programming formulation. The formulation aims to seek an optimal annual program that minimizes the weighted sum of deviations of the objective functions from their respective goals. It is concluded that the presented methodology provides a simple and versatile tool that is useful for developing an annual pavement program.

The decisions related to pavement preservation and restoration involve multi-million-dollar investments annually. Consequently, the management of a highway network will require informed and cost-effective decisions to develop annual and multiyear programs. It is crucial to optimally allocate resources for maintenance and rehabilitation (M&R) on the basis of sound principles of management and engineering.

The New York State Thruway Authority (NYSTA) and Rensselaer Polytechnic Institute are cooperating to develop a pavement management system (PMS) for the authority's toll network (*1*). An integral component of the PMS development effort is to provide optimization formulations for annual and multiyear pavement programs. The task of developing optimal programs can be accomplished by using network-level optimization methodologies (*2*). A literature review of current pavement management practices and proposed conceptual optimization formulations was presented by Ravirala (*3*). Many formulations use techniques such as dynamic programming, linear programming, or integer programming to aid in decision making. These techniques have the characteristic of selecting an optimal solution with respect to a single overriding or dominant objective. However, management of highway agencies frequently focuses on a variety of objectives—for example, to invest for economic growth, balance preservation and improvement actions, distribute work among various administrative sections, minimize long-term costs, and minimize poor pavement and maximize good pavement. Additionally, the various objectives present conflicting criteria, with different suborganizational levels aiming for specific goals.

Dominant objectives such as maximizing network ride quality may be used for optimization, provided that the decisions made adhere to certain constraints regarding other objectives. For example, the management could impose constraints on maximum investment during each year and the maximum highway mileage allowed in poor condition. An important limitation of this approach is the lack of logic in the modeling scheme to determine the best solution with respect to other objectives. Additionally, some constraints may totally dominate, making several others redundant. The active constraints may even cause infeasibility. Consequently, constraints regarding the other objectives may be satisfied, but different "optimal" solutions may yield different results with respect to many other objectives.

Thus, the problem of concern is enhancement of the optimization procedures to include multiple objectives in the decision-making process. The developed methodology must consider problem situations involving conflicting objectives that may be of varying importance to the decision maker.

The study described here aims to develop a multicriteria optimization methodology by using goal programming. The goal programming technique provides a way of striving toward several objectives simultaneously. A rational method used to determine the importance of each objective is also presented. The methodology would enable highway managers to develop the annual pavement program by considering various objectives in the decision-making process.

METHODOLOGY

The methodology of nonpreemptive goal programming involves four steps:

1. Identification of multiple objectives on the basis of condition evaluation measures and cost factors,
2. Development of policies that aid in establishment of specific goals for each objective,
3. Assessment of penalty weights for exceeding or falling short of each goal, and
4. Formulation of a goal programming model for constrained optimization.

Step 1 formalizes the objectives associated with the development and implementation of the annual program. It includes identification

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of measures for condition evaluation and cost factors that aid in the decision-making process. Multiple objective functions are defined to encompass needs at various levels of pavement management.

Step 2 addresses specific tasks of analyzing the network condition and development of policies leading to establishment of specific maintenance goals. Desirable values for each objective function identified in Step 1 are established as goals.

Step 3 involves development of a procedure for rating the importance of various objective functions in attaining their goals. The procedure involves assigning a priority for each objective—by setting a penalty weight for exceeding or falling short of each goal.

Step 4 concentrates on formulation of a mathematical model that uses goal programming techniques to conduct multiple-objective optimization. Goal programming can be used to incorporate conflicting objectives whose priority levels and relative importances can be preserved. Two cases that can be considered are (a) nonpreemptive goal programming, in which all of the objectives are of roughly comparable importance, and (b) preemptive goal programming, in which there is a hierarchy of priority levels for the objectives. In the latter case, the objectives of primary importance will receive first-priority attention, those of secondary importance will receive second-priority attention, and so forth.

In the present study nonpreemptive goal programming in which all the objectives are of comparable importance is used. Commercial and customized software was used to formulate the mathematical functions and solve the problem.

IDENTIFICATION OF OBJECTIVES

Objectives are specific to the agency needs and must be defined to be compatible with the decision makers' perspectives. The objectives should encompass needs at various levels of highway management. The following are some categories of objectives at various organization levels (in decreasing order of scope):

- Socioeconomic purpose,
- Overall organization objectives (strategic),
- Network-level (short- and long-range),
- Division-level (performance, cost, etc.), and
- Project-level (condition, implementation, etc.).

Examples of important objectives identified during this study are described next.

Socioeconomic

An important socioeconomic objective is to stimulate economic development and provide jobs. According to an FHWA study, 10.2 on-site construction jobs are created for every $1 million spent on roadway rehabilitation. NYSTA committed more than $300 million in 1992 toward its 8-year, $1.7 billion highway and bridge rehabilitation program. It is conservatively estimated that about 3,060 jobs were created. The goal for 1993 is to invest another $235 million ($68 million for highway work), creating 2,400 construction jobs.

Strategic

An important strategic objective is to allocate funds equitably among the agency's administrative divisions. This objective aims to minimize the total difference between the funds allocated for each division from that of the divisions' respective goals, which are determined on the basis of several criteria. Another strategic objective is to attain an acceptable balance between maintenance work done by contract and that done by agency forces and to ensure that unacceptable travel delays are not imposed on the patron.

Network Level

Some of the network-level objectives are to improve the ride quality and correct distressed pavement condition by setting up goals on the basis of condition measures, extend benefits to as many users as possible, equitably distribute funds to preservation and improvement programs, develop an annual program that is compatible with the multiyear program, and so on.

Division Level

The division objectives include minimizing the implementation costs and disruption to traffic. This can be achieved by limiting the number of projects by considering the equipment and personnel resources and other practical implementation considerations.

Project Level

Some of the project-level objectives are to minimize deferment of treatment to critical distress condition, implement preventive maintenance on an as-needed basis, select treatments that address as many problem situations as possible, and so on.

ASSESSMENT OF OBJECTIVES

In multiple-objective decision making it is imperative to associate a relative numerical degree of "importance" to each objective. The decision-making process involves an assessment of which objective is more important and of how much more. Rational methods are available to determine the importance of each objective with a specific goal (*4,5*).

Several objectives identified in this paper have goals whose underachievement and overachievement are undesirable, for example, capital investment and number of lane-miles desired at various condition levels. Hence, of utmost importance is the deviation of each objective function value from its goal. Two measures that demand particular attention are (a) maximum deviation of each objective from its goal, and (b) total (sum) deviation of all objectives from their goals. Both measures can be assigned penalty weights when determining the importance of objectives and their deviations from respective goals. However, minimizing one or the other might yield different results. The present study adopted the scheme of minimizing the weighted sum of deviations. This scheme provides better control over the decisions for management. Excessive deviations can be prohibited by establishing bounds on the maximum deviation of each objective from its goal.

A significant aspect of evaluating objectives and ascertaining weights is the use of experts' judgments. It is necessary to synthesize different judgments when more than one person is involved in the assessment. The weights are derived on the basis of individual assessments by using the following equations:

$$P_{ij} = \beta_{ij} / \sum_{i=1}^{m} \beta_{ij} \tag{1}$$

$$P_i = \sum_{j=1}^{n} P_{ij} / \sum_{j=1}^{n} \sum_{i=1}^{m} P_{ij} \tag{2}$$

where

P_{ij} = penalty weight computed for objective i by expert j,
β_{ij} = penalty assessed for objective i by expert j, and
P_i = penalty weight computed for objective i.

MODEL DESCRIPTION

The annual program development and budget allocation is modeled as a modified *assignment problem* (*6*). Nominal sections receiving specific treatments represent the origins, and fixed-length projects represent the destinations [Figure 1 (*a*)]. Each nominal section should receive only one type of treatment and should be assigned to only one project.

The modeling process can be described by the following four-step procedure:

- Establish tentative projects at fixed intervals along the entire network,
- Determine the nominal section boundaries and identify three neighboring projects to which each nominal section can be assigned,
- Specify the treatment alternatives for each nominal section, and
- Formulate the goal program.

In the present study tentative projects are established at every 8-km (5-mi) interval along the network [Figure 1 (*b*)]. Treatment alternatives are defined after analyzing the condition of each nominal section and assessing their individual needs [Figure 1 (*c*)].

As applied to this model the goal program assigns each nominal section to a project and determines the type of treatment to be performed for each nominal section. It also determines the optimal set of projects to be implemented as part of the annual program.

GOAL PROGRAM FORMULATION

The general form of a goal program can be expressed as follows (*4*):
Minimize

$$\sum_{g \in G} (P_{og} d_g^+ + P_{ug} d_g^-) \tag{3}$$

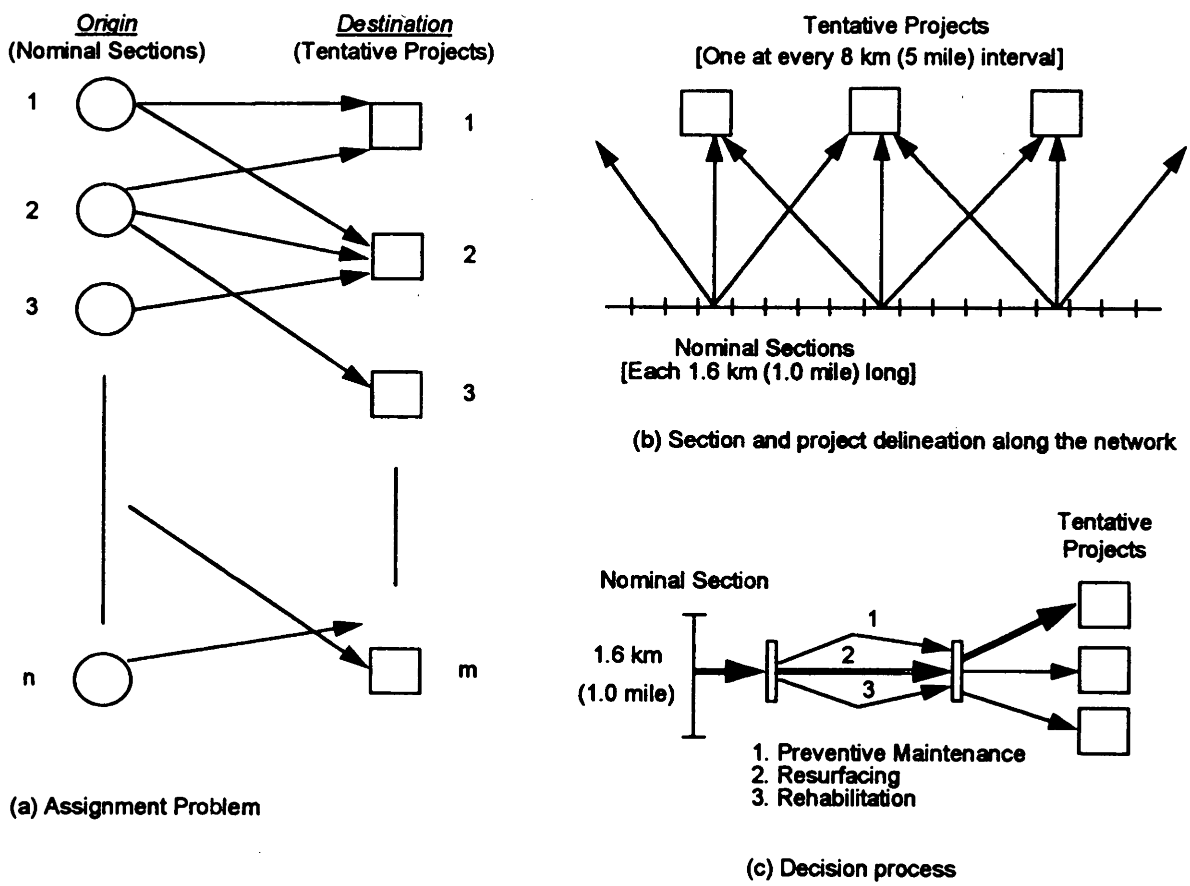

FIGURE 1 Illustration of assignment model and decision process.

Subject to

$$\sum_{v \in V} (a_{gv} X_v) + d_g^- - d_g^+ = b_g, \text{ for } g \in G \quad (4)$$

$$X_v, d_g^+, d_g^- \geq 0, \text{ for } g \in G \text{ and } v \in V \quad (5)$$

where

G = set of goals,
d_g^+ = overachievement of goal g,
P_{og} = penalty associated with d_g^+,
d_g^- = underachievement of goal g,
P_{ug} = penalty associated with d_g^-,
V = set of basic variables,
X_v = basic variables in the individual objective functions (or goal equations),
a_{gv} = coefficients of basic variables, and
b_g = targeted goals.

Equation 3 represents the unified objective of minimizing the weighted sum of deviations of individual objective functions from their respective goals. Equation 4 represents the goals to be approached as closely as possible.

Basic variables, objectives (with goals), and constraints specifically defined to conduct a case study are as follows:

Basic Variables

$$X_{ijk} = \begin{cases} 1 \text{ if section } i \text{ is assigned to project } j \text{ and receives treatment } k \\ 0 \text{ otherwise} \end{cases} \quad (6)$$

Objectives (with Goals)

Equation 7 addresses the socioeconomic objective of capital investment and the strategic objective of equitable allocation of funds among administrative divisions. The left side of the summation represents the total M&R expenditure within each geographical class, which is equated to the investment goal on the right side.

$$\left(\sum_{i \in I_s} \sum_{j \in J_i} \sum_{k \in K_i} l_i\, c_{jk}\, x_{ijk}\right) + d_{s1}^- - d_{s1}^+ = b_{s1} \text{ for all } s \quad (7)$$

where

I_s = set of all nominal sections within geographical class s [$s \in$ (Network, New York, Albany, Syracuse, Buffalo)],
J_i = set of projects to which section i can be assigned,
K_i = set of treatment alternatives for section i,
l_i = length of section i,
c_{jk} = unit cost of treatment k in project j, and
b_{s1} = M&R capital (total annual funds) goal for geographical class s.

Equation 8 represents the goals defined on the basis of condition measures. The left side is a summation of the total lane-miles that belong to a geographical class and that would be in a certain condition class after applying treatments. The right side goal is the desired number of lane-miles in that particular condition class. For example, the left side of a particular goal equation may represent the total mileage in New York that would be in *Excellent* ride quality after implementing the annual program.

$$\left(\sum_{i \in I_{sn}} \sum_{j \in J_i} \sum_{k \in K_i} l_i\, p_{ikn}\, X_{ijk}\right) + \mathrm{d}_{sn}^- - \mathrm{d}_{sn}^+ = b_{sn} \text{ for all } s \text{ and } n \quad (8)$$

where

I_{sn} = set of nominal sections belonging to geographical class s that would be in condition class n after implementing the annual program,
p_{ikn} = probability that nominal section i would be in class n after receiving treatment k, and
b_{sn} = condition goal (total lane-miles) for geographical class s and condition class n.

Constraints

Equation 9 ensures that each nominal section gets assigned at most to one project and will receive at most one treatment type.

$$\sum_{j \in J_i} \sum_{k \in K_i} X_{ijk} \leq 1 \text{ for all } i \in \mathrm{I} \quad (9)$$

Equation 10 ensures that the total mileage within each geographical class will remain constant during the transitions between condition classes. (Equations for several other types of goals identified earlier are not presented here for the sake of brevity.)

$$\sum_n d_{sn}^+ - d_{sn}^- = 0 \text{ for all } s \quad (10)$$

CASE STUDY

Inventory and Condition Data

The goal programming methodology was applied to develop an annual program by using information specific to NYSTA. The thruway consists of approximately 1,024 km (640 mi) of Interstate highway in each direction. It was originally constructed as portland cement concrete (PCC) pavement, but over the years approximately 87 percent has been overlaid with asphalt. For efficient management NYSTA is organized into four administrative divisions (New York, Albany, Syracuse, and Buffalo) and several administrative sections within each division.

NYSTA annually conducts distress survey and records severity and extent of 14 distress types on every 0.16-km (0.1-mi) segment. This information is used to derive two types of condition measures, namely, crack (joint) rating and surface (slab) rating for overlaid (concrete) pavement.

The network is divided into nominal sections of approximately 1.6 km (1.0 mi). For each nominal section the average value of both condition measures is derived. Cutoff values were established to define *Excellent, Good,* and *Fair* classes for each condition measure. Table 1 shows the distribution of mileage (percentage of network) among various divisions and condition classes. Only 931.6 km (582.3 mi) was rated in the year 1992.

TABLE 1 Percentage of Thruway Mileage in Each Condition Class

Condition Measure	Condition Class	Division New York OVL		Albany OVL		Syracuse OVL		Buffalo OVL	
		OVL	PCC	OVL	PCC	OVL	PCC	OVL	PCC
Crack/ Joint Rating	Excellent	10.6	0.0	7.8	0.0	12.0	-	11.2	0.0
	Good	3.1	0.0	4.7	0.0	4.3	-	7.4	0.1
	Fair	4.4	8.3	8.6	2.6	7.3	-	6.1	1.5
Surface/ Slab Rating	Excellent	11.3	0.0	12.6	0.0	18.9	-	9.0	0.0
	Good	4.3	0.0	3.5	0.0	4.2	-	3.3	0.0
	Fair	2.5	8.3	5.0	2.6	0.5	-	12.4	1.6

OVL = Overlaid PCC = Concrete

(-) indicates mileage non-existent

Objectives and Goals

Two important types of objectives, namely, capital investment and condition improvement, have been included in this case study. It was necessary to demonstrate the model's capability to address the decentralized nature of decision making in highway management. Hence, different goals were defined for the four administrative divisions and the overall network. The capital goals were specified as millions of dollars to be invested for pavement M&R. The condition goals were specified as desired percentage of network within various condition classes.

In all, 35 goals were targeted—5 capital goals (one for each geographical class) and 30 condition goals (six condition classes for each geographical class). The capital investment goal for each of the four administrative divisions was specified as \$10 million, and overall it was \$40 million. The percentages of mileage desired in each condition class are given in Table 2.

Assessment of Objectives

Although specific numerical goals have been defined for various objectives, it is well recognized that the deviations are not of equal importance. For example, a unit deviation from the overall capital goal is relatively more undesirable compared with a unit deviation from individual division capital goals. Hence, the penalty of deviating from the overall capital goal must be higher. In addition, the penalty for overachieving and underachieving a goal could be different. For example, exceeding the targeted mileage (overachieving) in excellent condition need not be penalized, whereas falling short of the targeted mileage (underachieving) must be penalized. Table 3 shows the penalty weights defined for capital and crack condition rating classes. Note that overachieving the excellent condition goals and underachieving the fair condition goals have a zero penalty. Also, as a simple case all four divisions have been judged to be equally important in achieving their respective goals.

TABLE 2 Magnitude of Condition Goals Targeted and Achieved

Condition Measure	Condition Class	Overlaid Mileage (%) Targeted	Overlaid Mileage (%) Achieved	Concrete Mileage (%) Targeted	Concrete Mileage (%) Achieved
Crack/ Joint Rating	Excellent	45.85	58.09	0.62	0.00
	Good	24.04	22.66	0.71	0.58
	Fair	17.65	12.63	11.13	6.04
Surface/ Slab Rating	Excellent	56.21	52.55	0.62	0.00
	Good	19.72	27.16	0.62	0.62
	Fair	11.61	13.71	11.21	5.95

Computer Implementation

The presented goal programming methodology involves a large-scale mixed-integer programming formulation. It requires extensive computer programming to process the input data, generate the equations, determine the optimal solution, and finally process the output to summarize the results. It was decided to develop customized software that included four main modules. The first module consists of data base routines that store and retrieve information on inventory, pavement condition, treatment alternatives, costs, and so on. The second module has routines that allow the user to (a) define the objectives to be included in the formulation, (b) specify numerical goals, and (c) set penalty weights. The third module has routines that (a) define meaningful names for basic and deviation variables, (b) generate the variable coefficients for the objective function, goal, and constraint equations, and (c) formulate the goal program using the mathematical programming system (MPS) file format (*7*). Finally, the fourth module consists of routines that generate reports that summarize the input data and the results.

The LINDO commercial software was used to read the formulation in MPS format and solve the goal program. Although the basic variables are defined as 0–1 integer variables, this restriction was relaxed and the problem was solved as a linear program.

Results

The most important result that enables assessment of the annual program's effectiveness is the magnitude of deviations from the targeted goals. For all divisions the target was to increase the Excellent and Good mileage by 5 percent and correspondingly to decrease the Fair mileage by 10 percent. Table 2 shows the magnitude of condition goals targeted and achieved. There is a substantial increase (13 percent) in the Excellent condition mileage of overlaid

TABLE 3 Penalty Weights for Various Goals

Geographical Class	Goal Type: Capital Investment	Overachieving Crack Rating: Ex	Gd	Fr	Underachieving Crack Rating: Ex	Gd	Fr
New York	0.25	0.0	0.25	0.5	0.5	0.25	0.0
Albany	0.25	0.0	0.25	0.5	0.5	0.25	0.0
Syracuse	0.25	0.0	0.25	0.5	0.5	0.25	0.0
Buffalo	0.25	0.0	0.25	0.5	0.5	0.25	0.0
Overall	0.5	0.0	0.5	1.0	1.0	0.5	0.0

Ex = Excellent Gd = Good Fr = Fair

pavement crack rating. Also, there is a significant decrease (5 percent) in the Fair condition mileage. This can be attributed to the zero penalty associated with either case. The Excellent mileage of the surface condition has slightly decreased, but the Good mileage has increased significantly (8 percent).

Table 4 summarizes the capital investment data for each geographical class. Although goals and penalties are set to be equal, the funds allocated among the four divisions vary widely. This variation can be attributed to several factors, such as (a) disparity in the current condition, (b) differences in the condition goals, and (c) penalty weights. For example, the New York Division has significant mileage of concrete pavement in Fair condition, which is undesirable. Hence, it was targeted to decrease the Fair condition mileage by 10 percent. Also, a higher penalty was given to the overachievement of target mileage in the Fair condition class. This decision demonstrates the control that management can exercise on the decision making.

Table 5 summarizes the percentage of mileage receiving each treatment type. The program recommends rehabilitation on 12 percent of the network and resurfacing on 6 percent, which are realistic recommendations. The program also suggests that more than 70 percent of the thruway requires some form of maintenance. This indicates the need to increase the scope of the case study to include a complete set of treatment alternatives and objectives. Both the numerical goals and penalty weights need to be refined after further analysis.

DISCUSSION OF RESULTS

Identification of Objectives

The objectives identified in the present study encompass the needs at various levels of highway management. However, a distinction can be made between the single-year and multiyear objectives involved in overall pavement management. The presented goal programming methodology incorporated only the single-year objectives. This assumes that the single-year objectives are compatible with the multiyear objectives. An agency must have an established multiyear program to ensure such compatibility. A state increment optimization methodology is used to develop an optimal multiyear program for NYSTA (*2*). The optimal multiyear program defines the capital investment options, long-term condition goals, and M&R strategies for the entire network. The results from multiyear analysis provide the capital and condition goals for the annual program. The state increment method also determines the lane-miles of pavement in each state that should receive each of the possible treatment options. This information can be used to define additional goals on treatment quantities for the annual program and ensure congruency between the single-year and multiyear programs. For example, if the multiyear program recommends a 10 percent network rehabilitation in the first year, then an additional goal would be to develop an annual program that targets a 10 percent network rehabilitation.

TABLE 4 Comparison of Capital Investment Data for Each Geographical Class

Division	Capital Goal ($Million)	Deviation ($Million)	Achieved ($Million)	Mileage (km)	Investment $1000/km
New York	10.00	1.43	11.43	492.96	23.20
Albany	10.00	-3.58	6.42	441.76	14.32
Syracuse	10.00	-6.91	3.09	440.64	7.02
Buffalo	10.00	-4.50	5.50	487.84	11.26
Overall	40.00	-13.56	26.44	1863.2	14.18

Assessment of Objectives

Assessment of objectives, in the context of multiple-criteria optimization, is a process of defining the relative importance of criteria. This process involves ranking the criteria with *priority* or *weight.* Priority refers to the case in which the criteria are ordered according to importance and, unless the higher-level criterion is considered, the next one does not come into play. In other cases weights are attached to differentiate the relative importance of several criteria with equal priority.

The case study presented here involved only a few objectives of comparable importance. It was assumed that all objectives are of equal priority. Hence, the importance of each objective was assessed by assigning penalty weights, and the nonpreemptive goal programming technique was used to obtain an optimal solution. The methodology can easily be extended to include multiple objectives with different priority levels. Such objectives can be unified into a single objective function by manipulating their weights, thus converting a preemptive goal program into a nonpreemptive goal program. To accomplish this conversion the weights of the highest-priority objectives need to be multiplied by a number that is vastly larger than the weights of the objectives at the next priority level.

TABLE 5 Percentage of Mileage Receiving Each Treatment Type

Treatment Type	Division							Overall
	New York		Albany		Syracuse	Buffalo		
	OVL	PCC	OVL	PCC	OVL	OVL	PCC	
Do Nothing	0.88%	0.00%	2.26%	0.00%	3.80%	0.72%	0.00%	7.66%
Maintenance	15.78%	1.06%	16.30%	0.26%	19.15%	21.12%	0.77%	74.44%
Resurfacing	0.60%	1.00%	1.70%	0.17%	0.70%	1.55%	0.52%	6.23%
Rehabilitation	0.86%	6.28%	0.88%	2.15%	0.00%	1.25%	0.26%	11.66%
Total	18.12%	8.34%	21.13%	2.58%	23.65%	24.64%	1.55%	100.00%

OVL = Overlaid PCC = Concrete

Evaluation of Computational Aspects

The presented goal program is a mixed-integer type program that has both 0–1 integer variables (basic variables) and real variables (nonnegative deviation variables). It is well recognized in practice that it is computationally expensive to solve large-scale integer programs. In this case study the restriction on obtaining an integer solution was relaxed, and the problem was solved as a linear program. This resulted in a significant reduction in computation time. For example, a formulation—with approximately 3,400 variables and 1,300 rows—was solved within 5 min by using an IBM 3090–200S computer. An equivalent integer program requires several hours. Only 2 (of more than 3,300) 0–1 integer variables resulted in noninteger solutions, indicating that the formulation can be efficiently solved as a linear program.

SUMMARY AND CONCLUSIONS

This paper presented a nonpreemptive goal programming methodology for developing an annual pavement program. The emphasis was on incorporating multiple objectives into the decision process involved in the decentralized management of a pavement network. Three major steps of the methodology are (a) identification of objective functions with specific numerical goals on the basis of condition evaluation measures and cost factors, (b) assessment of the importance of each objective in the form of penalty weights for exceeding or falling short of each goal, and (c) formulation of a goal programming model for constrained optimization. The formulation aims to seek an optimal solution that minimizes the weighted sum of deviations of the objective functions from their respective goals. The usefulness of the methodology was demonstrated through a case study. Presented is an annual pavement program that was developed on the basis of information specific to NYSTA.

From the findings of the study, the following conclusions may be drawn:

- Developed goal programming methodology effectively incorporates multiple, conflicting, and prioritized objectives that are present in highway management.
- Presented assignment model for annual program development has the advantage of simplicity and versatility because it yields simple linear functional forms for objectives and constraints.
- Employment of objective functions and their deviations from their respective targeted values provides a useful tool to decision makers in their effort to explicitly prioritize objectives and establish an acceptable trade-off.

GLOSSARY

Attributes: Characteristics that are used for certain physical and functional features of the infrastructure, for example, condition, safety, ride quality, and costs.

Constraints: Mathematical expressions for restrictions on attribute levels.

Goals: Although objectives are aspirations *without* the decision maker specifying their levels, goals are aspirations with given a priori levels of desired attributes.

Nominal Section: A continuous length of pavement that can be classified into a pavement state and has properties "similar" to those of any other section classified into the same state. Such sections may be aggregated in the decision process.

Objectives: Mathematical expressions for aspirations that indicate directions of improvement of selected attributes such as minimize costs and maximize ride quality.

Pavement Program: A plan that identifies the maintenance, rehabilitation, and reconstruction projects (either specific or nominal sections) tentatively scheduled for implementation. It can be either an annual (single year) or a multiyear program.

State: A combination of specific levels of variables that describe the dynamic behavior of the system. The variables used for the definition of state may be pavement condition parameters, traffic parameters, or any others that affect the decision process.

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Impact Analysis of Road Keeping: Case Study of Lapland District in Finland

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The northernmost highway district of Lapland, Finland, differs considerably from other regions in Finland. Its road network is very long, about 9000 km, whereas traffic volumes are low because of an extremely low population density. The arctic climate puts severe restrictions on the technology used, the amount and quality of winter maintenance, and road standards. To fully comprehend the implications of proposed budget cuts, the district initiated an analysis on the impacts of alternative strategies. A network-level pavement management system was used to allocate and optimize funds for rehabilitation and to analyze the development of the condition of the network with nonoptimal funds. Summer and winter maintenance were analyzed by an analytical hierarchical process. As to investments, in addition to a traditional cost-benefit analysis, indirect economic effects were studied with a regionalized input-output model. According to the results the pavement condition targets cannot be achieved with the current budget level, maintenance moneys can be decreased somewhat, and cuts in investment funds have very harmful effects on the local companies and employment. The preliminary results suggest that rehabilitation, summer and winter maintenance, and highway investments are not, mathematically speaking, separable and thus cannot be viewed independently of each other. This being the case, road keeping must be seen as a whole and the optimization of the funds allocated to its subcomponents must be done simultaneously with a global and comprehensive optimization function.

The Finnish Road Administration (FinnRA) is the central administrative body for nine highway districts. FinnRA is responsible for policy making for the whole country, development of standards and guidelines, evaluation of the districts' efficiencies and productivities, and relations with the Ministry of Transportation and Parliament. The districts execute the program and policies independently within a given budget framework. FinnRA allocates funds for rehabilitation by following the results of its network-level pavement management system. The amount of maintenance money that each district receives depends on the length of its road network and traffic volumes. Investment funds, on the contrary, are decided by the Parliament on a project-by-project basis.

Lapland, the northernmost highway district of Finland, differs considerably from other highway districts. The area is very vast, but population density is very low. This implies a long road network, about 9000 km, whereas average traffic volumes are low.

For the planning period of 1992 to 1996, about 400 million to 500 million marks per year (US$1 = about 4.7 Finnish marks) had been allocated to the Lapland road district. Almost 50 percent of the funds were for rehabilitation, 30 percent were for summer and winter maintenance, and the remaining 20 percent of the funds were for investment projects. However, in the beginning of 1991, the performance of the Finnish economy slowed and there were severe pressures on the government's budget. As a consequence FinnRA suggested a decrease in resources for road keeping in the Lapland district. To fully comprehend the implications of the proposed budget cuts the district initiated an analysis on the impacts of alternative strategies on the road network condition, agency and user costs, maintenance level of service, and the regional economy.

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REHABILITATION OF PAVED ROAD NETWORK

The Model

FinnRA has a two-level pavement management system: a network-level model (HIPS) to assist managers at the top level of administration in doing strategic planning and a project-level model (PMS91) to help district engineers in developing yearly programs and budgets.

HIPS consists of a long-term model for analyzing long-term budget and quality goals and a short-term model for finding policies that can bring the current road network closer to the long-term goals. Long-term goals are determined by minimizing the sum of agency and user costs, taking into account budget and condition constraints. The road network can be divided into smaller networks according to environmental and traffic characteristics.

PMS91 is used to prepare a specific list of projects that meet the policy and budget guidelines according to the results of HIPS. Although the network-level model deals rigorously with the economic implications of projects, the project-level system is meant to be used more subjectively according to local circumstances.

Inputs to HIPS Model

For the purpose of the present study the road network of Lapland was divided into nine subnetworks. There were three pavement classes: asphalt concrete main roads, soft asphalt main roads, and soft asphalt secondary roads. The average daily traffic (ADT) classes were chosen to reflect relative traffic volumes on the respective pavement types. The classification is summarized in Table 1.

Analysis with the HIPS model requires the following input data from each of the nine subnetworks: *current condition* data on bearing capacity, roughness, ruts, and defects; *average daily traffic* to calculate the user cost; and *current budget levels* for different subnetworks. *Condition constraints* were chosen according to the FinnRA's policy targets; *agency costs,* input data on *allowable states* and *transition probability models* were the same as those used by FinnRA. The *discount rate* was 6 percent.

Results of HIPS Model

The HIPS model analyzes the distribution of the condition states of all paved roads and predicts the change in distribution in an 8-year

TABLE 1 Classification of Subnetworks

Functional class	Pavement type	ADT Vehicles/day
Main roads	Asphalt concrete	< 1500
		1500 - 6000
		> 6000
	Soft asphalt	< 350
		350 - 800
		> 800
Secondary roads	Soft asphalt	< 350
		350 - 800
		> 800

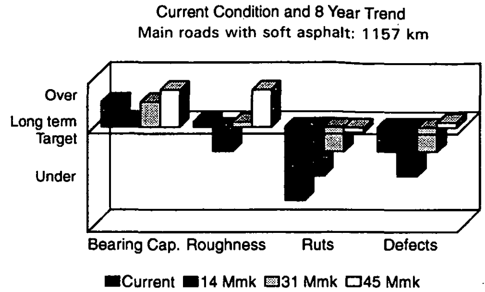


FIGURE 2 Deviation from target condition with different annual budget levels (percent of length).

time horizon. The objective of the HIPS model is to find the optimal condition level of roads by minimizing total costs to the society, that is, the sum of both road maintenance and rehabilitation costs and road user costs. The road condition targets are set centrally by FinnRA. More detailed information of the HIPS model has been published previously (*1*).

The yearly rehabilitation actions as well as user costs change subject to different rehabilitation policy alternatives and budget constraints. Comparison of the impacts and costs of different rehabilitation actions allows the district to choose the most efficient strategy within a given budget framework. In Figure 1 user cost savings for the aggregate network are shown as a function of different rehabilitation budget levels. The savings are estimated after 6 years of rehabilitation actions proposed by the HIPS model, and they are calculated as a difference from the current situation, in which the annual budget is 100 million marks.

There is a steep decline in user costs as a response to rehabilitation actions up to a yearly budget level of 150 million marks. For a 25-million-mark increase in the pavement management budget, user costs decrease by about 15 million marks annually. On the other hand, the relative user benefits from a rehabilitation budget of 150 million marks or more are less striking. The 25-million-mark increase in pavement management investments benefits the users by only less than 5 million marks, implying diminishing marginal returns.

In Figure 2 the development of the road condition with different annual budget levels is presented for the subnetwork of soft asphalt main roads. Its total length is 1157 km. The condition variables

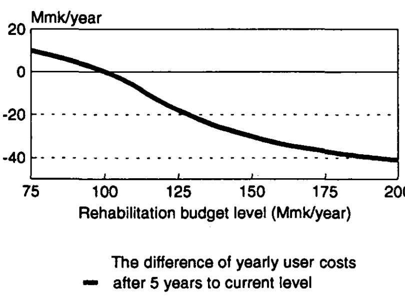


FIGURE 1 User cost difference in the sixth year [in millions of Finnish marks (Mmk)].

(bearing capacity, roughness, ruts, and defects) are presented after 8 years. The different budget levels used were 14 million, 31 million, and 45 million marks, whereas the current budget level is 25 million marks.

The actual budget level for bearing capacity and roughness is satisfactory, whereas the amount of ruts and defects would increase in comparison with the target condition. The target condition would be attained for all condition variables only in 8 years with a considerable increase, almost a doubling (from 25 million to 45 million marks), in the funds allocated for this pavement class.

However, when the targets concerning ruts and defects are obtained, bearing capacity and roughness exceed the target level. This is mostly because the rehabilitation actions necessary to decrease ruts and defects to an acceptable level inevitably ameliorate the road's bearing capacity and roughness. This might imply a mathematical nonseparability and thus simultaneous optimization of the two components.

As a summary the following conclusions concerning pavement maintenance of all nine subnetworks can be obtained:

- Current budget level (132 million marks) is satisfactory because the main road network can be kept in good condition and the other subnetworks approach the target condition level, although they do not reach it within the study period;
- It is unacceptable to decrease the budget level to 75 million marks, because it would mean that the conditions of all subnetworks would deteriorate unless the condition targets are reviewed simultaneously;
- Economic depression that Finland is facing does not allow for an increase in the current budget level, although the optimal budget resources should be increased to nearly 165 million marks to meet the pavement condition targets set by FinnRA;
- HIPS model needs to be further developed so that the achievement of targets concerning ruts and defects does not lead to such rehabilitation actions that bearing capacity and roughness target levels are exceeded; and
- It could be worthwhile to study the possibility of emphasizing the difference between short- and long-term rehabilitation policy so that condition targets or discount rate reflects more accurately the economic situation of the country.

SUMMER AND WINTER MAINTENANCE

Because a summer and winter maintenance model has not yet been fully developed in Finland, for the purposes of the present study an analytical hierarchical process (AHP) methodology was chosen. AHP was developed by T. L. Saaty to answer the problems of decision making in the face of risk and uncertainty, diverse and controversial factors, and different opinions and judgments [for the theory of AHP, see Saaty (*2*)]. AHP is a method of pairwise comparisons and it allows for

- Selection of the best action or alternative among different policy options,
- Development of a framework for analyzing factors that affect the results of a chosen policy, and
- Performance of planning by iterative alignment of the priorities of projected and desired targets.

In the present study AHP was used to evaluate the appropriateness of the level of funds allocated between different summer and winter maintenance actions, assuming, however, that the total budget is optimal. The respondents consisted of district management and operation staff; road users were not interviewed at this stage.

The hierarchy contained two levels: road users and pavement classes. Road users were divided into four functional classes: commerce/distribution, local inhabitants, industry, and tourism. The respondents agreed that the most important classes were the last two. These got a more important weight, implying that maintenance actions on the subnetworks serving these two user groups were valued more than actions on other subnetworks irrespective of, for example, total traffic volumes.

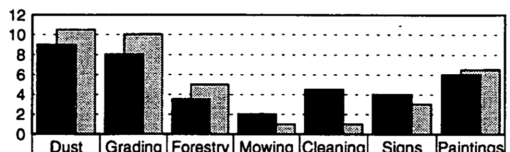
The pavement classes were asphalt concrete roads, soft asphalt main roads, soft asphalt secondary roads, and gravel roads. The respondents emphasized the importance of the two main road classes.

Major divergences between respondents were found in funds used and funds needed to accomplish actions related to the level of service standard, which represent 12 percent of the total maintenance funds. The maintenance personnel seemed to prefer construction-based actions to actions that served road users. However, if the road users were included in the hierarchy setting, the results would probably have been quite different in this respect. The results for summer maintenance are summarized in Figure 3.

The respondents would increase the funds allocated to dust removal, grading and forestry by more than 20 percent, or by 5 million marks to 25 million marks. On the other hand they believed that the funds allocated for cleaning could be decreased to one-third of the actual budget.

INVESTMENTS

The average yearly investment budget for Lapland has been almost 100 million marks. Although highway construction technology is more capital-intensive than labor-intensive, when indirect effects are taken into account effects on regional employment are important. The following analysis was made assuming that the investment level is decreased to 70 million marks or increased to 130 million marks. The calculations were made with a regionalized input-output model, and the monetary results are summarized in Table 2.



	Dust	Grading	Forestry	Mowing	Cleaning	Signs	Paintings
Real expenses■	9	8	3,5	2	4,5	4	6
AHP weighted costs▤	10,5	10	5	1	1	3	6,5

FIGURE 3 Summer maintenance, actual expenses, versus AHP weighted budget.

TABLE 2 Indirect Effects of Road Investments

	Yearly budget level (Mmk)		
	70	100	130
Sector	Indirect effect (Mmk)		
Agriculture	3	3	5
Industry	53	75	98
Transports	53	77	100
Services	27	38	50
Total	136	193	253

When indirect effects are taken into account, the economic impact of highway construction is twice as big as the initial investment. The employment effect is 600, 720, and 950 persons employed, respectively. On the other hand, if the investment budget for Lapland is cut from 100 million marks to zero, as suggested, the region's enterprises lose almost 200 million marks as direct and indirect incomes and the number of unemployed increases by more than 700 persons.

Because road keeping is not market driven in Finland but is financed through general taxation, the long-term effects, direct as well as indirect, should simultaneously be taken into account by the government when allocating funds for road keeping. If highway investment moneys are cut it is necessary to increase unemployment funds or other means of stimulating the regional economy.

A preliminary comparison of user cost savings from highway investments and pavement management rehabilitation showed that the allocation of funds between these two subcomponents was not straightforward. When indirect economic impacts are taken into account and depending on traffic volumes and the composition of traffic, the total net benefits from the construction of new roads in urban areas seemed to exceed those of pavement maintenance of rural gravel roads.

It is thus presumable that rehabilitation and investments are, mathematically speaking, nonseparable. This being the case, a simultaneous optimization model for the two subcomponents is essential to maximize the welfare of Lapland. It is also possible that maintenance is nonseparable from rehabilitation or investments. This being the case, it would be important to develop a comprehensive model to cover all road-keeping actions, because nonseparability implies that optimization or decision making for different actions cannot be done separately but that the allocations of funds must be done centrally and simultaneously.

The possible sources for nonseparability stem from several factors, for example, common inputs in the rehabilitation, maintenance, and construction technologies such as machines, planning, and supervisory staff and the indivisibility of equipment and personnel.

CONCLUSIONS

From the results of the Lapland case study several conclusions were made concerning the district's road-keeping policy:

- Unless the pavement condition targets, set centrally by FinnRA, are reviewed to reflect the scarcity of funds because of the economic recession, Lapland cannot achieve the targets with the current rehabilitation budget. In the long term, however, an increase in rehabilitation funds is necessary to prevent deterioration of the network.
- As to summer and winter maintenance, there are a few components for which funds could be decreased somewhat. On the other hand, more funds seem to be needed for some other factors to offer an adequate level of service to the road users. However, road users, the demand side, should be included in the hierarchy setting and valuation process in the future.
- Decisions on the level of road investments have a considerable impact on the economy of and employment among individuals in the Lapland region. In addition, some of the investment projects seem to be more beneficial to road users and the agency than rehabilitation of those road sections with very low traffic volumes. This question necessitates further research, however.

The main suggestions of this study concerning Lapland's long term development strategy are listed below.

- The wood and paper sector will be even more important in the future for the well-being of Lapland, and it is essential that investments be made in the roads used for transport by that sector;
- Tourism will increase considerably in the future; and the main road network to the holiday resorts in the north, toward the Norwegian border, to the northwest and to Sweden, and to the east, crossing the Russian border, will get special emphasis;
- Most of the population resides in the triangular area in the southwest part of Lapland; in some sections congestion is already severe, and the road standard does not always meet the national targets set by FinnRA;
- The population of northernmost Europe is considerable, and increased contacts between Norwegian, Finnish, and Russian Lapps have a strong potential and depend on a good west-east road network; and
- The enormous potentials for commerce and tourism between Lapland and the Kola Peninsula put further emphasis on a transborder road network.

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Development of Project-Level Urban Roadway Management System

XIN CHEN, TERRY DOSSEY, AND W. RONALD HUDSON

The second part of the Urban Roadway Management System (URMS), the project-level pavement design and maintenance subsystems, is described. In the design subsystem the AASHTO flexible pavement design procedure is applied to the process of structural design of new pavements and overlay pavements. A linear programming model for obtaining the least-cost solutions for new flexible pavement design problems is included. As a part of the URMS package the design subsystem can automatically retrieve needed information from the network-level URMS. The design subsystem can also be used as a stand-alone program. The maintenance subsystem is an application of a simplified expert system for selecting cost-effective distress repair methods. The simplified expert system is designed in such a way that each distress type is related to a maximum of three variables and each variable is further divided into a maximum of three levels. At least 27 repair methods for each distress type can be defined. Both the design and the maintenance subsystems are easy to learn and use with the graphical user interface.

The Urban Roadway Management System (URMS) is a comprehensive pavement management system at the network level and project level developed primarily for application in small and medium-sized cities. It is a simple, flexible, and user-friendly computer program with a graphical user interface.

As shown in Figure 1 the complete URMS consists of four subsystems: planning at the network level and design, construction, and maintenance at the project level. The objective of the planning subsystem is to identify and select cost-effective Maintenance and rehabilitation (M&R) projects at the network level. The M&R strategies assigned for the candidate projects selected in the planning subsystem are combined into four types: reconstruction, overlay, routine maintenance, and do nothing. The design subsystem selects materials and determines the layer thicknesses for those projects scheduled for overlay and reconstruction. The management of the work zone for the overlay and reconstruction will be included in the construction subsystem. The maintenance subsystem is used to select cost-effective distress repair methods for projects targeted for routine maintenance by the planning subsystem.

The network-level planning subsystem has been documented elsewhere (*1*). This paper focuses on the project-level design and maintenance subsystem. In the design subsystem the two major models are the AASHTO flexible pavement design procedure and the linear programming model for new pavement design. The AASHTO pavement design procedure has previously been described in detail (*2*), and the linear programming model for AASHTO flexible pavement design has also been documented elsewhere (*3*). This paper concentrates on the computer program developed for the design subsystem. The maintenance subsystem is constructed as a simplified expert system for selecting cost-effective distress repair methods. It is designed in such a way that each distress type is related to a maximum of three variables and each variable can have a maximum of three levels. The system can be used as an expert system tool for any user to build his or her own expert system with little knowledge of expert systems.

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DESIGN SUBSYSTEM

The design subsystem consists of four major modules: a data base module, an AASHTO design model module, a linear programming model (LPM) module, and a report module. The data base module stores all of the information related to a flexible pavement design problem. The AASHTO flexible design procedure is used to calculate the structural number of each layer for the overlay and new pavements. The LPM module can find the least-cost solutions for the thicknesses of the three layers to the new flexible pavement design problems. The report module displays and prints the input and output.

Figure 2 shows the data flow diagram for the design subsystem. The design data base retrieves the related information from the planning data base if the design subsystem is used as a part of the URMS; additional data are entered manually. For stand-alone use all of the input data are entered manually.

The program selects the least-cost materials by using the ratio of the layer coefficient multiplied by the drainage coefficient to the unit cost of each material (*3*). The AASHTO model then determines the structural number for the material selected for each layer. For overlay design the surface overlay thickness is determined directly. For new pavement design two procedures are adopted: optimal design and conventional design. The optimal design can determine the optimal thicknesses for each of the three layers (surface, base, and subbase), which minimizes the total construction material cost. The conventional design allows the user to design layer thickness interactively; that is, the user enters one or two of the layer thicknesses and the program calculates the rest. This function provides the user with a useful tool for construction dynamic quality control (*4*) in which the pavement layer thicknesses during construction can be adjusted to meet the design specification.

One of the features of the design subsystem is to import data from the planning subsystem. Those M&R sections selected for overlay (including thin, medium, and thick overlays) will be retrieved from the planning data base and saved to the design data base. The main screen for importing data is shown in Figure 3. All the sections retrieved from the planning data base are drawn in the color-coded map (dotted lines in the map in Figure 3), and the section identification displayed in the left part of the screen is highlighted in the map (not shown in Figure 3). The street names are not shown on the

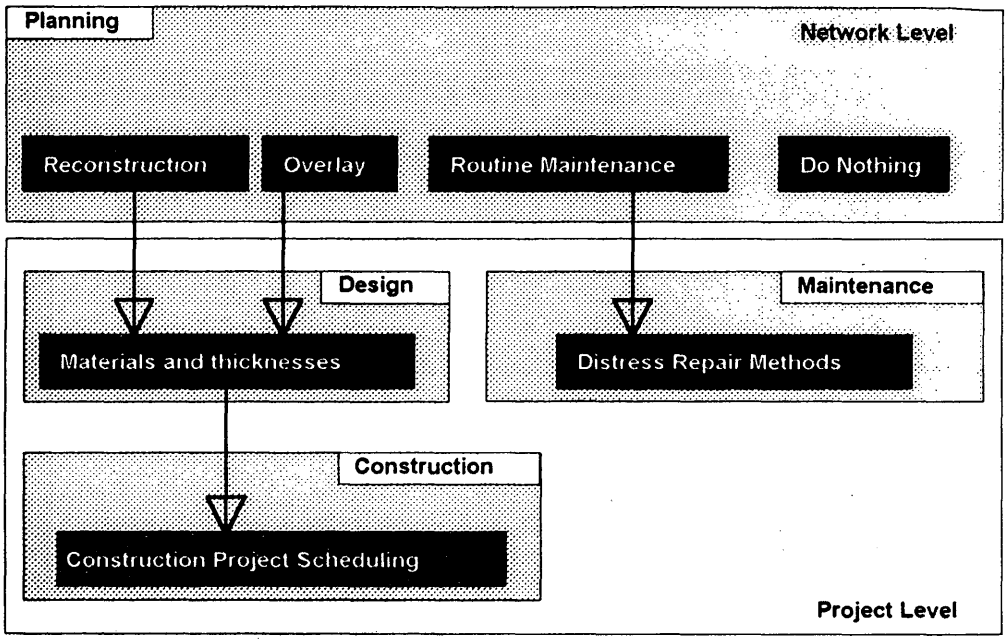


FIGURE 1 Overall structure and data flow diagram of URMS.

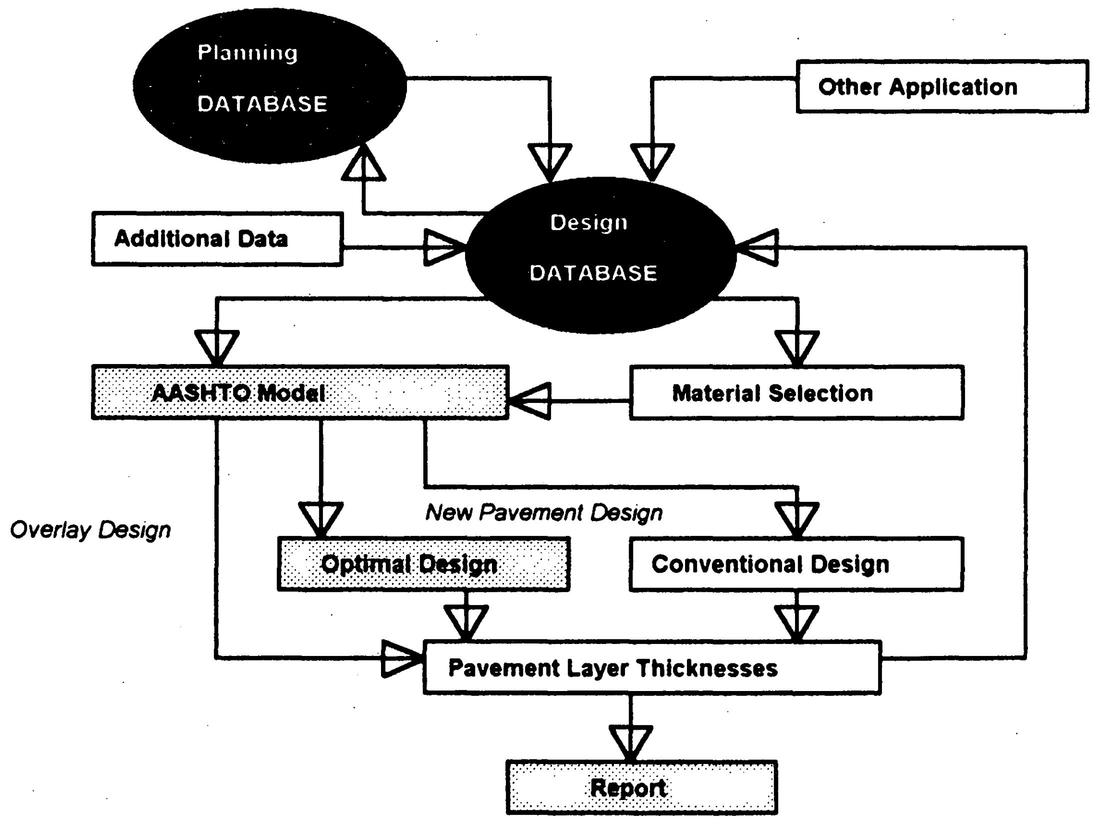


FIGURE 2 Data flow diagram for design system.

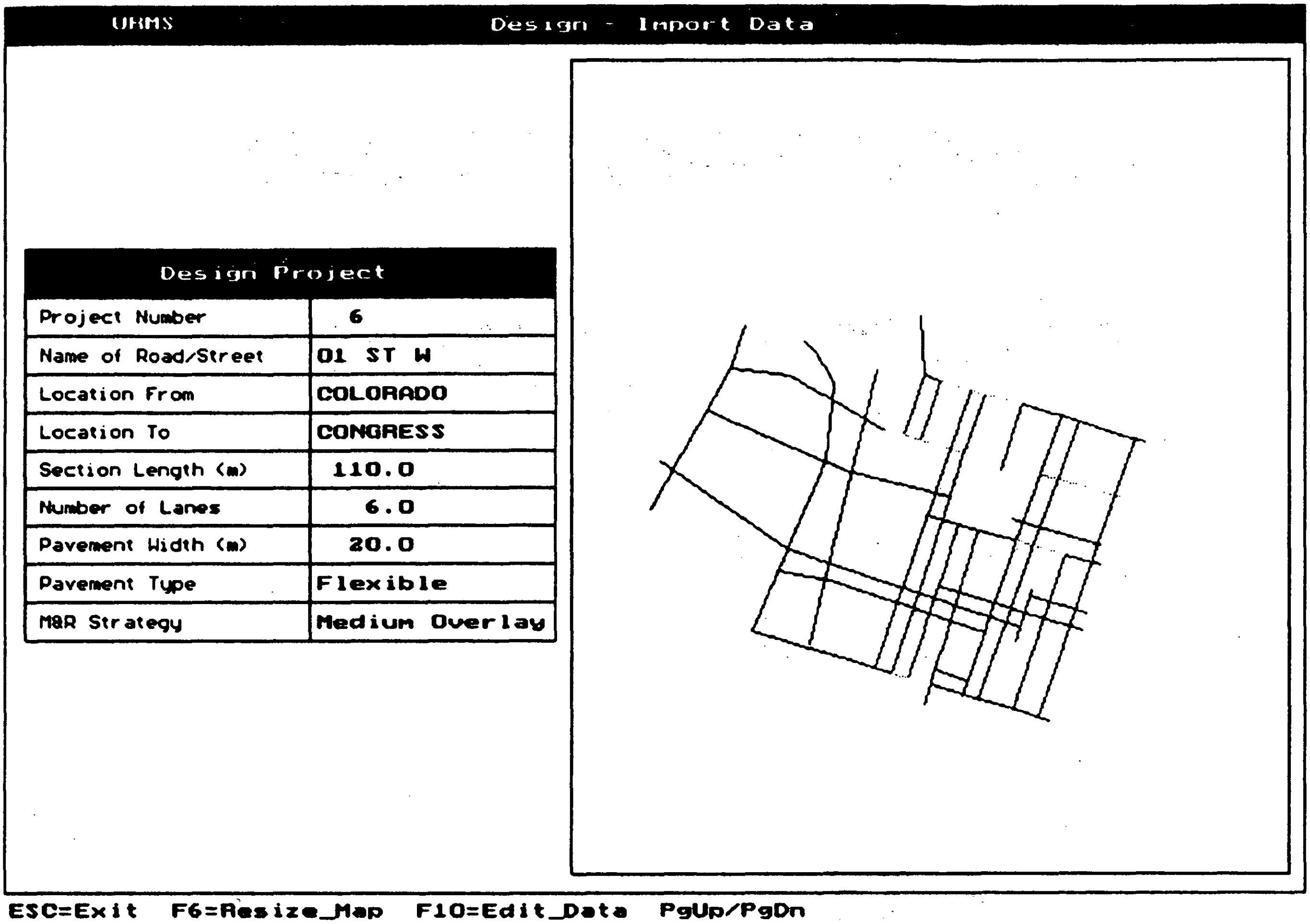


FIGURE 3 Main screen for data import.

map, but those selected for design can be identified in the left box by pressing the page up and page down (PgUp/PgDn) keys. Since the network-level data base does not store all of the information needed for the project-level design subsystem, additional data must be entered by the user for each section. These data are described in detail in the AASHTO guide (*2*) and are tabulated in the URMS user's guide (*5*).

The main advantage of the design subsystem compared with other design programs such as DNPS86 (*6*) is that both the selection of materials and the determination of optimal thicknesses for the pavement structure are incorporated into the subsystem. Up to seven types of material for each layer can be entered.

Figure 4 shows an example for new flexible pavement design. Both the conventional and optimal design results are presented. The scale diagram of the layer thicknesses for each solution is displayed accordingly. The optimal design alternative gives the noninteger optimal solution (the least-cost solution) to the problem, whereas the conventional design gives solutions by taking the user's input into account. The user can enter one or two of the layer thicknesses and the program will compute the other(s). For example, given the thickness of the surface, the program then calculates the thicknesses of the base and subbase courses, or given the thicknesses of the surface and the subbase, the program calculates the thickness of the base course. Because this program uses an embedded linear programming model, the optimal design produces only the noninteger solutions. The noninteger solution is economically correct, but it may not be accepted in practice. This manual design function provides more flexibility to the user in pavement design.

MAINTENANCE SUBSYSTEM

The objective of the maintenance subsystem is to provide the user with a simplified expert system tool rather than an end product expert system for routine pavement maintenance. Since the repair method for a specific distress may vary from city to city, it is impossible to build an expert system for every city without changing the knowledge base. The simplified expert system aims to provide a tool for engineers with little knowledge of expert systems to build their own expert systems without any coding.

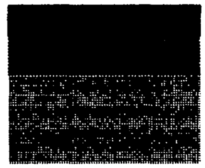
An expert system is a computer program that simulates the thought process of human experts to solve complex decision problems in a specific domain. Expert systems can increase the probability, frequency, and consistency of making good decisions, help distribute human expertise, and facilitate real-time, low-cost, expert-level decisions by the nonexpert. Expert systems are suitable

URMS Design Data Base

New Flexible Pavement Design

Output of New Flexible Pavement Design

Conventional Design			
No.	Description	Material Type	Thickness(cm)
1	Surface	ASPH CONC TY A	20.1
2	Base	AGGR(TY A GR4)	24.9
3	Subbase	AGGR TYPE E	55.2
		Unit Cost($/m*m)	18.4
		Total Cost(1000$)	58.5
Optimal Design			
No.	Description	Material Type	Thickness(cm)
1	Surface	ASPH CONC TY A	30.5
2	Base	AGGR(TY A GR4)	15.2
3	Subbase	AGGR TYPE E	15.2
		Unit Cost($/m*m)	17.8
		Total Cost(1000$)	56.9
Saving of Optimal Design: 1647 ($), 2.8 (%)			



File Name	DEMO.DES
Page Number	7 - 1

ESC=Exit F9=Clear Input F10=Show_Results ArrowKeys

FIGURE 4 Output screen for design subsystem.

for those problems that are well-bounded and focused, not numerically involved, and algorithmic in nature.

Routine pavement maintenance is an important daily task in public works departments. Correct and effective repair of pavement distresses not only can reduce the pavement deterioration rate to save millions of dollars for pavement rehabilitation but can also save vehicle operation costs such as tire wear, vehicle maintenance and repair, and travel time and can possibly even prevent some traffic accidents.

Routine maintenance is a suitable area for the application of expert systems. The major task of routine maintenance is to repair pavement distresses. Pavement repair ranges from crack sealing to full-depth repair. In the selection of a repair method for a specific pavement distress, many variables such as distress type, density, severity, traffic, climate, materials, roadbed soils, and others may be taken into account. There is no quantitative method that can be used to solve this problem. Much is done by intuition and experience. The use of expert systems can help to improve routine maintenance activities.

Much work has been done in the development of expert systems for pavement maintenance (*7–9*). Most applications were developed by using expert system shells such as EXSYS, VP EXPERT, and NEXPERT. Although these shells greatly reduce the time and effort required to build an expert system in a specific domain, they still require training of maintenance technicians or engineers before they can develop their own expert systems.

Since the knowledge bases for most of these shells are built by using text-based rules, modification of the knowledge base requires specially trained personnel. It is desirable to develop a simplified expert system developmental tool so that engineers and technicians with little knowledge of expert systems can build, modify, and expand their own expert systems for selecting effective distress repair methods without any coding.

Design of Expert System Tool

Assume that the repair method for a distress type depends on m variables and that each variable can be divided into n levels; then there will be a maximum of m^n combinations. For some combinations there may be more than one repair method, so at least m^n repair methods for that type of distress can be defined. For example, let m equal 3 and n equal 3; then there will be at least 27 possible repair methods. In practice, for most cases three variables with three levels, giving at least 27 repair methods for each distress, are sufficient (*7,10*).

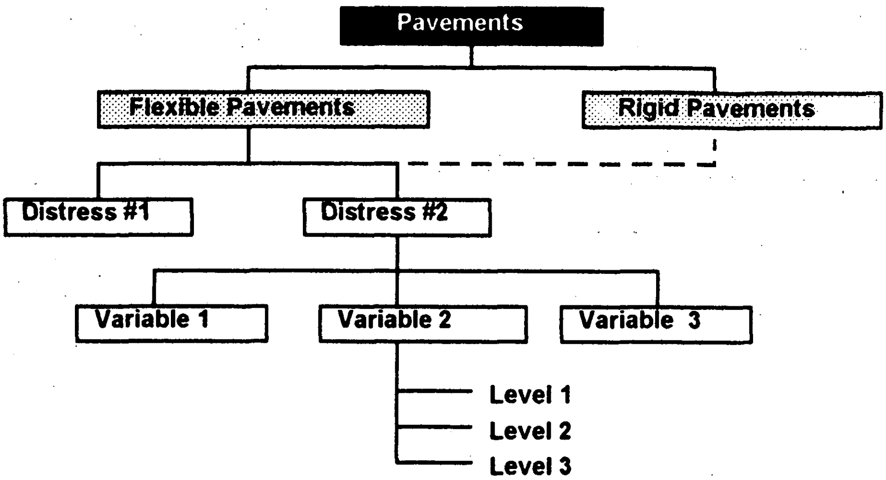


FIGURE 5 Variable hierarchy of knowledge base.

Although the number of variables and the number of levels of each variable can be changed in the computer program without any difficulty, the tool developed in this study considers three variables and three levels for each variable.

Figure 5 shows the hierarchy of the variables and the levels of each variable used to define the structure of the knowledge base. Pavements are divided into flexible pavements and rigid pavements. Distress types are identified for each pavement. A knowledge base is constructed as a random access file. Each distress is a record in the file. Basically, a record is composed of two arrays; one stores the variables, the levels of each variable, and the description of each level, whereas the other one stores the distress repair methods. A record can handle a maximum of 27 rules, as in the case of a text-based knowledge base.

A decision tree is built for each distress type. The structure of the tree may differ from one distress type to another, depending on the number of variables and the number of levels of each variable specified for the distress.

Because the knowledge base is a random access file, all the repair methods related to a distress type are stored in the same record, and the computation is done in random access memory, the size of the knowledge base has little influence on the time spent searching for a conclusion.

Use of Expert System Tool

The simplified expert system tool was developed primarily for selecting pavement distress repair methods, but it can also be used for pavement preventive maintenance and network-level M&R strategy assignment. The process of building the knowledge base is simple. First, the user defines the variables, defines the levels of

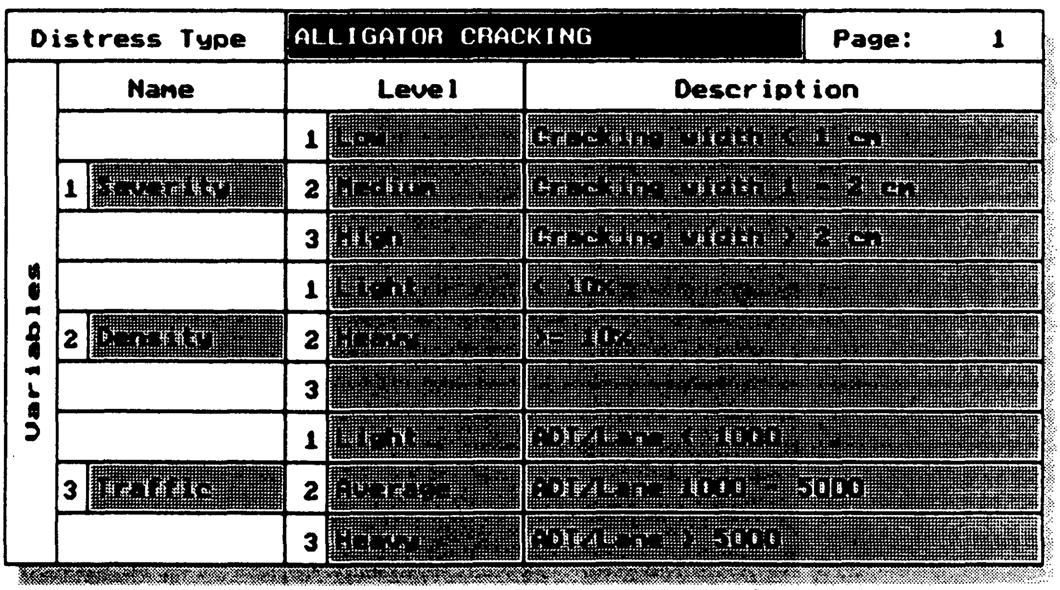


FIGURE 6 Define variables and levels for and describe each variable.

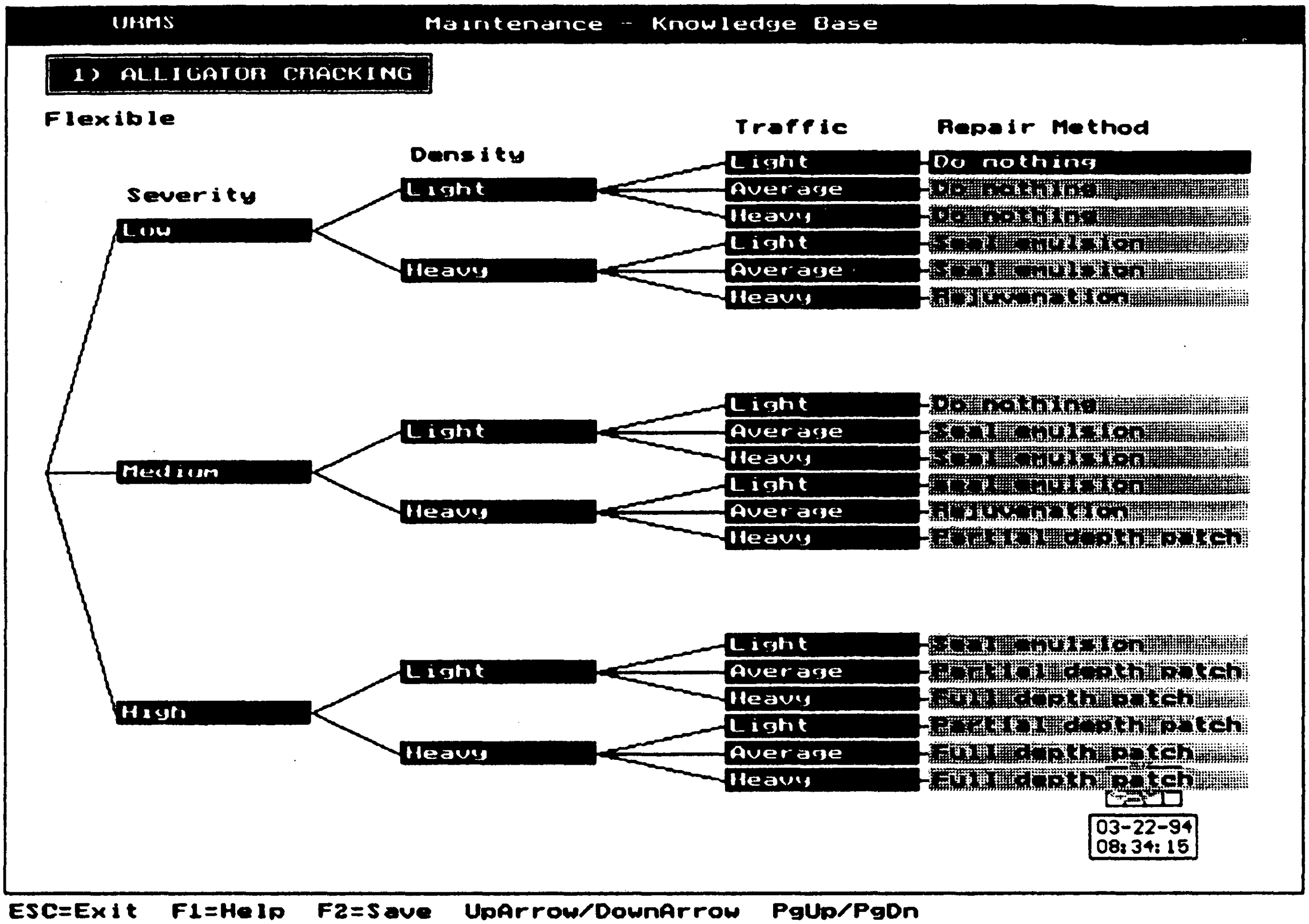


FIGURE 7 **Define repair methods.**

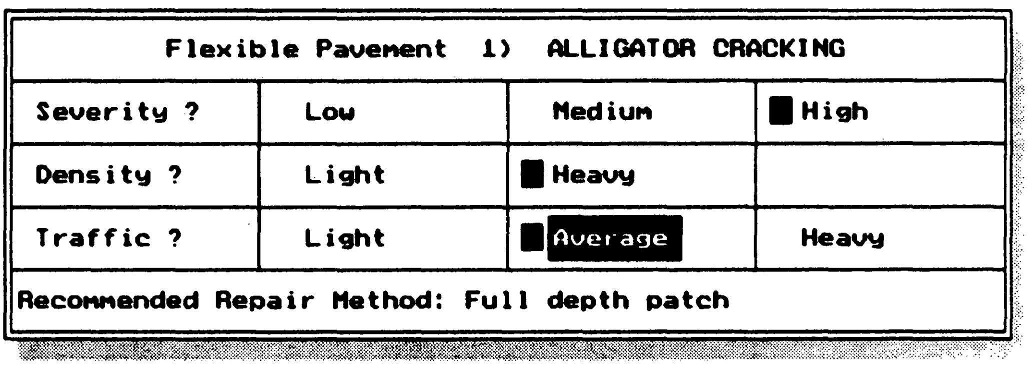


FIGURE 8 **Consultation menu.**

each variable, and describes each level as shown in Figure 6, and then the user specifies the repair method for each variable combination in the decision tree as shown in Figure 7.

Once the development of the knowledge base is completed the expert system is ready and consultation can be conducted. To get the recommended distress repair method the user selects one of the levels of each variable as shown in Figure 8. The recommended repair method is then displayed at the bottom of the consultation box. The tracing of the decision can also be seen in the decision tree.

If there is more than one distress type in the area each distress type is run separately and the distress repair method that is suitable for all of the distress types is selected. There is no priority ranking procedure for prioritizing the distress repair methods in the current version of the program.

The program can convert the decision trees to rule-based knowledge. This option provides a useful method for knowledge communication among expert systems. For example, the rules converted from this simplified expert system can be exported to VP EXPERT or other expert system shells with only a few additional lines of coding.

CONCLUSIONS

The project-level design and maintenance subsystems of URMS are described in this paper. The advantage of the design subsystem is that both the selection of materials and the determination of the optimal thicknesses of pavement structures are incorporated in the subsystem. The advantage of the maintenance subsystem is that the simplified expert system tool can be used to build expert system applications for routine pavement maintenance without any coding. Both subsystems are integrated into the network-level URMS with a graphical user interface, but they can also be used as stand-alone programs.

ACKNOWLEDGMENT

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Proposal of Universal Cracking Indicator for Pavements

WILLIAM D. PATERSON

The progress of automated image analysis technology for road monitoring and the demand for worldwide interchangeability of equipment and predictive models serve as incentives for establishing a norm for harmonizing measures of pavement cracking. A cracking indicator (CI) is proposed. The CI meets critical criteria such as being objective, simple, transferable, and relevant. The CI is defined by the product of the areal extent, intensity, and crack width of a set of cracks in a dimensionless form with a scaling factor of 100,000. It provides a zero-anchored, open-ended integer scale with a common range of three digits and a practical maximum of four digits. With modifying indicators of cracking type (pattern) and location, the CI has an appealing additive and subdivisible property that makes its use highly flexible and simple. The CI is evaluated critically from a number of perspectives, including measurability; relevance to performance impacts, diagnostic analysis, and maintenance decisions; applicability to all pavement types; relation to existing cracking measures; flexibility and discriminatory capability; and suitability to differing levels of application from simple to complex. It is concluded that the proposed CI is sound and powerful in concept, flexible, and broadly applicable, and therefore warrants consideration as a universal norm by standards organizations, highway agencies, and the road monitoring equipment industry worldwide. Several specific points to be considered are identified.

One of the remaining areas in pavement management that is lacking a widely recognized norm for application in procedures and decision support models is that of surface distress and, in particular, cracking. Currently, there is an array of measures evident in practice, most of them specific to particular agencies and most developed around the survey procedure. A leading example for surface distress is the pavement condition index (PCI) (*1*), which is becoming more widely used in North America through the adoption of standard procedures, but other examples of surface distress indexes abound as various agencies have attempted to quantify and weight the amounts of the various modes of distress in a way that is relevant to pavement management decisions. Common to all of these, and ranking as key to most maintenance decision criteria in some form or other, is cracking. But the variety of measures of this key type of distress is so great that the developments in the field of cracking prediction have been hampered by the number of dimensions and by the limited interchangeability of the research findings.

In pavement management there is a primary need for objective measures that are reproducible, transferable, and above all relevant. This is becoming particularly urgent with the evolution from the largely customized individual systems of the past two decades toward generic models capable of being applied in many jurisdictions and situations. A prerequisite for market competition is to have acceptable comparators with which objective comparisons and evaluations can be made. In pavement roughness, harmonization was achieved through an international experiment producing a mathematically based index, the international roughness index, as a norm to which the wide variety of instruments worldwide could be calibrated and as a measure that could be used to express roughness in units that were unambiguous to users anywhere (*2*). In skid resistance and friction, a similar international experiment has been conducted through the Permanent International Association of Road Congresses and ASTM with the purpose of identifying measures that would serve as a norm for harmonizing the very wide range of technologies also now available and using the latest research knowledge to identify the most relevant measure from the complex relationships between the various parameters (*3*). These efforts reflect international standardization and harmonization efforts that are ongoing in North America, Europe, developing countries (through the World Bank) and elsewhere.

It is noticeable that procedural standards become outdated as technology advances. Examples include roughness measures based on the roughometer or bump integrator trailers, skid trailers, and pavement deflection measurements based on the Benkelman beam method, which have had to be reinterpreted in fundamental terms since new and automated technologies have emerged.

Now the automation of surface distress surveys and the improving capabilities of image analysis techniques are shifting the focus of cracking surveys from manually or visually based ratings to more detailed quantitative measurement techniques. Automation is also greatly expanding the scope of what can potentially be measured. For example, although it was once completely impractical to expect field crews to measure crack lengths and spacings, these dimensions will become available as very easy products of automated image processing. The international market for automated surveys also aids harmonization through the rapid spread of acceptable norms that come as standard options on the equipment.

The time is right, therefore, for identifying a universal norm for quantifying cracking. The aim is to define a measure of cracking, which is

1. Objective and universally transferable;
2. Relevant to the various mechanisms and visible evidence of cracking development;
3. Relevant to decisions on maintenance needs;
4. Able to be applied for differing pavement types, environments, and standards;
5. Measurable by a variety of methods at differing levels of detail and precision;
6. Simple in concept; and
7. Progressive, not constrained by present approaches and technology.

World Bank, Washington, D.C. 20433.

CHARACTERISTICS OF CRACKING AND EXISTING MEASURES

Cracking is characterized by five attributes, namely

1. *Extent,* being the area of pavement covered by cracking technically defined by the perimeter bounding all of the area covered by a set of cracks and expressed in units of either area or percentage of total area of pavement;
2. *Severity,* being defined usually by the average width of crack opening at the pavement surface and expressed in terms of either level (high, low or wide, narrow, etc.) or the average width dimension itself (e.g., millimeters);
3. *Intensity,* being the length of cracks per unit area expressed, for example, in meters per meter squared foot (per foot squared) and sometimes alternatively expressed as crack spacing (e.g., for transverse cracks);
4. *Pattern,* representing the orientation and interconnectedness of the cracks, usually expressed by cracking type (alligator, block, longitudinal, transverse, D-cracking, etc.); and
5. *Location,* identifying which part of the pavement is cracked, for example, wheelpath, between wheelpath, edge, joint, midslab, or random.

Various combinations of these attributes are used in the existing measures of cracking. Major examples are summarized in Table 1, and most other measures are variants of these main types. The Texas Department of Transportation (DOT) score (*4*) combines the evaluation of extent and severity into four classes. The most traditional method in the United States is the AASHO method (*5*), which indicates the measured extent of cracking in each of four classes of severity defined by practical ranges of crack width. This has been followed in other major studies such as the Brazil–U.N. Development Program–World Bank study (*6*) and certain pavement rehabilitation design methods. The standard defined in the Strategic Highway Research Program (SHRP) distress manual (*7*) requires three parameters, namely, type, severity level, and extent. None of the standards utilizes all attributes. The inherently most comprehensive is the cracking component of the PCI, which rates intensity and width together under the severity attribute, weights the score according to type, and has the capability of being defined by location through the user's choice of sample areas. However, currently, the cracking component is not used separately from the PCI.

The difficulties with these existing methods are due in part to the simplification that has been adopted to make them practical and in part to the particular application for which they were intended. The score system provides discrete steps so that a natural progression is muted, making it difficult to detect an underlying trend for forward prediction and masking both the attributes and small changes of the cracking. The AASHO classification yields not a single indicator but four indicators, which complicates the task of developing prediction models, and it contains no measure of intensity so that cracking can continue to intensify within the cracked area and the indicator shows no change (except insofar as the severity classes may change).

CONCEPT OF PROPOSED INDICATOR OF CRACKING

Basic Cracking Indicator

The basic cracking indicator (CI) proposed here is the simple product of the three primary physical dimensions of the amount of cracking, that is

TABLE 1 Some Existing Measures of Cracking

Measure	Definition	Agency or Method
Cracking Score	Area score (0-0.50 for 0 to 30 percent extent) + Severity score (0-0.50 for rating "None" to "Severe")	Texas DOT *(4)*
Extent by Severity Class	Percentage pavement area (by worst severity in area): Class 1 = cracks less than 1 mm (1/16 in.) width Class 2 = cracks 1-3 mm (1/16-1/8 in.) width Class 3 = cracks more than 3 mm (1/8 in.) width Class 4 = spalled cracks of more than 3mm (1/8in.) width	AASHO *(5)* World Bank *(6)*
Intensity	Measured crack length in a 1 m square frame averaged over 10 frames in the pavement area.	TRL, for Kenya, etc.
Extent by Type and Severity	Extent (sq.ft or sq.m) and severity level [High, Medium, Low as f (type)], by Type	SHRP-LTPP *(7)*
Pavement Condition Index (part)	Areal extent for 3 severities (L, M, H) (combining crack width and intensity) factored by Deduct Values (dependent on crack type, extent, severity, and pavement type), normalized as portion of 0-100 score.	PCI *(1)*

$$\text{CI} = \text{extent} \times \text{intensity} \times \text{crack width} \quad (1)$$

where

CI = cracking indicator (dimensionless);
extent = area of cracked pavement defined within a sample area, which may be the perimeter bounding a set of cracks, expressed as a percentage of total pavement area;
intensity = total length of cracks within the area defining the extent (expressed in m/m^2); and
crack width = mean width of crack opening at the surface of a set of cracks (expressed in mm).

This formulation of a CI uses the openness or aperture size of the pavement surfacing caused by cracking as the objective function. This is a measure of the surface integrity due to cracking that relates to the area that could potentially admit surface water. It also relates to the loss of horizontal tensile integrity at the surface because that is proportional to the total length of cracks (extent × intensity) and to interparticle load transfer, which is related in part to crack width.

It is, however, a purely physical dimensional measure and has no prior dependency on cause or type of cracking. Any evaluation of the cause of the cracking, or of its impact on pavement performance, would be made independently together with other factors relevant to the particular purpose, as elaborated later. For these reasons no weighting factors are applied in the CI.

The indicator is dimensionless and contains two scaling factors, namely, 100 (percentage area) and 1,000 (mm/m). It would have the same value when applied in either metric or imperial units (e.g., when crack width was expressed in thousandths of an inch and intensity in in./in.^{2}).

Modifiers of Basic Indicator

For the interpretation of cracking data information on the type (or pattern) and location of the set of cracks is useful. These parameters are used in diagnosing the mechanism causing the cracks, in determining the urgency and type of maintenance that are warranted, and in determining the quantities of maintenance and repair. For these reasons, the CI is modified by two supplementary measures, that is, type and location, as follows.

Type

There are two alternatives for presenting type information. In the general method, the *dominant type* (alligator, longitudinal, etc.) observed in the cracking set is stated, and no reference is made to the other types of cracking that may be present. This is a practical device that minimizes detail and provides only the key information, and it suffices for most applications.

In the detailed method the total amount of cracking can be subdivided into as many *types* as are present or desired. Thus, for example, a pavement with CI of 3,200 may comprise 2,000 alligator, 700 longitudinal, and 500 irregular cracking. Note that the numbers are purely additive since the individual areas of each type add to the total amount of cracking, even when the areas are overlapping.

Location

Likewise, location information can be given either generally, as the *dominant location* (e.g., in wheelpath), or in detail (e.g., between kp5+300 and 5+500, Lane 1). Again, in the detailed method, the location-subdivided parts simply add to the total CI, for example, for a CI of 3,200, 2,300 in the outer wheelpath and 900 not in wheelpaths.

Type and Location

Clearly, a combination of these modifiers gives the most detailed information of all, for example, 2,000 alligator in outer wheelpath, 300 irregular in outer wheelpath, 200 irregular not in wheelpaths, 200 longitudinal on centerline, and 500 longitudinal at edge, for a total CI of 3,200. This simply subdivisibility and additive property is a key appeal of the CI indicator.

Computational Example

By way of illustration of the computation of the CI, Figure 1 shows a pavement section of total area A containing a longitudinal crack of length l_L and width w_L, alligator cracking of crack length l_A, average width w_A, and bounded extent of area c and a transverse crack of length l_T width w_T intersecting the alligator cracking area.

First, the CI for longitudinal cracking, CI_L, can be computed by various ways, reaching the same result, for example, dividing the section into a subarea a in the vicinity of the crack and the remaining area of section b, we have

$$\text{(a)}\ \text{CI}_L = 100\left[\frac{a}{A}\cdot\frac{l_L}{a}\cdot w_b + \frac{b}{A}\cdot\frac{0}{b}\cdot 0\right] = \frac{100\, l_L\, w_L}{A}$$

or, calculating on the basis of the whole section

$$\text{(b)}\ \text{CI}_L = 100\left[\frac{(a+b)}{A}\cdot\frac{l_L}{(a+b)}\cdot w_L\right] = 100\,\frac{l_L w_L}{A}$$

Second, the CIs for alligator cracking (CI_A) and transverse cracking (CI_T) are

$$\text{(c)}\ \text{CI}_A = 100\left[\frac{C}{A}\cdot\frac{l_A}{C}\cdot w_A\right] = \frac{100\, l_A w_A}{A},$$

and

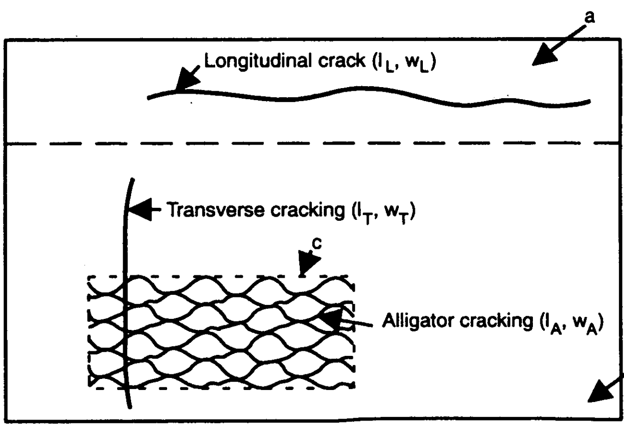

FIGURE 1 Example for computation of CI.

(d) $CI_T = 100\left[\frac{A}{A} \cdot \frac{l_T}{A} \cdot w_T\right] = 100\,\frac{l_T w_T}{A}$

Finally, the aggregate CI is given by the sum of CI_A, CI_L, and CI_T, that is,

$$CI = 100\ [l_L w_L + l_A w_A + l_T w_T)/A$$

It can therefore be seen that it is not necessary for the section's CI to identify or compute the localized extent of any cracking manifestation provided that the crack length has been measured. In practice, with manual methods it is likely that the crack length would be deduced from the localized extent (*c*) and a sample estimate of intensity (e.g., average crack spacing of 80 mm, a local extent of 2.4 by 2.0 m indicates a crack length of approximately $30 \times 2 + 25 \times 2.4 = 120$ m and intensity of $120/4.8 = 25$ m/m^2).

EVALUATION OF PROPOSED CRACKING INDICATOR

In addition to explaining the basis for formulating the indicator, it is necessary to evaluate it critically against the various objectives established for such an indicator to be acceptable and practicable.

Scalar

The indicator is a continuous scale, anchored at zero for nil distress, and is open-ended, which allows continuing and unconstrained discrimination up to any high level. The factors allow the indicator to be expressed always as an integer value, which has some practical advantages for reporting purposes. In practice, the common range is 0 to 1,000, and the likely highest level is about 10,000, for example, 100 percent area, 20 m/m^2 (100-mm spacing), and 5-mm width. The CI could therefore be limited to four digits, that is, 9,999, for practical purposes and would usually comprise only three digits. For simplification, the numbers could be rounded to the nearest tens (e.g., 3,230) or hundreds, but for detailed applications all units would be expressed (e.g., 3,234).

Simplicity

The definition is simple, being based on three readily understood dimensions combined into one indicator. The conceptual basis of total crack openness is also simple and plausible.

Objectivity and Stability

Since the components of the indicator are measurable physical dimensions, it is objective and unambiguous, requires no judgment on the part of the user, and is readily transferable to other users and countries.

Measurability

The three components that make up the indicator are each measurable, either manually or with automated technology incorporating image processing and analysis. They could also be estimated, although clearly with less precision and accuracy than direct measurements.

Direct manual measurement would of course be laborious and would probably be conducted on a sampling basis [e.g. like the approach followed for PCI cracking (*1*)]. However, it is feasible, and that feature is important because it does define the stable reference against which the accuracy of automated methods can be determined.

Automated methods that capture an image of the surface and analyze the cracking detected in the image do so in terms of pixels and gray scales and estimate the connectivity of the cracks through the application of various kinds of analytical logic. From the analyzed image of the scanned portion of the pavement the process is able to determine the *perceived* length and width of the set of cracks "seen" in the scanned image. From a lengthwise evaluation of successive scanned images, the automated method can estimate the areal extent of sets of cracking. If the scanned width is less than the full width of pavement, the image is a transverse sample of the true set of cracks and pavement area.

Visual estimation methods are also feasible. In preliminary field trials it was found that the extent and intensity parameters were relatively easy to determine in a walkover survey of a 10 percent sample of the pavement [10 percent is required to produce a 95th percentile confidence level (*1*)]. It was more difficult to judge the crack width, however, and a 1-mm (40-microns) error on fine cracks of 1 to 2 mm in width results in a 30 to 50 percent swing in the overall indicator. The various solutions to this problem are considered later in the section Measurement Methods. One of these is the alternative visual estimation method, that of identifying the appropriate class or score, which could very easily be related to ranges of the CI, for example, 0, up to 299, 300 up to 999, 1,000 up to 2,999, and 3,000 and above. When made by drive-over (windshield) survey, the lower ranges for fine cracks could not be detected and would only register for wide cracks that are visible from a moving vehicle (in common with all drive-over methods).

Although the accuracies of the various methods may differ considerably, it is clear that measurement or estimation is feasible by all methods and that correlations could be established as required.

Relevance to Technical Impact

As noted in the explanation for the basic concept the CI is closely related to the potential for water ingress and to the loss of horizontal tensile continuity. For a full analysis of the impact of the cracking the CI would have to be supplemented by other relevant information, such as pavement type and materials, environment, and traffic, as with any CI. An advantage of the proposed CI is that there is a clear conceptual linkage and the strong likelihood of a direct functional relationship with such impacts. For example, at a certain CI value, the wetter the environment for a given pavement and traffic the greater the likely water ingress, most likely in proportion to both the precipitation and the CI. It is also apparent that the concept and parameters measured are applicable for all pavement types, that is, flexible, rigid, and composite, even though the interpretation of a CI would differ according to the pavement type.

Relevance to Diagnostic Analysis and Performance Modeling

For diagnosis of the mechanism causing the cracking, the type and location information, in addition to the CI, should also be available.

With this additional detail continuous functions of CI growth in each cracking type can be collected, analyzed, and modeled. The simple additive and subdivisible nature of the CI is of particular value in this respect. Also, it is an ideal parameter for theoretical modeling of crack propagation or progression because it includes all three dimensions of growth: length, width, and area. The observed progression rate will therefore be a continuous function, convenient for modeling. Consequently, it is also ideal for representing the impact of cracking on other pavement performance parameters such as rut depth and roughness. Diagnostic analysis naturally also needs considerable information on the pavement type, materials, environment, traffic, and age, but in each case the CI and its modifiers will have an impact and consequence that are specific to those circumstances. There is no need to require that two pavements with the same CI under differing circumstances behave or perform similarly or have similar priorities for maintenance, which has been the rationale for applying weighting factors in other indexes.

Relevance to Maintenance Decisions

The CI provides a sound and logical way for weighing the overall need for maintenance when there are mixed types of cracking, for example, longitudinal and alligator cracking. The CI maintenance intervention level would be set at differing levels depending on the road class, the environment, the pavement type, and the sensitivity of the pavement materials to water and freezing, where applicable. The CI also relates in direct proportion to the volume of crack filling required, if all cracks were to be sealed, which would usually only apply to those wider than 2 to 3 mm. For determining patching areas, the data on extent would need to be stored and evaluated in parallel with the CI. If considered necessary or desirable, it would be possible to introduce a weighted CI, in which the sub-CIs for each crack type were aggregated with weightings, so as to produce a cracking indicator that fitted some other objective function that was perceived to relate, say, to maintenance intervention urgency.

RELATION TO OTHER MEASURES

Establishing the relationship between the CI and existing measures of cracking must be done with a recognition that existing measures represent only one or two of the physical dimensions of cracking, whereas the CI represents three. Thus, unique relationships do not exist. The relation to the AASHO measures is shown in Table 2 for the two most common AASHO classes, Class II (1- to 3-mm widths represented by an average crack width of 2 mm) and Class IV (widths greater than 3 mm represented by an average crack width of 4 mm). The relation is shown as a domain between boundaries of a typical range of intensity, 1 to 5 m/m^2 (0.3 to 1.5 ft/ft^2). Determining the relationships with other measures that have more subjective content will require specific correlation studies.

From a different perspective the indicator and its components provide a more robust spectrum of attribute detail level (ADL) [a term that is superseding the information quality level (IQL) defined in the World Bank draft guidelines (*8*)] than previously. These are shown in Table 3 from the full detail of ADL-1 to the summary, impact-normalized index of ADL-7. ADL-1 provides the detail of length, width, orientation, and location of each crack or set of cracks. ADL-2 provides the extent, intensity, severity, type, and location information. ADL-3 provides the CI, type, and location or, alternatively, the CI, type, and severity level [for example, like the SHRP–long-term pavement project (LTPP) or AASHO measures]. ADL-4 provides the CI and only the dominant type or dominant location. ADL-5 provides just the CI. ADL-6 reduces the CI from a continuous scale to discrete realistic ranges represented simply by a score. ADL-7 reduces this further by normalizing the CI for impact on performance or maintenance priority to a closed scale number such as 0 to 100 (like for the deduct-weighted component of PCI, for example).

TABLE 2 Relation Between Proposed CI and AASHO Classification

AASHO Measure		CI at given Intensity [1]	
Class II	Class IV	1 m/m^2	5 m/m^2
II-0	IV-0	0	0
II-10	or IV-4%	20	100
(II-30	+ IV-6)	90	450
II-50%	or IV-20%	100	500
	IV-80%	400	2,000

Note 1. Assumes average crack widths of 2 mm for Class II and 5 mm for Class IV cracks.

MEASUREMENT METHODS

The methods of measuring the CI of a pavement fall into four broad categories, with possible subdivisions, which follow in generally decreasing order of accuracy.

1. *Precise absolute measurement.* Measurement of the length, width, orientation, and location of each crack in a set of cracks. Manual measurements are most accurate, with width being measured by a feeler gauge at a series of points within the set and averaged. Automated methods that use image processing have variable levels of accuracy, depending on the reliability of crack recognition and the pixel resolution. Those combining image processing with fine surface profile measurement may have the highest precision and least error.
2. *Sample absolute measurement.* The length, width, pattern, and location of cracking in a sample area of pavement, typically a 2 to 10 percent sample (for 50 to 95 percent reliabilities, respectively), are measured. In manual methods, the precise method (Method 1) would be applied to a series of 1-m^2 frames or lengths of full lane width to amount to the required sample area. In automated methods the scanned width or sequence frequency of successive images represents the sampling pattern that must make up the required amount of sample area.
3. *Visual estimation.* Estimation is made visually of the extent, intensity, and average width of cracking of the pavement area, and the CI is calculated and recorded. Optionally, the dominant pattern (type) and dominant severity of cracking might be noted. The most

TABLE 3 Levels of ADL Applied to Proposed CI

Attribute Detail Level (ADL)	Attributes
ADL-1	Length, orientation, width, location of each crack.
ADL-2	Extent, Intensity, Severity, Type, Location
ADL-3	CI, Type, Severity or CI, Type, Location
ADL-4	CI, Dominant Type (or Dominant Severity)
ADL-5	CI
ADL-6	Score (e.g. 3-6 Discrete CI-ranges)
ADL-7	Normalized index (closed scale, e.g. 0-100; normalized by impact, e.g. through "deduct values").

appropriate mode of measurement is a walkover survey, which may be made on a sample basis (100 m/km) or a full sample, depending on the level of precision required. Experience from the preliminary trials has shown that it is advisable to summarize crack width into one of three distinctly identifiable ranges and not to attempt a more precise estimation, for example, 0.5 mm, 2 mm, and 4 mm for AASHO classes I, II, and III-IV, respectively. A drive-over (windshield) survey has considerably lower accuracy than walkover surveys, particularly when substantial narrow cracking is present.

4. *Score estimation (ranges).* Estimation of the level of CI in three to six ranges, for example, 0, 0 to 100, 100 to 500, 500 to 1,000, and above 1,000 or, alternatively, 0, 0 to 300, 300 to 1,000, 1,000 to 3,000, and over 3,000. Arranging the ranges in geometric progression (multiples of two to five or three) has a sensible relation to the relevance to pavement performance and maintenance decisions and makes the estimation easier and more reliable. This is the quickest and least precise method of all, but it provides sufficient accuracy for many applications.

CONCLUDING COMMENTS

The CI proposed here meets a need for an objective measure of cracking that will utilize the greatly expanded survey capabilities becoming available in automated image scanning and analysis technology. At the same time it can also be measured or estimated by simpler manual techniques, and can thus be related to existing measures of cracking. The multiplicative combination of all three physical surface dimensions of cracking has a rational basis that has been shown to have good scalar properties; strong relevance to the potential for impacts on pavement performance and structural capacity; applicability to flexible, rigid, and composite pavements; simplicity; and a mathematical robustness that makes it readily transferable around the world. In these respects the proposed indicator meets all of the objectives and criteria established at the outset.

The CI also has convenient additive and subdivisibility properties that enable its use to be expanded to show the incidence of individual cracking types or patterns and cracking location. This provides tremendous flexibility for using the CI at differing ADLs to suit particular applications.

There is no presumption that two pavements with the same CI under differing circumstances would behave or perform similarly or have similar priorities for maintenance. The interpretation and impact in each case rest on a diagnosis and analysis relevant to the circumstances and objective.

It is necessary now for the proposed CI and its concept to be critically reviewed and refined as necessary by technical standards organizations, road management agencies, and the road monitoring technology industry. Among the aspects to be considered are the following: (a) the precision and bias (accuracy, repeatability, and reproducibility) of measurements of the CI achievable by various methods; (b) the option for attenuating the rather sensitive impact of crack width on the CI either by adopting nominal values for the primary width classes such as the already familiar classes [< 1 mm (0.5 mm), 1 to 3 mm (2 mm), > 3 mm (4 mm)] or by replacing it with the square root of width, a dimension that may reflect more accurately the potential for water ingress in the presence of surface water because of surface tension, hydraulic flow, and so on; (c) the scaling factors (100,000) incorporated into the definition of the proposed CI and the magnitude of the working range of values; and (d) a name for the indicator that would have international applicability and recognition.

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The views expressed are the author's and not necessarily those of the World Bank organization, its affiliated associations, or member countries.

DISCUSSION

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The author is commended for this timely presentation of a proposed standard of pavement crack data processing. The proposed CI is a very useful concept for the analysis of crack measurements in view of office video/photo image processing and fully automated image recognition techniques that are being implemented in addition to the traditional visual distress survey procedures. The CI formulation is an excellent approach for pavement performance model development and maintenance treatment analysis in pavement management. The ADL provides other important information, for example, the types and locations of cracks. The universal application of CI will require its compatibility with a broadly used and well-accepted pavement distress identification and measurement standard such as the SHRP-LTPP distress identification manual (*7*).

The LTPP distress identification manual (*7*) is being used for the distress measurement of SHRP-LTPP sections to create a national pavement data base that will be used for future performance model development. Visual methods and office distress data reduction of 35-mm continuous photo film are being used for this purpose. This manual is also an excellent universal standard for distress analysis in pavement management applications. A preliminary study of procedures of distress measurement in the LTPP distress identification manual and the proposed CI standard shows the following potential problems that need attention.

PAVEMENT TYPE

The proposed CI approach is valid for all types of flexible and asphalt surfaced pavements; however, it may not be appropriate to combine these pavements with portland cement concrete (PCC) pavements. PCC pavements perform differently from flexible pavements and require different types of maintenance treatments. For example, PCC pavements also exhibit different types of cracking (D-cracking and map cracks). It is suggested that a separate identifier for pavement types may be necessary.

UNIT OF MEASUREMENT AND SEVERITY

The logistic of field measurement or office image analysis requires the rater to establish a scheme of measurement intervals. For example, in image analysis this interval would be an image frame representing a finite length of the section that can be 1, 2, or 3 m, for example, on the basis of the image scale. In the manual field survey, one can make distress measurements in each 6 m (20 ft) of section length, which would be the measurement interval. The measurement interval also facilitates the breakdown of one crack type by severity level. Other distress types are also measured in this manner. It is recognized that crack severity is a function of average crack width as well as that of the condition of spalling. It is suggested that the CI for each cracking type should be reported at each severity level by modifying the attribute detail level ADL-3. It will be essential for performance modeling and maintenance treatment selection analysis.

EXTENT AND INTENSITY

Another potential problem is the discrepancy in actual units of measurement for different cracking types. For example, some distresses (alligator cracking, block cracking) are measured in units of the area of extent, but other distresses (longitudinal and transverse cracking) are measured in linear units only. There is an obvious problem in describing *intensity* for alligator cracking. Probably, *extent* for longitudinal and transverse cracking can be described by assuming an average affected width of 15 cm (6 in.) or 30 cm (12 in.).

FINAL REMARKS

It is suggested that CI should be defined as *cracking index*. The author has certainly taken a lead in the formulation of CI for the benefit of the pavement community in the areas of pavement performance research and pavement management. It is recommended that an expert task group be established within the relevant TRB technical committees to establish a final format for the proposed CI. Perhaps a standard for deformation index should also be developed for uniform reporting of rutting, depression, faulting, and shoulder drop-off distresses.

AUTHOR'S CLOSURE

The support of the CI concept is welcome, and the discussion raises useful points.

First, a separate identifier is not considered necessary to account for differences between flexible and rigid pavements, because pavement type is not an attribute of the cracking but a separate attribute that is brought into any interpretation of the significance or impacts of cracking, just as traffic, age, or environment parameters would also be needed. A cracking type that is distinctive of a particular pavement type, such as D-cracking in rigid and composite pavements, would invariably be associated with those pavement types so an additional classification would be superfluous. It deserves stressing again, perhaps, that the proposed CI is a quantitative measure and not an interpretative one, by design, because interpretation is considered an analytical function that can be applied at varying levels of decision logic and is necessary but separate from the direct mensuration process. Such interpretation may also be contentious, would delay consensus building, and is unlikely to have the universal applicability desired for a standard measure.

Second, the selection of sample intervals will depend in part on the method of measurement and processing and in part on the amount of detail desired on the variation of cracking along the road. A small sample interval could be used to identify specific localized repair needs or local weak areas, whereas aggregation over longer areas would be used to identify the extent of particular treatment types. As shown in the example, the CI is normalized by the sample area and is thus independent of the sample interval except insofar as the incidence of cracking varies spatially along and across the road. The suggestion of introducing severity as one of the attribute levels at ADL-3 is good, and Table 3 has been revised to include that as an option.

Third, the definition of CI handles both linear and areal cracking features, without modification, as illustrated in the example.

Finally, it would clearly be useful in the SHRP-LTPP data base to have an index such as CI in addition to the detailed attribute measures. This would provide a uniform single-dimension measure of cracking that would be useful for both statistical reporting and model development.

Wisconsin's Pavement Management Decision Support System

PHILIP DECABOOTER, KAREN WEISS, STEPHEN SHOBER, AND BILL DUCKERT

The development of a comprehensive pavement management system (PMS) began in Wisconsin in 1987. Since its inception the objectives of the PMS have continued to evolve. Wisconsin has a geographic information system-based PMS that provides needed spatial and mapping capabilities. The backbone of the system employs pavement inventory data and a decision support system to develop improvement/maintenance programs. The system also provides a data base for complex pavement modeling efforts as well as network (statewide) planning efforts. Wisconsin's PMS is an expert system incorporating the knowledge and wisdom of Department of Transportation engineers/practitioners into decision rules for problem definition, treatment selection, and prioritization of projects/programs. First, the systems logic determines the problems associated with each pavement section (nominal 1 mi in length) and suggests a range of treatments to repair all of the problems noted. Highway emphasis levels that give more intensive treatments to the higher-emphasis routes are assigned. The pavement sections are then aggregated into improvement sections (a section whose length is generally more typical of improvement or maintenance projects), with low-, nominal-, and high-level treatment strategies recommended for the entire section. The final treatment selected is based on the relative impacts of these five factors: improvement in ride, improvement in distress rating, user inconvenience, initial cost, and life cycle cost. The final step takes all projects with their final treatment selections and places them into priority order by using the five factors listed above plus a determination of the remaining service life. The ultimate product is a recommended 6-year improvement program and a 3-year maintenance program.

The elementary principles of a pavement management system (PMS) have historically existed in Wisconsin in one form or another. For decades a group of experts in each district would annually evaluate pavement conditions (using their own rules and methodology), propose corrective treatments, estimate the associated project costs, and finally place the projects in a priority order. There was little uniformity in rules and methodology from district to district, consistency from year to year was not assured, and there were no objective measures of pavement conditions. In the 1970s ride data began to be collected, providing at least one needed measure of objectivity for the process. In the early 1980s a consistent, reliable, statewide pavement distress (condition) survey was added to the process and greatly advanced the state of the art. In 1987 a formal PMS began to be developed. The resulting system is well-documented; is designed to be used uniformly and consistently within the state; employs objective performance measures and expert system logic; and develops treatment strategies, costs, and prioritized pavement improvement/maintenance programs. Since 1987 the objectives for the PMS have continued to develop, including the objective of providing a network (statewide) analysis capability to maximize overall pavement performance within the constraints imposed by funding levels.

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This paper concentrates on that portion of the PMS that leads to the prioritized improvement/maintenance programs (the network capabilities are still rudimentary and are not discussed in any detail).

OBJECTIVES

There are numerous objectives for implementing a PMS for the Wisconsin Department of Transportation (WisDOT). These objectives range from network-specific (statewide) to project-specific analysis capabilities, for example,

1. To provide statewide planners and programmers with a way of analyzing the impacts of different treatment strategies.
2. To support upper management planning and programming decisions with objective data and expert system analyses regarding pavement condition and proposed treatment strategies.
3. To provide planners and programmers with prioritized listings of pavement projects, that is, a 6-year improvement program and a 3-year maintenance program.
4. To assist pavement designers in obtaining basic pavement condition and cross-sectional data.
5. To assist pavement management/pavement structural design staff in developing models of pavement performance.
6. To assist DOT engineers in developing treatment strategies based on the field performance of pavements.

APPROACH

To achieve the objectives outlined above, two steps were taken within WisDOT. First, a Pavement Management Unit was created to collect, manage, analyze, and report pavement condition data. Second, a decision support system [the Pavement Management Decision Support System (PMDSS)], which could use inventory data and decision logic to provide the backbone of Wisconsin's PMS, was developed. This system was designed to provide reasonable and reliable solutions to pavement condition problems regardless of the background or experience of the end user. The solutions selected by this system had to be consistent with both current engineering practice and WisDOT policy.

To provide the "reasonable and reliable solutions" discussed, PMDSS was formulated as an expert system. In other words the collective wisdom of DOT engineering practitioners was (and is) periodically sampled and encoded as decision rules for problem definition, treatment selection, and project prioritization (program development).

A further requirement relating to the development of PMDSS was that it be based on a geographic information system (GIS). GIS is able to provide several unique features that could not be provided with a standard relational data base. First, GIS can use spatial analysis routines to integrate pavement inventory, performance, and rehabilitation history data. Geographic locations of these different data elements can logically be linked together and combined for subsequent analysis by the PMDSS user. Second, the interactive graphics and display provide the user with an easy way to review and interpret complex data relationships. Third, the display and cartography tools can be used to show or map inventory data, proposed improvements/maintenance programs, and so on—a host of possibilities.

SCOPE

PMDSS performs analysis on all pavement sections that are part of the state trunk highway system, including all Interstate highways. The only restriction that exists is that, to be analyzed, the pavement section in question must have both a current pavement distress rating and a ride rating. The ride rating is not collected for many pavement sections in urban areas and therefore is not analyzed by PMDSS.

GENERAL OVERVIEW

From its earliest conception through the current implementation PMDSS has been viewed as a way to augment instead of replace the professional judgments of the planners, analysts, and engineers who use it. The distress assessments, problem identification, and rehabilitation recommendations made by the system represent the combined experience with pavement performance and rehabilitation of a diverse group of WisDOT highway engineers. As a result its knowledge base provides a consistent, uniform approach across many individual and organizational boundaries to ensure that departmental goals, priorities, and objectives are achieved.

PMDSS combines WisDOT's pavement inventory data, consisting mainly of pavement distress data, pavement ride data, pavement age, and pavement type, with a knowledge base consisting of rules for distress evaluation, problem identification, and rehabilitation recommendations. Because the knowledge base was compiled by using the expertise of WisDOT's highway engineers, the rules that make up the knowledge base reflect the department's current practices in pavement management and can be modified as those practices change. The results obtained from PMDSS should be comparable to what the user would get when using engineering judgment combined with departmental policy. It is important to emphasize that any treatment recommendation produced by the system can be overridden if local experience contradicts PMDSS logic.

The current version of PMDSS was constrained from dealing with problems of project-specific pavement design. In other words the system may tell the user that an overlay is needed, but the thickness of the overlay will not be specified.

CONCEPTS AND LOGIC

The following discussion provides an overview of the PMDSS concepts and logic.

Data Base

PMDSS uses six main data elements to perform its analysis. These six elements include the individual distresses that make up the pavement distress index (PDI), the PDI value itself, the pavement serviceability index (PSI; which is WisDOT's measure of ride), emphasis of the pavement, pavement type, and pavement age.

The primary analysis unit used by PMDSS is the pavement section, an approximate 1-mi stretch of pavement used to collect ride (PSI and international roughness index) and distress (PDI) observations. The ride and distress observations, the pavement section locations, and the pavement section types and ages are maintained in pavement information files (PIFs). The locations of these pavement sections remain fixed over the life of the pavement structure so that historical pavement performance data can be analyzed. The first step when using PMDSS for analysis is to update the decision support data base from the PIF system.

Emphasis Level

One data element that is used by PMDSS yet that is not stored and maintained in PIFs is the emphasis level assigned to a particular segment of highway. Those pavements with higher emphasis levels will have much higher performance expectations, and subsequently, more intense treatments will be recommended for them than for those pavements with a lower emphasis level. Emphasis level plays a role in (a) determining which pavement performance thresholds are used, (b) analyzing life cycle costs, and (c) determining what treatment to assign to a pavement section. Because emphasis level cannot be determined by some simple formula, each transportation district assigns the highway emphasis level locally. PMDSS uses three categories of emphasis: high, regular, and low.

High-emphasis pavements are those that, because of their relatively high levels of importance or traffic volumes, warrant a sustained high level of pavement quality and particular attention to minimizing user inconvenience.

Low-emphasis pavements are those that, because of their relatively low levels of importance or traffic volumes, are unlikely to be candidates for either geometric improvement or complete pavement reconstruction in the near future. These pavements can generally be preserved by maintenance activities. Low-emphasis roads include, at a minimum, all roads classified as collectors.

Regular-emphasis pavements are those that are not classified as high- or low-emphasis pavements.

Assessment of Pavement Distress

After updating the PMDSS data base with the PIF data, the distress observations, for example, cracking, rutting, and faulting, for each pavement section are assessed. This assessment involves taking the field observations of a distress and assigning a PMDSS severity. Tables such as the one shown in Figure 1 are used. Field observations generally involve noting both the severity and the extent of each distress. By using Figure 1, if the distress survey had noted alligator cracking with cracks of greater than ½ in. in width and covering 80 percent of the survey area, PMDSS would say that the pavement section had *severe* alligator cracking. This same procedure is followed for every distress on every pavement section.

Ride observations, or PSIs, are also assessed and assigned a ride quality level of satisfactory, questionable, or unsatisfactory according to set PSI threshold levels (Figure 2). Again, these threshold levels differ by highway emphasis. These ride assessments are later used by the system to define problems and to determine appropriate treatment levels.

EXTENT

SEVERITY	1-24%	25-49%	50-74%	75-100%
< 1/2 in	MINOR	MODERATE	MODERATE	SEVERE
> 1/2 in	MODERATE	MODERATE	SEVERE	SEVERE
Dislodgement	SEVERE	SEVERE	SEVERE	SEVERE

FIGURE 1 Alligator cracking extent assessment.

PSI VALUES

EMPHASIS	Satisfactory	Questionable	Unsatisfactory
High	5.00 - 3.00	3.00 - 2.25	2.25 - 0.00
Regular	5.00 - 2.50	2.50 - 2.00	2.00 - 0.00
Low	5.00 - 2.25	2.25 - 1.75	1.75 - 0.00

* PSI Scale is from 0.00 (worst possible) to 5.00 (best possible)

FIGURE 2 PSI threshold levels*.

Identifying Pavement Problems

The conceptual core of PMDSS revolves around establishing the nature of the pavement problem. Examples of problems would be "cracking due to pavement aging," "distortion due to use over time," and "slab break-up in jointed plain concrete pavement." Ten pavement problems are defined for asphalt concrete pavements, and 14 pavement problems are defined for portland cement concrete (PCC) pavements. A list of these problems is presented in Figure 3. An example pavement problem is defined as follows:

Problem Name: Insufficient Structure

We have this problem in an asphalt pavement when the following is true:

longitudinal distortion greater than or equal to MINOR
and/or
rutting greater than MINOR
and/or
alligator cracking greater than or equal to MINOR

As shown by this example, problems are identified by using rules containing specific combinations of PMDSS-assessed distress observations. All problem identification rules are statements designed to include (or exclude) problems on the basis of the assessed pavement indicators. Because the rules are central to the logic of PMDSS, they are embedded in the system and cannot be modified by the user.

In addition to indicating whether a particular pavement section has a defined problem, PMDSS will also determine whether a pavement section is deteriorating at a faster rate than expected. PMDSS will calculate a pavement's apparent age, which is determined by how old the pavement "looks," and compare this with the pavement's actual age. The apparent age is calculated by using a pavement section's current PDI and comparing this against the PDI deterioration model for that pavement type. For example, if a pavement section has a PDI of 80 and according to the deterioration model for that pavement type a PDI of 80 can be expected when a pavement is 21 years old, PMDSS will indicate that the section in question is aging prematurely if the actual age of the pavement is less than 21 years old.

Once the nature of the problem is identified, the severity of the problem is then defined. This severity level is determined by using the problem's decision elements. By using the insufficient structure problem as an example, the decision elements include the three distresses of alligator cracking, longitudinal distortion, and rutting. To determine the severity of a problem, a matrix such as the one shown in Figure 4 is used. Continuing the example, assume that the only distress noted on a particular pavement section is severe alligator cracking. In that case the matrix in the upper left corner of Figure 4 would be used (where it says Decision Element = Severe Alligator Cracking). The distress Longitudinal Distortion runs across the top of the matrix, whereas the distress Rutting runs down the side of the matrix. Because the section in question does not have longitudinal distortion or rutting, the severity of this problem will be determined by going down the 0 column for longitudinal distortion and across the 0 row for rutting. This will indicate a severity rating of *severe* for the problem (S = severe, M = moderate, m = minor, and 0 = none). This same process is performed for every problem on every pavement section. PMDSS will store up to three problems for all the pavement sections, rank ordered in terms of their severities.

Rehabilitation Recommendations

After the problems have been identified for each pavement section, the next step in the decision support process is to recommend a

ASPHALTIC CONCRETE PAVEMENT PROBLEMS

1. Cracking Due to Aging
2. Unexpected Bad Ride
3. Poor Mix - Flushing
4. Poor Mix - Soft
5. Unstable Mix Over PCC
6. Insufficient Structure
7. Unstable Base
8. Distortion Due to Use
9. Poor Aggregate - Asphalt Adhesion
10. Joint Deterioration in Asphalt over PCC

PORTLAND CEMENT CONCRETE PAVEMENT PROBLEMS

1. Faulting on Jointed Plain Concrete Pavement Without Dowels
2. Distressed Joints and Cracks on Jointed Plain Concrete Pavement without Dowels
3. Slab breakup on Jointed Plain Concrete Pavement without Dowels
4. Faulting on Jointed Plain Concrete Pavement with Dowels
5. Distressed Joints and Cracks on Jointed Plain Concrete Pavement with Dowels
6. Slab Breakup on Jointed Plain Concrete Pavement with Dowels
7. Faulting on Jointed Reinforced Concrete Pavement
8. Distressed Joints and Cracks on Jointed Plain Concrete Pavement
9. Slab Breakup on Jointed Plain Concrete Pavement
10. Pavement Deterioration
11. Patching Problem
12. Surface Distress
13. Base/Subgrade Problem
14. Unexpected Bad Ride

FIGURE 3 Pavement problems identified by specific combinations of distress observations.

Decision Element = Severe Alligator Cracking
Decision Element
Longitudinal Distortion

Decision Element Rutting	s	M	m	o
s	s	s	s	s
M	s	s	s	s
m	s	s	s	s
o	s	s	s	s

Decision Element = Moderate Alligator Cracking
Decision Element
Longitudinal Distortion

Decision Element Rutting	s	M	m	o
s	s	s	s	s
M	s	s	s	s
m	s	s	M	M
o	s	s	M	M

Decision Element = Minor Alligator Cracking
Decision Element
Longitudinal Distortion

Decision Element Rutting	s	M	m	o
s	s	s	s	M
M	s	s	M	M
m	s	M	M	m
o	M	M	m	m

Decision Element = 0 Alligator Cracking
Decision Element
Longitudinal Distortion

Decision Element Rutting	s	M	m	o
s	s	s	M	M
M	s	M	M	M
m	s	M	m	m
o	M	M	m	o

FIGURE 4 Example of matrices used to determine severity of a problem.

range of treatments that would repair all of the problems in the pavement section at the indicated severities. This process is performed by using decision tables such as those in Figure 5.

As shown in Figure 5 there are three different treatment tables, one for each emphasis level. More intense treatments tend to be assigned to higher-emphasis pavements.

There are 11 PMDSS-recognized treatments for asphalt concrete pavements and 11 for portland cement concrete pavements. The treatment lists for both asphalt concrete pavements and portland cement concrete pavements are given in Figure 6. These treatments are ordered so that Treatment 1 is the least intense treatment and Treatment 11 (Reconstruct) is the most intense treatment. It is ensured that a higher-number treatment will correct a problem that a lower-number treatment may correct, but not vice versa. For example, if PMDSS suggested Treatment 2 (Thin Asphaltic Overlay) to correct a particular problem on portland cement concrete pavement, it is assured that a higher-number treatment such as Treatment 10 (Rubblize and Overlay) could also correct it, but it would not be necessary to use such an intense treatment.

With portland cement concrete pavements a problem's severity along with the pavement's age are used to determine the range of treatments. With asphalt pavements a problem's severity and ride assessment are used to determine a range of treatments that will correct the indicated problem. Age is the predominant controlling factor for treatment selection for portland cement concrete, and ride is viewed as the predominant factor for treatment selection for asphalt concrete pavements. For example, by using Figure 5, if a high-emphasis pavement had a problem of severe insufficient structure and a ride assessment of questionable, the range of treatments that would correct that problem are Treatments 9 to 11. PMDSS will do this for up to three problems on a particular pavement section. PMDSS will then aggregate these three ranges into one range to be used to correct all the problems in a pavement section.

PROBLEM SEVERITY — HIGH EMPHASIS

RIDE	Min Hi	Min Lo	Mod Hi	Mod Lo	Sev Hi	Sev Lo
Good	0	0	8	7	9	9
Quest	8	7	8	8	11	9
Bad	9	9	11	9	11	9

PROBLEM SEVERITY — REG. EMPHASIS

RIDE	Min Hi	Min Lo	Mod Hi	Mod Lo	Sev Hi	Sev Lo
Good	0	0	0	0	8	7
Quest	0	0	8	7	9	8
Bad	7	7	9	8	11	9

PROBLEM SEVERITY — LOW EMPHASIS

RIDE	Min Hi	Min Lo	Mod Hi	Mod Lo	Sev Hi	Sev Lo
Good	0	0	0	0	7	7
Quest	0	0	7	7	8	8
Bad	0	0	7	7	11	11

Treatment Range

FIGURE 5 Example of matrices used for insufficient structure to determine range of treatments

FOR ASPHALTIC CONCRETE PAVEMENTS

0. Do Nothing
1. Spot Repair
2. Crack Filling
3. Seal Coat
4. Cold Recycle
5. Rut Filling
6. Surface Mill/Mill Ruts
7. Thin Overlay
8. Thick Overlay
9. Partial Mill and Overlay
10. Full Depth Mill and Overlay
11. Reconstruct

FOR PORTLAND CEMENT CONCRETE PAVEMENTS

0. Do Nothing
1. Seal Cracks
2. Thin Asphaltic Overlay
3. Partial/Full Depth Repairs
4. Repair and Grind
5. Repair, Grind and Thin Overlay
6. Spot Replace, Patch, Repair, and Thin Overlay
7. Spot Replace, Patch, Repair, and Thick Overlay
8. Repair, Patch, Crack and Seat, and Thick Overlay
9. PCC Overlay
10. Rubblize and Overlay
11. Reconstruct

FIGURE 6 Recognized treatments for asphalt and portland cement concrete pavements.

Aggregation into Project-Length Improvement Recommendations

Because most construction projects are of a length greater than 1 mi, it is necessary to combine the treatment recommendations for the pavement sections into treatment recommendations for the project-length sections (called *improvement sections*). An improvement section is made up of one or more contiguous pavement sections. All of the pavement sections within the improvement section must be of the same pavement type and approximate pavement age.

The aggregation of pavement section treatment recommendations into improvement section treatment recommendations is done by a process referred to as the *15-30-50 percent Rule.* What the 15-30-50 percent rule does is look at treatment ranges assigned to each pavement section within the improvement section and determine which treatment would undertreat no more than 15 percent of the pavement sections that it comprises, which treatment would undertreat no more than 30 percent of the pavement sections that it comprises, and which treatment would undertreat no more than 50 percent of the pavement sections that is comprises. These three levels—15, 30, and 50 percent—are also referred to as high-, nominal-, and low-level treatment strategy levels, respectively.

The 15-30-50 percent rule is sensitive to the number of pavement sections within the improvement section. Usually, the more pavement sections the less intense the treatment recommended. For example, assume that the recommended treatment for a pavement section was reconstruct. If the improvement section contained just this one pavement section, the nominal strategy treatment level would obviously be reconstruct. As more pavement sections, each with a recommendation of do nothing, are added to the improvement section, the overall recommendation would change to do nothing as soon as the initial pavement section length fell below 30 percent of the total (increasing) improvement section length. That is, less than 30 percent of the total length would be undertreated by the do nothing recommendation.

For each improvement section the low-, nominal-, and high-level treatment strategies are passed onto the final treatment selection and prioritization part of the program.

Final Treatment Selection and Prioritization

This final process involves two steps. The first step determines which of the three treatment strategies proposed should be selected as the final treatment for that project. To accomplish this the user must provide values representing the relative importance of the following five factors: the improvement in the PSI after the treatment has been applied, the improvement in the PDI after the treatment has been applied, how much the treatment will inconvenience the user, initial cost of the treatment, and the life cycle cost of the treatment. When entering the relative importance values, the user must ensure that the total of all values entered is 1.0. So if a user gives equal importance to two of the factors, the relative importance of each would be entered as 0.50. Usually, all five factors are used in some way.

During the treatment selection process a treatment is chosen for each project solely on the basis of the impact that that treatment will have on the individual project. The impact on the entire highway system is not considered when determining the treatment strategy for a single improvement section.

The second step involves taking all the projects, with the final treatment selection already made, and placing them in priority order. Again, the user must provide values representing the relative importance of the following seven factors: improvement in the PSI after the treatment has been applied, improvement in the PDI after the treatment has been applied, how much the treatment will inconvenience the user, initial cost of the treatment, life cycle cost of the treatment, "time to must" for the PDI, and "time to must" for the PSI. "Time to must" is used to define the remaining life of the pavement on the basis of either the PDI value or the PSI value. Remaining life is defined by predicting the time, by using PDI and PSI deterioration models, when a pavement will reach the critical level set for PSI and PDI. These seven factors and their relative importances are then combined into a single prioritization value. The projects are then listed from highest to lowest priority.

Once the projects have been prioritized the program will begin placing projects in either the 6-year improvement program or the 3-year maintenance program, depending on the type of treatment chosen for the section, using the budgetary constraints entered by the user. This output is then used by transportation district planning and programming personnel.

FUTURE PMDSS INITIATIVES

Deterioration Modeling

The present deterioration scheme calculates a "time to must" for PDI and PSI via a linear model. This scheme does not alter treatments in the future years. The scheme should deteriorate the individual distress factors to determine if the treatments change during the future years. To be able to accurately predict problems and treatments past the first year, a deterioration scheme that will predict distresses and their severities over time is being developed. This will give WisDOT practitioners valuable information such as how long a treatment can be postponed on a section before a higher level of treatment is recommended. This information would be used to decide between multiple projects when one must be deferred because of budgetary constraints.

Layer and Base Information

Information about a highway's cross section, along with its treatment history, is stored in the layer and base data base. These data will be used in future studies to determine optimal treatment strategies for pavements by looking at historical pavement performance.

Network-Level PMDSS

To assist statewide planners and programmers a network-level PMDSS is being developed. Because statewide program developers do not need to know about detailed problems and specific treatments, a more simplistic model is being designed for their use. The network-level model assigns a general treatment, such as high-level rehabilitation, to sections of pavement solely on the basis of what the PSI and PDI values are on that pavement. This model will be used to analyze budgetary impacts on the overall pavement performance of Wisconsin's highways and to assess the impacts of treatment strategies on the budget.

Benefits from Research Investment: Case of Australian Accelerated Loading Facility Pavement Research Program

GEOFFREY ROSE AND DAVID BENNETT

Since 1984 the Accelerated Loading Facility (ALF) has played a central role in a major Australian pavement research program. Results from an economic evaluation of this research program are reported. Benefits are quantified in terms of the reductions in road authority costs, reflecting a direct hard dollar return on the research investment. The overall benefit-cost ratio depends on the selected inflation-free discount rate and ranges from 4.0 at 8 percent to 5.0 at 4 percent. A number of significant, but unquantifiable benefits were also identified, including ALF's role as a catalyst for pavement research and field innovation, improved cooperation between researchers and practitioners active in the pavements area, and enhancement of Australia's technical reputation overseas. Overall, the results indicate that a healthy return is being obtained on the investment in this pavement research program.

Since 1984 the Australian road authorities, through the Australian Road Research Board Limited (ARRB) and AUSTROADS (the national association of road and traffic authorities in Australia), have supported a major pavement research program centered around the Accelerated Loading Facility (ALF). ALF (Figure 1) is a relocatable road testing machine that applies full-scale rolling wheel loads to a test pavement. To date eight trials have been completed on different pavement types in four mainland Australian states (Figure 2), and the ninth trial, which is being undertaken for the U.S. Army Corp of Engineers, is currently in progress on the Beerburrum test site in Queensland. Technical aspects of the Australian ALF research program have been reviewed elsewhere, (*1*).

Between 1984 and 1992 expenditures on the program totaled approximately $11 million (1992 Australian dollars), and although there is continued interest in the ALF program, an evaluation process was initiated to assess the benefits obtained from this research expenditure. Addressing the issue of value for money is of fundamental importance to research organizations like ARRB as they strive to provide accountability for their expenditures in response to increasing pressures on their budgets. The aim of this study was to produce a credible, justifiable evaluation of the ALF trials on the basis of the dollar value assessments of their benefits and costs. The study focuses on the first seven trials because it was undertaken while the eighth trial was in progress.

The structure of this paper is as follows. Initially, general issues associated with evaluating research benefits are addressed and then the methodology used in this study is outlined. The evaluation of the individual ALF trials is then described before the evaluation of the ALF program as a whole is considered. Finally, the conclusions are presented.

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METHODOLOGY

The general steps in this, as in other economic evaluations, are to

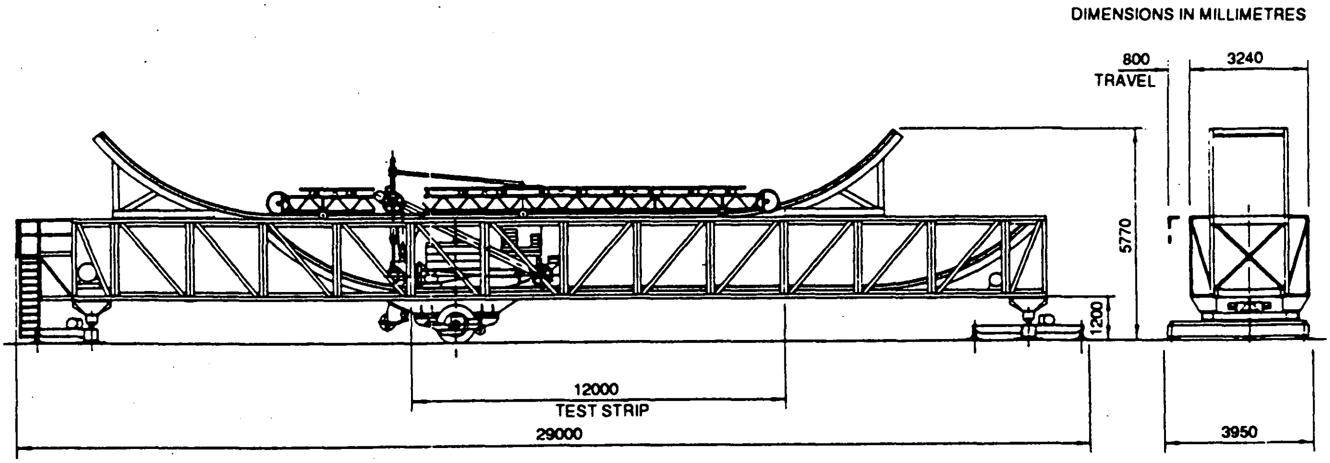
1. Identify the range of benefits and costs associated with the ALF program,
2. Value the benefits and costs in dollar terms, and
3. Determine the values of appropriate benefit-cost criteria.

In a relative sense, costs were easier to quantify than benefits. These costs refer to the actual expenditures involved in conducting each trial. Costs are generally split between the host state road authority (SRA) and ARRB, with the SRA covering construction of the test pavement and staff costs associated with monitoring the test, whereas ARRB costs include those associated with data collection and analysis.

The measurement and dollar valuation of benefits presented the greatest challenge in the evaluation of the ALF program. It is important to appreciate the manner in which benefits are realized. Benefits may be reflected in reduced road authority costs or in terms of reductions in road user costs (Thorensen, unpublished data). (The term *road authority* is used here to cover both state and local government interests.) In the current economic climate, reductions in road authority costs are regarded as the most appropriate measure of benefits. This benefit measure is clearly of interest to road authorities because it reflects a direct hard dollars return on the investment made in pavement research through the ALF program. Although the use of this measure of benefit could be debated, it is important to appreciate that it would not have been feasible to develop justifiable measures of road user benefits given the time frame for this project.

The reference point against which benefits are to be measured must also be defined. As illustrated in Figure 3, the outcomes of the ALF trials, that is, the knowledge that is gained, affect the decisions and practices of road authorities, and these determine authority costs. Because the focus here is on road pavement research, the relevant road authority costs are those that relate ultimately to either the establishment or preservation of roadway assets. The benefit of the ALF trial is then the difference in the authority's costs with and without the ALF trial. These benefits may arise, for example, because an ALF trial results in accelerated introduction into practice of an asphalt additive that reduces maintenance costs or because ALF provides reassurance that a cheaper pavement design will provide adequate service life.

The linkages between ALF trial outcomes and road authority costs are illustrated in greater detail in Figure 4. Here a distinction is made between whether the outcome of the trial affects authority



Test wheel	Dual tyres, 12 - 22.5 Michelin 'X' type, ZA pattern, 16 ply rating, tubeless
Mass of Test wheel assembly	4 to 8 tonne in 1 tonne steps
Suspension for variable mass	Air bag
Power drive to wheel	2 x 11 kW electric geared motors, uni-directional operation, wheel off pavement on return
Transverse movement of test wheel	User programmable
Test speed	20 km/h
Cycle time	9 seconds
Pavement test length	12 metres
Overall length of ALF	26.3 metres
Overall width of ALF	4.0 metres (operating) 3.2 metres (transport)
Overall height of ALF	5.7 metres (operating) 4.4 metres (on transporter)
Total mass of ALF	45 tonne approx.

FIGURE 1 Description of ALF.

decisions/practices in the areas of pavement design, construction, or maintenance. These decisions/practices in turn determine the magnitude and timing of asset establishment and preservation costs. These cost categories should be interpreted broadly, so that, for example, expenses associated with laboratory testing of materials or conventional field trials could be classified as part of the asset establishment cost.

The first step in the assessment of the benefits of ALF is to develop qualitative descriptions of the linkages between each outcome of each trial and authority costs. Some linkages are direct, whereas others are much more indirect. The trial conducted at Benalla, Victoria, illustrates a fairly direct linkage in which one outcome was the confirmation of the strength of a particular pavement design which led to a decision to proceed with that design, thereby determining the construction cost. A less direct linkage is illustrated by the trial conducted at Callington, South Australia, in which one outcome was the cross-referencing, within a short time period, of laboratory tests on materials with their field performance. This calibration of the laboratory tests will enable the field performance of other combinations of materials to be predicted from the laboratory tests. Those subsequent laboratory tests could then influence decisions on the selection of maintenance treatments that could ultimately translate through to authority maintenance costs.

As the linkages between the outcome of an ALF trial and the authority's costs become less direct it becomes more difficult to develop credible and justifiable quantitative estimates of the corresponding authority costs. Comparing the Benalla and Callington examples cited, a broader range of assumptions would have to be made to quantify the final authority costs in the second case. It is important to emphasize that this discussion is focused on a particular outcome of each trial. There is no implication that all outcomes of the Callington trial are difficult to translate through to authority costs.

In keeping with the objective of producing a credible analysis, emphasis has been on quantifying those outcomes of each ALF trial that have fairly direct linkages through to authority costs. These outcomes will lead to benefits that are clearly recognizable and whose magnitudes can be readily justified.

As noted in Figure 3 it has also been necessary to assess authority costs in the absence of the ALF trial to estimate the benefits. This involved consideration of authority decisions/practices and their links to asset establishment and preservation costs in a manner similar to that illustrated in Figure 4. An additional difficulty arises, however, because it is not known for certain which decisions/practices would have been adopted in the absence of a particular ALF trial. For example, in the absence of the ALF trial conducted in Benalla, the Victorian State road authority (VIC ROADS) could have proceeded with the proposed pavement design or selected one of a number of alternative pavement designs. It would be incorrect to simply select the most expensive alternative pavement design and attribute all of the cost savings to ALF.

The methodology recognizes the importance of uncertainty about the outcome, in the absence of the ALF trial, by including it explicitly in the analysis process by using techniques developed in the field of decision analysis (*2,3*). The relative importance of this issue of uncertainty of outcomes in the absence of ALF varies across trials, so further discussion is held until individual trials are considered in the following section.

Another important issue that must be addressed in the context of each trial is over what period of time benefits can be directly attributed to that trial. In some cases an ALF trial may have accelerated the introduction of a new material by 3 to 5 years, whereas in other cases design procedures may have remained unchanged for 10 years or more, and therefore the benefits from the trial are likely to be realized over an extended period. Since each ALF trial is unique they must be assessed individually to determine the time period over which benefits are realized.

Once the benefits and costs are valued in dollar terms, the economic evaluation is relatively straightforward and involves the use of evaluation criteria such as the benefit-cost ratio to compare the magnitudes of the benefits and costs. Results of the economic evaluation are reported for values of the discount rate of 4, 6, and 8 percent per annum. The analysis of each trial is conducted in constant dollars so that these represent real, or inflation-free, discount rates. Given that that average inflation rate over the period of the ALF trials is about 7 percent, these discount rates would be equivalent to market interest rates of approximately 11, 13, and 15 percent, respectively.

EVALUATION OF INDIVIDUAL ALF TRIALS

Within the space limitations of a technical paper it is not possible to fully describe the evaluation of each trial. However, to adequately illustrate the application of the methodology, one trial, conducted at Benalla, Victoria, is examined in detail. The discussion of the other trials is brief, but in each case the primary outcomes of the trial, the nature of the benefits, and the results of the economic evaluation are reviewed. Readers are referred to the final report of the ALF program evaluation (*4*) for additional details.

Benalla, Victoria

The Benalla ALF trial, conducted between June 1985 and February 1986, examined a new section of the Hume Highway before it was opened to traffic. The Hume-Highway is a major interstate highway with a design traffic load of 3×10^7 equivalent single axles (ESAs) that connects Australia's two largest cities: Melbourne and Sydney. A heavy-duty unbound pavement comprising a double seal over 400 mm of crushed rock base and 170 mm of ripped sandstone subbase was tested. The granular material was compacted to a high density, thereby achieving a relatively stiff layer that would be expected to

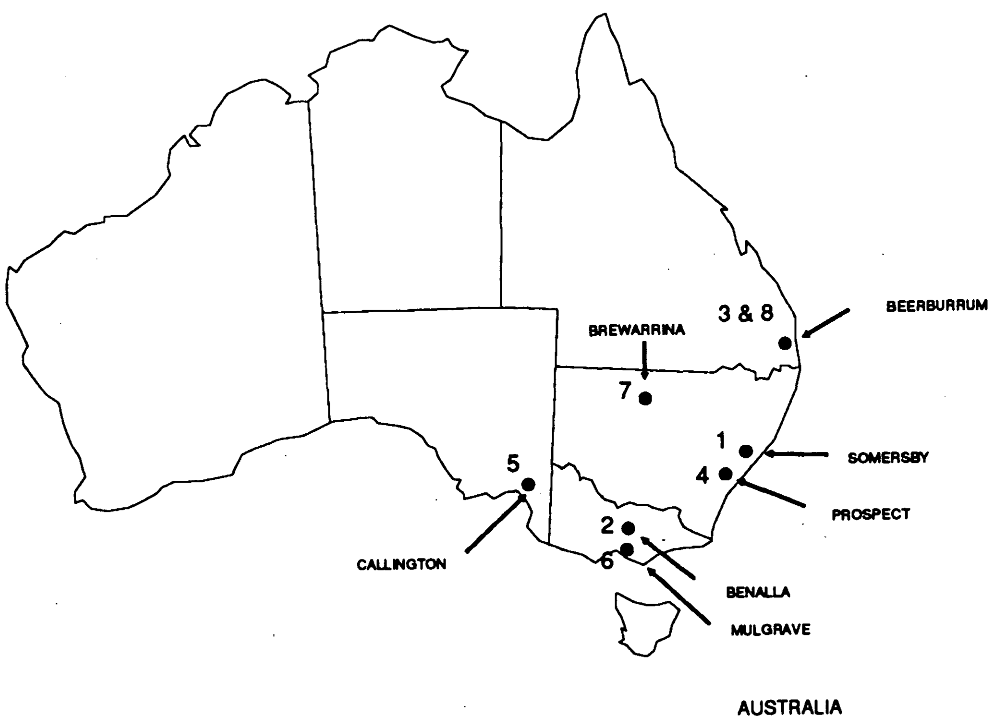


FIGURE 2 Locations of ALF trials in Australia.

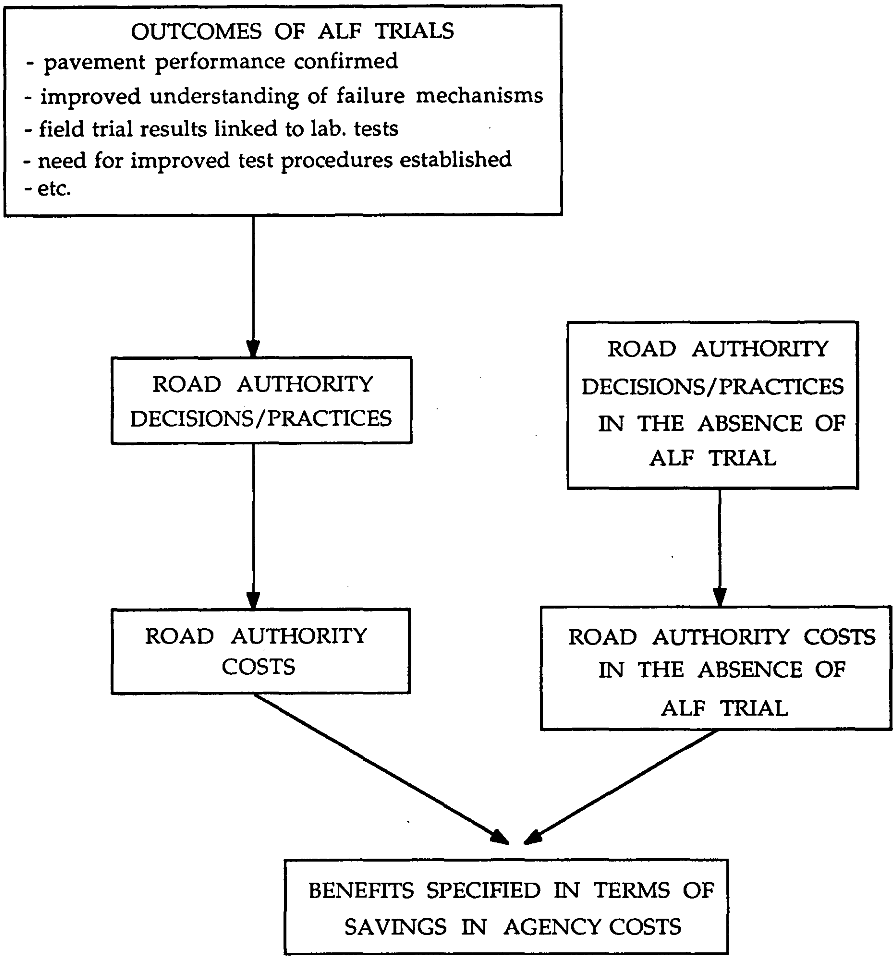


FIGURE 3 Effects of outcomes of ALF trials.

enhance pavement performance. Significantly, the Victorian State road authority has a national reputation for its ability to achieve a high degree of compaction in road pavements. The use of natural materials and a sprayed surface seal challenges overseas experience in which pavements of a different design would be standard for heavily trafficked situations.

This trial was not expected to cause the test pavement to fail. Sufficient load repetitions were applied to prove that the pavement would provide a long service life and to demonstrate that massive maintenance problems would not arise so long as the integrity of the seal was maintained. Fundamentally, then, this trial provided reassurance in the performance of this heavy-duty unbound pavement. This outcome has implications for the cost of establishing roadway assets (Figure 4) because the cost of other proven pavement designs for heavily trafficked situations is generally higher than that for the type of pavement tested at Benalla.

The greatest benefit from the Benalla trial has been realized in Victoria, where extensive lengths of roadway with this heavy-duty unbound pavement have been constructed since the ALF trial. Therefore, additional costs have been avoided by not adopting the more expensive designs.

Within Victoria, the Benalla ALF trial had most relevance to the selection of the pavement for the Hume Highway. Substantial proportions of the Hume Highway were still to be constructed in the decade following the ALF trial. The flexible granular pavement, costing approximately $25/m^2 (1986 Australian dollars) to construct, offered substantial savings over alternative pavement types, with the cheapest alternative adding $15/m^2 to the cost of the pavement. The long-term performance of this pavement was of special concern because Victoria had experienced failures in granular pavements with thin surfacings elsewhere in the state. In contrast to the Victoria practice, New South Wales was constructing rigid pavements on its portions of the Hume Highway. These factors may have created a perception, perhaps at the political level, that the remaining portions of the Hume Highway in Victoria should be constructed by using a rigid pavement.

At the very least there was pressure on the Victoria state road authority to consider other types of pavements for the Hume Highway. The ALF trial confirmed the performance of the granular pavement, and this type of pavement has now been used on the remaining sections of the Hume Highway and on one part of the Calder Highway, northwest of Melbourne, Victoria. It will never be known with certainty what choice would have been made in the absence of the ALF trial; however, there is a real chance that a more expensive pavement would have been selected to reduce the risk associated with the future performance of the pavement. The benefit of the ALF trial to Victoria is the expected savings in costs resulting from continued use of the low-cost granular pavement.

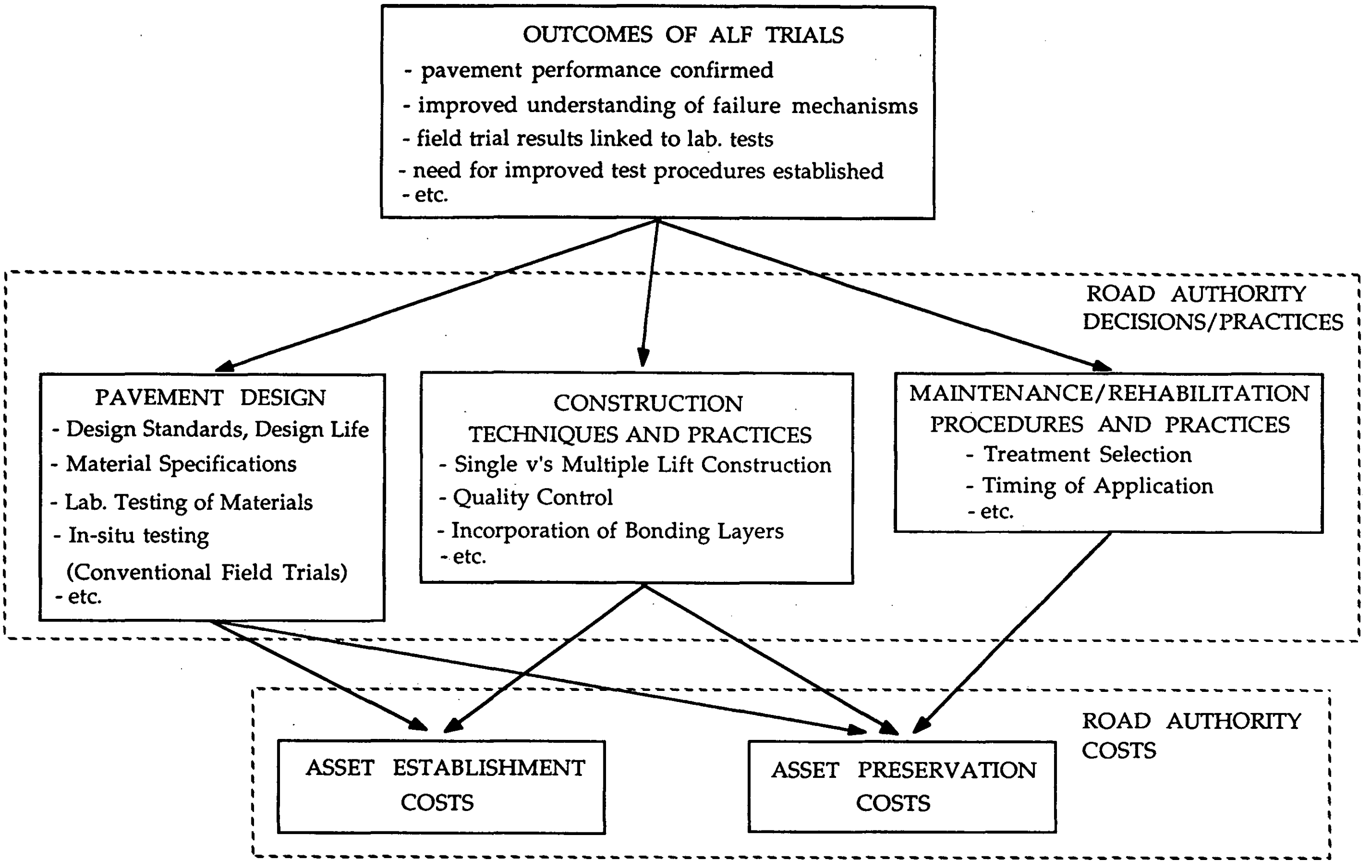


FIGURE 4 Linkages between ALF trial outcomes and road authority costs.

A decision tree representation of the Benalla ALF trial is shown in Figure 5. This decision tree highlights the uncertainty associated with the choice of pavement in the absence of the ALF trial. The options range from constructing an unbound, sprayed seal pavement or an asphalt-surfaced pavement through to a rigid (concrete) pavement. Options connected to a circular, chance node have a probability of occurrence. As will be described, these probabilities were assessed on the basis of discussions with representatives of road authorities.

The ALF trial tested and confirmed the performance of the unbound, sprayed seal pavement, and as a consequence of the trial, VIC ROADS proceeded with this flexible granular (FG) pavement at a life cycle cost of $26/m². (The life cycle cost reflects initial construction, maintenance, and rehabilitation costs over the economic life of the pavement.) Without the ALF trial VIC ROADS could have continued to use the flexible granular pavement, or one of three alternative, more expensive, pavements (Bethune, unpublished data) could have been selected. These alternatives and their associated life cycle costs were as follows:

1. AS, asphalt surfaced, costing $A41/m²,
2. DS, deep-strength asphalt, costing $A46/m², and
3. R, rigid pavement, that is, a jointed unreinforced concrete pavement, costing $A48/m².

Clearly, the flexible granular pavement offers considerable cost savings over alternative pavement designs.

As noted in Figure 5 there is a probability of selecting each pavement type in the absence of the ALF trial. The expected cost of the pavement, in the absence of the ALF trial, is simply the sum of the pavement costs multiplied by their respective probabilities of selection. The benefit, expressed as the saving in cost owing to the ALF trial, is the difference between the pavement cost after the ALF trial and the expected cost in the absence of the ALF trial.

Although hard data were available to quantify costs, the assessment of the probabilities was, of necessity, subjective. Discussions were held with a number of individuals who were working in the pavements area of the Victoria state road authority around the time of the Benalla ALF trial. Very consistent opinions were expressed by these individuals indicating that in the absence of the ALF trial there was still a 70 to 80 percent chance (i.e., $P_{FG} = 0.7$ to 0.8) that the flexible granular pavement would have continued to be used on the Hume Highway. There was a perception that there would need to have been evidence that the unbound pavement "would not work" rather than evidence that the other pavements "would work" for the design to be changed, given its considerable cost savings over the alternatives. In the results presented in this paper P_{FG} was set equal to 0.8. From the discussions with present and former VIC ROADS employees, it seems reasonable to expect that the probability of selecting the other pavement types would decline with increasing cost, and this expectation guided the selection of their probabilities. The other values used in the analysis were P_{AS} equal to 0.12, P_{DS} equal to 0.06, and P_R equal to 0.02. Because the estimation of benefits depends directly on the values of the probabili-

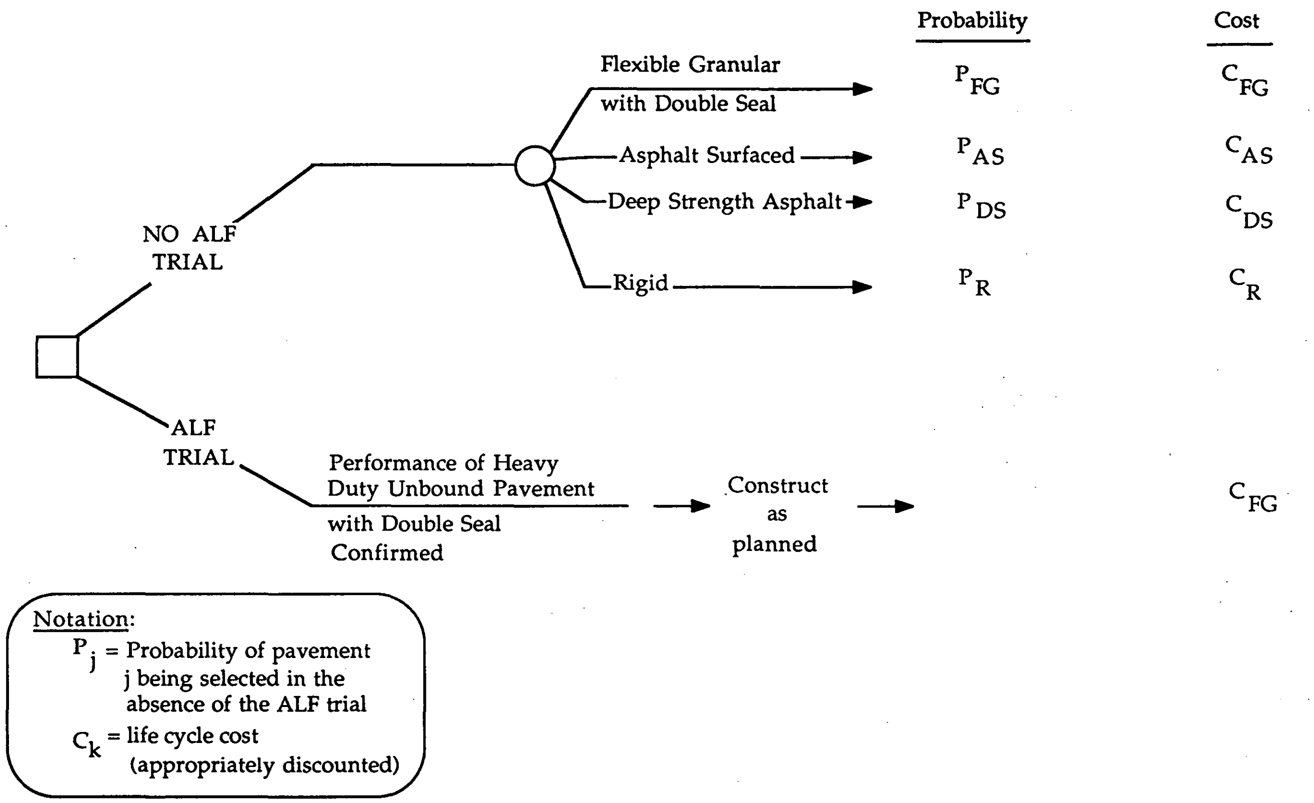


FIGURE 5 Decision tree representation of Benalla ALF trial.

ties, a sensitivity analysis approach was adopted in the study by repeating the analysis with three sets of probabilities; however, in this paper results are reported only for the probability values cited above.

On the basis of the cost and probability data mentioned above the expected cost savings associated with the ALF trial can be calculated at $3.44/m^2 (1986 Australian dollars).

To estimate the total benefits to Victoria of the Benalla ALF trial, the savings per square meter must be multiplied by the number of square meters and discounted to account for the time lag between the ALF trial and the completion of a particular portion of pavement. It is not regarded as appropriate to assume that the Benalla ALF trial would influence pavement choice for an indefinite period. Early sections of the Hume Highway constructed as flexible pavements would provide indications of field performance, which would influence subsequent pavement decisions. In the present study the perhaps conservative assumption has been made that the ALF trial could only be regarded as having an influence on pavement choice for about 7 years after the trial (i.e., up to about the time that this evaluation was being undertaken, since the trial was completed in 1986). During this period extensive sections of the Hume Highway have been completed, and it is regarded as appropriate to assume that the ALF trial would also have influenced the choice of pavement for one portion of the Calder Highway. In all approximately 90 carriageway km of road was identified in which the choice of pavement type could have been influenced by the ALF trial.

The present value of the benefits was estimated to be $3.4 million (1986 Australian dollars) when a 6 percent discount rate was used. Given that the total cost of the Benalla ALF trial was estimated to be $630,000 (1986 Australian dollars), the resulting benefit-cost ratio is 5.4.

As noted discussions with VIC ROADS staff identified that the most likely scenarios, in terms of the probability of still constructing the flexible granular pavement in the absence of the ALF trial, are represented by the probabilities underlying the figures presented above. These produce benefit-cost ratios in excess of 5 even at an 8 percent discount rate. Although there is inherent uncertainty associated with all estimates obtained in this type of analysis, even when using more conservative assumptions about the probabilities, the Benalla trial would still be economically justified. Results of the sensitivity analysis have been previously reported (*4*).

As noted an important factor that contributed to the high level of performance of the pavement tested at Benalla was the high density achieved in the granular layer. In individual ways pavement design procedures and specifications in Queensland, New South Wales, and South Australia have been influenced by the Benalla ALF trial to explicitly include the benefits of higher compaction. South Australia was close to making these changes in any case, and since ALF may have accelerated the introduction by at most 1 or 2 years, no benefits were quantified. The use of high-density granular bases offers the potential for savings in construction costs in Queensland and New South Wales; however, given the time that has elapsed since the ALF trial, it is difficult to assess whether these design changes could have been introduced as a result of field experience in other states, particularly Victoria. Therefore, no benefits were attributed to the ALF trial in these states. In the remaining states and territories heavy-duty granular pavements of the type tested at Benalla see rare application, and so no benefits were quantified.

Summary of Benefits from Individual ALF Trial Evaluations

As noted previously, this evaluation covered the first seven ALF trials. Table 1 summarizes the focus, outcomes, and primary benefits of these trials. In this section the nature of the benefits resulting from the trials is reviewed. The results of the economic evaluation are summarized in the following section.

Of necessity the first ALF trial, which was conducted at Somersby, New South Wales, had more to do with testing ALF than it did with testing a particular pavement. As a result of the proof testing that ALF underwent at Somersby, it emerged as a reliable testing machine for future trials. No mechanisms that would enable estimates of direct benefits from the Somersby trial to be quantified were identified. In the economic evaluation of the ALF program, the costs of undertaking the Somersby trial are regarded as part of the development costs of ALF and are therefore classified as "sunk" costs.

The second trial was conducted at Benalla, and as described in the previous section, the benefits were identified and quantified. The benefits related to savings in pavement construction costs relative to more expensive alternatives.

The trials that followed Benalla were guided by a more explicit experimental design. As noted in Table 1, these later trials examined a variety of pavement types, including cement-treated base and deep-strength asphalt and a range of materials including polymer-modified asphalt, geotextile seals, and blast furnace slag.

The primary benefits have accrued through savings in construction costs and maintenance and rehabilitation costs. For each trial conservative assumptions were made about the duration over which the benefits could be attributed to the trial.

A number of trials have led to improvements in pavement design, which in turn have produced construction cost savings. For example, the Prospect trial facilitated the design of more cost-efficient slag-based roads by designing them as heavily bound instead of lightly bound, resulting in a reduction of typical base course thicknesses from 450 to 350 mm. The Mulgrave trial found that significantly higher modulus values were achieved in the cement-treated crushed rock (CTCR) examined in the trial than are normally assumed in the design process and also that the existing fatigue relationships for CTCR were conservative. Revision of the design guides for CTCR and achievement of higher modulus values will lead to construction cost savings through a reduction in pavement thickness.

In some cases construction practices have been altered as a result of an ALF trial, resulting in savings in rehabilitation costs through extended pavement life. The Beerburrum trial, for example, focused attention on the importance of achieving an adequate bond between layers of cement-treated base (CTB) material to reduce water infiltration and thereby reduce the rate of pavement deterioration. Without the knowledge provided by this trial, a number of states would have continued to construct CTB pavements that would have experienced premature failure because of layer debonding. The benefit of the ALF trial is therefore measured by the savings in rehabilitation expenses made possible by revising construction practices to prevent debonding. It seems reasonable to expect that the debonding mechanism would have been identified by selected excavation

TABLE 1 Overview of Individual ALF Trials

Trial Location (Duration)	Pavement type/ trial focus	Primary Outcomes	Primary Benefits	Assumed Duration of Benefits
Somersby (7/84 to 4/85)	Heavy duty unbound, asphalt surfaced	Pavement did not fail	Proof testing of ALF	
Benalla (6/85 to 7/87)	Unbound, sprayed seal	Pavement did not fail	Lower pavement construction cost relative to alternatives	7 years
Beerburrum (2/86 to 7/87)	Cement treated base	Failure through layer debonding rather than fatigue	Reduced rehabilitation expenses through improved construction practices which bonded layers	5 years
Prospect (8/87 to 6/88)	Blast furnace slag as base material & stabilising agent	Performance of blast furnace slag confirmed	Construction cost savings relative to conventional materials	5 years
Callington (7/88 to 10/89)	Asphalt surfacings in pavement rehabilitation	Improved design procedures for overlays	Reductions in rehabilitation costs due to reduced depth and extended life for overlays and intersection reinstatements	5 years
Mulgrave (6/90 to 3/91)	Asphalt fatigue life	Shell Asphalt Fatigue relationship found to be conservative	Existing relationships still used pending probabilistic design process therefore no benefits quantified	
	Cement Treated Crushed Rock fatigue life	Higher modulus for CTCR than assumed in design Fatigue life relationship for CTCR found to be conservative	Construction cost savings possible through reduced pavement thickness due to less conservative fatigue life and higher modulus for CTCR	10 years
Brewarrina (7/91 to 2/92)	Geotextile seals for low volume all weather roads	Performance confirmed	Construction cost savings relative to alternatives	5 years

or drilling of boreholes in in-service CTB pavements that exhibited cracking and pumping, so it was therefore assumed that benefits could be attributed to ALF for only 5 years.

Of all the ALF trials conducted up to early 1992, Callington had perhaps the widest relevance across Australia. That trial addressed the relative performances of a variety of asphalt surfacings in the context of pavement rehabilitation. In particular, it compared a number of polymer-modified asphalt (PMA) overlays, investigated different binders in open-graded friction courses (OGFCs), and examined the effect of flexible stress-alleviating membrane interlayers (SAMIs) and the use of a high-binder mix in the tensile zone in the bottom of full-depth asphalt pavements. The primary savings resulting from the Callington trial arose from decreases in frequencies and reductions in reinstatement depths for rut repairs at urban intersections because of the use of PMAs and reduced frequencies and thicknesses of general road rehabilitations.

Although most of the trials focused on pavements for heavily trafficked applications, the Brewarrina trial demonstrated that research on pavements for lightly trafficked roads can also produce significant benefits. That trial examined the structural performance of an expansive clay pavement with a geotextile reinforced seal wearing surface. The knowledge gained from Brewarrina is relevant to roads carrying very low traffic volumes, in which road user cost savings may not justify sealing but the provision of all-weather access is seen as a community service obligation. The benefits from the Brewarrina trial arise simply from savings in road construction costs. The geotextile seal road was consistently estimated to cost around $A60,000/km less to construct than the most probable all-weather road alternative—the placement on the black clay subbase of 300 to 350 mm of gravel, to which a spray seal was then applied.

OVERALL ALF PROGRAM

In this section the ALF research program as a whole is placed into perspective. The performances of individual ALF trials are compared, the viability of the entire program is assessed, and some of the broader benefits, which are attributable to the program as a whole instead of to any one trial, are described.

Overall Economic Viability

Table 2 summarizes the economic evaluation results for all trials, with the benefits and costs expressed in the common units of 1992 Australian dollars. For each trial these figures represent the best estimates of the benefits. Prospect is the only trial in which the benefit-cost ratios were close to 1.0. For all other trials the benefit-cost ratios tend to be greater than 3.0, with the Brewarrina result higher at around 11.0.

On the right side of Table 2 the total column shows a combined result for all the trials from Benalla to Brewarrina. The overall benefit-cost ratios range from 4.0 to 5.0 depending on the discount rate. These results suggest that the internal rate of return would exceed 20 percent, indicating that a healthy return is being obtained on the investment in this pavement research program.

As noted earlier no mechanisms that would enable estimates of the benefits from the Somersby trial to be quantified were identified, so the costs of that trial have been regarded as "sunk" in the context of the economic evaluation. The Somersby trial cost approximately $A1 million at the time, which is equivalent to $A1.7 million in 1992 Australian dollars. If this cost was added to the cost of the other trials shown in Table 2, the benefit-cost ratios for the entire program would remain in the range from 3.0 to 4.0 even if no benefits are attributed to the Somersby trial. Clearly, the overall conclusion about the economic viability of the ALF program is robust with respect to the treatment of the costs of the Somersby trial.

Also of interest is the distribution of benefits. Some trials, for example, the Callington trial, have produced significant benefits in a number of states, whereas others, most notably the Prospect trial and to a lesser extent the Benalla trial, have tended to produce the

TABLE 2 Summary of Economic Evaluation Results

	Trial Location and Focus						
	Benalla	Beerburrum	Prospect	Callington	Mulgrave	Brewarrina	TOTAL
	Heavy Duty Unbound	Cement Treated Base	Blast Furnace Slag	PMA, OGFC SAMI's	Asphalt & CTCR	Geotextile Reinforced Seal	
Cost ($M, $1992)	0.9	2.2	1.8	2.4	1.8	0.7	9.8
Benefits ($M, $1992) given a discount rate of							
4 %	5.4	13.9	2.5	8.3	10.1	8.1	48.3
6 %	4.8	11.4	2.3	6.9	9.3	7.7	42.4
8 %	4.6	9.0	2.1	5.8	8.7	7.3	37.5
Benefit-Cost Ratio given a discount rate of							
4 %	6.0	6.2	1.4	3.5	5.7	11.6	4.9
6 %	5.4	5.1	1.3	2.9	5.2	11.0	4.3
8 %	5.1	4.0	1.2	2.5	4.9	10.4	3.8

Notes: PMA = Polymer Modified Asphalt
OGFC = Open Graded Friction Course
SAMI = Stress-alleviating Membrane Interlayer
CTCR = Cement Treated Crushed Rock

most benefits for the host state. Most benefits have accrued to those states that have hosted ALF trials. Western Australia, Tasmania, and the Northern Territory have derived limited direct benefits from the trials conducted to date.

Broader, Nonquantified Benefits

The past decade has seen an acceleration in the rate of advancement of pavement technology. Pavement design has advanced from crude empirical approaches to a more theoretically rigorous design process. Throughout this period ALF has acted as a focus for pavement research, bringing together researchers and practitioners from ARRB, the SRAs, and industry.

By its nature and scale of operation ALF has generated interest in pavement engineering research, and it has acted as a catalyst for field innovation. Cases were reported relating to the Beerburrum and Brewarrina trials in which regional engineers were motivated by these trials to experiment with cement slurries to bond cement-treated bases or to construct sections of geotextile sealed pavements. ALF's ability to have an impact on road authority practices has been enhanced by conducting the trials in different states. This generates a sense of local ownership, which provides an incentive to implement the results within the host SRA. The greater spirit of cooperation in road pavement research fostered by the ALF program has contributed to an increased willingness on the part of SRA staff to assess interstate innovations and practices on their merits.

ALF's credibility in accelerated pavement testing has established Australia as an international leader in pavement research. In 1987 the U.S. manufacturing rights were purchased by FHWA, and its ALF is currently stationed at the Turner-Fairbanks Research Station, McLean, Virginia. A second ALF has just been manufactured for the Louisiana Highways Department, and testing is expected to commence later this year. Another ALF was built in Australia and transported to the Peoples' Republic of China for use by the Research Institute of Highways. This overseas use of ALF has enhanced Australia's technical reputation. It is impossible to place a dollar benefit on these kudos; however, these export initiatives are clearly consistent with the thrust of the Australian federal government's economic strategy, which aims to increase Australia's role as an exporter of high-value-added items.

The accelerated nature of the testing undertaken with ALF produces data that are not only of higher quality but are also more reliable than could be obtained from other forms of full-scale testing. The only real alternatives to the use of ALF are to rely on overseas results or to conduct field trials. Overseas research programs have objectives and priorities that are unlikely to be consistent with Australian pavement research needs, and differences in climatic conditions, particularly the incidence of frost heave, mean that there are very few overseas countries where research results are directly transferable. Field trials have inherent shortcomings because the time required to obtain results has implications for the quality of data obtained. Over the 10 years or more that a major field trial would be conducted, normal staff turnover and retirements may mean that by the time the trial is completed there is no one about who has a thorough understanding of what was being tested or what was being measured. The field trial approach is also an ineffective testing method in which there are rapid changes in technology, for example, in the area of polymer additives for asphalt. One benefit claimed from ALF's success as an accelerated testing device is that expenditures on less effective field trials have been reduced. In New South Wales it was estimated that expenditures on field trials have been reduced by between $A-200,000 and $A-400,000 per year for this reason.

Another important contribution of ALF relates to the link between results obtained from the accelerated field trials and laboratory tests. Once field performance can be reproduced in a laboratory test this provides a capability to predict the field performance of other material combinations from laboratory tests without the need for further costly field trials.

CONCLUSIONS

This study has developed a framework for quantifying the benefits of ALF trials conducted to date. The framework addresses a number of issues relevant to evaluating research benefits. It is incorrect to simply compare the outcome of an ALF trial with the most expensive alternative outcome that could have eventuated by, for example, comparing the cost of the flexible granular pavement with the rigid pavement in the context of the Benalla trial. The methodology highlights that because the outcomes in the absence of an ALF trial are not known with certainty it is essential that the probability of each alternative outcome be recognized in the analysis. It is also important to consider the characteristics of each trial and assess the likely time period over which benefits can be attributed to that trial. Although not described in this paper, the methodology outlined here provides a sound foundation for a priori evaluation of future ALF trials. The framework for evaluating future ALF trials has been described elsewhere (*4*).

The overall benefit-cost ratio for the ALF program is between 4.0 and 5.0. It is important to note that all the trials were found to have benefit-cost ratios in excess of 1.0. The Prospect trial had the lowest benefit-cost ratio, slightly in excess of 1.0, but this is not surprising given that the benefits are predominantly accrued in only two areas of New South Wales. The other trials had benefit-cost ratios ranging from 3.0 to 6.0, with the exception of Brewarrina, where the value was about 11.0.

The present study has identified and quantified substantial economic benefits that are directly attributable to the ALF program. In addition, a number of significant but unquantifiable benefits were identified, including ALF's role as a catalyst for pavement research and field innovation, improved cooperation between researchers and practitioners active in the pavements area, and enhancement of Australia's technical reputation overseas. The results indicate that a healthy return is being obtained on the investment in this pavement research program.

ACKNOWLEDGMENTS

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Analysis of Arizona Department of Transportation's New Pavement Network Optimization System

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

The award-winning network optimization system (NOS) has been revised, improved, and implemented in an advanced 32-bit microcomputer environment in the Arizona Department of Transportation (ADOT) for more than 1 year. The new NOS is named AZNOS, which stands for Arizona Network Optimization System. NOS has been the primary instrument used by ADOT in planning its highway preservation program since 1980. An analysis of the microcomputer implementation in the 32-bit operating environment by using a newly developed linear optimizer, NOSLIP, is presented. The desktop AZNOS encourages use of the model for extensive sensitivity analysis and testing. The sensitivity analysis and sample runs of the new AZNOS are demonstrated. The results show that the budgetary requirements from steady-state runs should not be used for an actual highway preservation program. Instead, a *pseudo-steady state*, when the budget based on AZNOS multiperiod runs stabilizes after a transition period, is used for budget planning. Rules to set up an infeasible action list were also established to improve the effectiveness of the model. Finally, discussions were made to demonstrate how ADOT management, engineers, and university faculty teamed up to apply true optimization techniques in solving real-world pavement management problems.

Extensive research has been conducted in the last 20 years in the area of network-level pavement management systems (PMSs). The methodologies used in PMSs have been evolving along with the advancement of new technologies in computer science and mathematical modeling. In the early 1980s a major PMS development occurred in the Arizona Department of Transportation (ADOT). It represented the pioneering efforts of applying operations research techniques in PMSs (*1*). The system methodology used in ADOT PMS is called the Network Optimization System (NOS). It uses a Markov process to define the transitions of pavement conditions and a linear programming model to minimize the total agency cost and maintain the highway network at specified standards for a multiyear horizon. An estimated $40 million was saved for the state of Arizona from 1980 to 1985 (*2,3*). Subsequently, a national Management Science Achievement award was awarded to ADOT (*3*). The basic model of NOS has been used by the Alaska Department of Transportation (DOT), the Kansas DOT (*4*), Finland (*5*), and Saudi Arabia (*6*). For more than 10 years the highway preservation program based on these answers from NOS has been providing ADOT management, the state transportation board, and the state legislature information on the needs of the state highway system.

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However, the original NOS was implemented on a mainframe computer and required the user to lease a proprietary optimization program on a monthly basis. In addition to the pavement engineering staff required to run NOS, extra manpower in the information system group had to be dedicated to maintaining and updating the data base and programs. The user interface of the original development is archaic by today's standards. Since the initial implementation of NOS on a mainframe computer in 1980, technological advancement in microcomputer and problem-solving know-how have provided tremendous opportunities and insights into improving the PMS.

Therefore, enhancing the system, improving its accessibility, and simplifying its use should help more pavement engineers to use optimization techniques in their highway preservation programs. In 1991 ADOT management decided to implement enhancements to the NOS in the microcomputer environment.

The original NOS model determines the optimum long-term (stationary) rehabilitation policy and the optimum short-term rehabilitation policy (before reaching steady state) for pavements in each road category. The policies are optimum because they satisfy the prescribed performance standards with minimum cost.

The output of NOS enables ADOT management to determine

- The proportion of the pavements in each road category that will be expected to be in various condition states at the beginning of each time period, and
- The expected annual costs of pavement rehabilitation and routine maintenance.

The specific form of a rehabilitation policy is in terms of the proportion of roads of a given category in a condition state i to which a specified rehabilitation action k is applied at the l time period. The proportion can be interpreted as the probability that a given pavement would be in state i at time l and action k is taken.

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

REVISIONS OF NOS

NOS is an effective financial planning tool for pavement preservation programs on the basis of the relatively small amount of current pavement information. Only roughness and cracking information on existing pavement is needed to conduct NOS runs. In addition, the capability of conducting long-term pavement financial analyses and providing reliable information are the important driving forces for ADOT to continue relying on this important tool for the preservation program.

The mathematical model of NOS is sophisticated and includes two major operations research techniques, Markov process and linear programming. A mathematical model is intended to be a representation of the real problem in the major areas of concern. Approximations and simplifications are generally required for the model to be effective and tractable. In addition, there must be a reasonably good correlation between the performance prediction and what would actually happen in the future. On the basis of the experience in the use of and examination of the mainframe-based NOS, a comprehensive analysis of the current system was conducted in 1992. Subsequent revisions and improvements were made to the system as documented previously (*7*), resulting in an enhanced, microcomputer-based AZNOS, which stands for the Arizona Network Optimization System (*8*).

Because of the computation intensity and memory requirement of AZNOS, it was hosted in a 50-MHz 486 computer with 24 megabytes of RAM. The IBM OS/2 2.X was selected as the operating system because of its 32-bit flat memory model capability and excellent DOS and Windows compatibility with existing ADOT PMS data bases. In addition, a native 32-bit OS/2-based linear optimizer, NOSLIP, was developed for the implementation of the system.

It was revealed in this study that the factor of crack change is not significant in determining the acceleration of pavement deterioration in Arizona. Therefore, this factor was removed from the system. A new structure of pavement condition states was set up for the optimization model. The number of condition states was reduced from 120 to 45 because of the removal of the cracking change factor. The number of rehabilitation actions was reduced from 17 to 6 on the basis of the discovery that a number of rehabilitation actions were redundant. The six new actions are as follows:

1. Routine maintenance,
2. Seal coat,
3. ACFC, ACSC (asphalt concrete surface course),
4. ACFC + AR (asphalt rubber), ARAC (asphalt rubber + asphalt concrete),
5. 2-in. (5.1-cm) AC (asphalt concrete) + AR, 3-in. (7.6-cm) AC + FC (friction course), and
6. 4.5-in. (11.4-cm) AC + FC and other heavier actions.

In addition, new level boundaries for both roughness and cracking were redefined to reflect today's engineering practice. New transition probability matrices (TPMs) were established for both the Interstates and non-Interstates on the basis of the 13-year pavement performance data base in Arizona. The TPMs were modified with accessibility rules to improve the prediction of pavement behavior. Two approaches were used to evaluate the TPMs in the study. First, the current pavement performance data base was used to develop new TPMs. Second, the Chapman-Kolmogorov method was used to examine the logical extension of the transition probability matrices from a single step to the long-term pavement behavior.

As a result the concept of pavement probabilistic behavior curve (PBC) is established on the basis of the Chapman-Kolmogorov equations (*9*). Figure 1 demonstrates a set of PBCs for the region of high-traffic, desert Interstates. It can be seen from Figure 1 that the proportion of pavements remaining in the best condition state (low levels of cracking and roughness) after 20 years of service is approximately 5 percent. The curves of *best ride* (low roughness only) and *worst ride* (low cracking only) demonstrate the rapid deterioration of riding quality over time consistently.

AZNOS SAMPLE RUNS AND ANALYSIS

AZNOS is capable of conducting the steady-state and multiperiod runs in either batch or single mode. In the 50-MHz 486 computer a steady-state run usually takes less than 15 sec. For a 5-year multi-

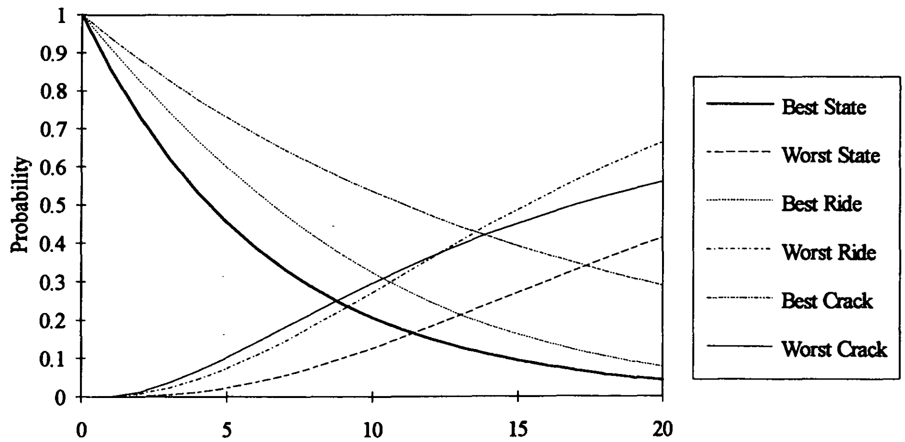

FIGURE 1 Pavement PBCs under routine maintenance after new construction, high-traffic road category of Interstates in desert region.

period run it takes from 3 to 10 min to run mostly depending on the setup of infeasible actions, the tolerance on the performance standards, and how multiple tasks in OS/2 are scheduled. The user time required on a microcomputer is much lower than that on a mainframe computer in ADOT, where it usually takes 30 min or more to get an output of a multiperiod run. Figure 2 is an example of actual AZNOS output from a NOSLIP multiperiod run for the high-traffic, desert region Interstates. The first portion of the output contains a description of the problem and its aggregate results, including the parameters used in the optimization and the optimized cost. The second portion contains the performance standards and achieved standards for each year. The summary of the AZNOS-recommended annual budget for each year of the planning horizon is also shown in Figure 2. One page is used for the detailed AZNOS budget recommendation, which is not shown in Figure 2.

Selection of Infeasible Actions

The introduction of infeasible actions was based on the finding in the original NOS development that low-level surface applications, such as asphalt concrete friction course (ACFC), were selected a disproportionate amount of the time. The selection of infeasible actions for certain condition states should be based on engineering judgment because there are no mechanistic procedures to determine the selection. However, on the basis of numerous runs conducted during this research, the rules for the selection of infeasible actions for AZNOS can be generalized as follows:

1. Routine maintenance should be feasible for all condition states. For pavements in very poor condition states, it provides AZNOS the ability to defer rehabilitation action;
2. All actions should be feasible for pavements in the best condition state. When high pavement condition standards are needed, portions of the pavements in the best condition state may need structural overlay to keep the whole network in pristine condition;
3. More than one action should be feasible for pavements in any condition state. This provides different alternatives that AZNOS can choose to achieve cost minimization; and
4. Low-cost rehabilitation actions, excluding routine maintenance, are not used for pavements in very poor condition states, such as pavements in the worst condition state with high roughness and cracking levels.

Steady-State, Multiperiod, and Performance Standard

The solution from steady-state runs represents the uniform rehabilitation strategy to keep the pavement network at required condition level. The proportion of pavements in each condition state becomes constant, and the necessary rehabilitation actions for pavements in any condition state are fixed for every year. The solution from steady-state AZNOS runs could provide important information about long-term pavement behavior and corresponding budgetary needs. Pavement steady-state rehabilitation policy is independent of time, and the actual pavement condition data at any time are irrelevant.

An observation based on the AZNOS runs is that the budget needs from steady-state runs are substantially higher than those from multiperiod runs. This observation is different from the information presented in the original NOS development, which shows that the budget requirement for steady state is less than that for the periods before the steady state. Because there are no demonstrations that budget levels from steady state must be lower than the ones from multiperiod runs, when the pavement condition standards are the same, it cannot be concluded that the relationship between steady-state budget and multiperiod budget presented in the original NOS development (*1*) are universally true. In fact, the steady-state model originally formulated only seeks to determine the budget and actions required to keep the pavement network in a constant condition with a repeated set of actions. It does not seek to determine the minimum budget required to maintain the steady-state condition when the current pavement conditions are known.

A hypothetical example can be used to explain why a budget based on steady-state runs can be higher than that based on multiperiod runs. Assume that a 100-mi (160.9-km) highway system was built 5 mi at a time each year. The design life of each section was 20 years and reconstruction was required at the end of the design life if no rehabilitation action was taken before reaching the design life. It is also assumed that pavement performances of all the 20 5-mi (8-km) sections were identical. Two rehabilitation policies could be used. The first was the steady-state policy that mandates reconstruction of a 5-mi section at the end of its design life at a cost of \$16/yd^2 (\$19/m^2). Therefore, steady state was achieved after 20 years of the construction of the first section of pavements. The second policy is to maintain the system at the same performance levels as in the steady state, but to rehabilitate any place of the system at the necessary time. Therefore, rehabilitation maintenance action of a 2-in. (5.1-cm) structural overlay can be used at a cost of \$6/yd^2 (\$7.2/m^2). The resulting budget from this multiperiod policy will be well below the budget determined by the steady-state run.

Therefore, the network steady state may never be achieved because the budgetary requirement can be too stringent to be met. In practice, steady-state results can be used as engineering references only. The budgetary recommendations of a 10-year AZNOS run is shown below for a high-traffic, desert region of Interstate highways:

Y-1	*Y-2*	*Y-3*	*Y-4*	*Y-5*	*Y-6*	*Y-7*	*Y-8*	*Y-9*	*Y-10*	Total
\$9.6	\$18.0	\$22.4	\$22.1	\$22.4	\$22.8	\$23.2	\$23.5	\$23.8	\$24.1	\$211.9

The network arrived at the performance standards at the third year. From Year 3 to Year 10 the budget need for each year was about \$22 million to \$24 million. Therefore, the network can be considered stabilized and a pseudo-steady state was achieved at Year 3. The mathematical model for the pseudo-steady state is shown in the following:

The objective:

$$\text{Minimize} \sum_{l=1}^{T} \sum_{i,k} w_{i,k}^{l} \cdot d_l \cdot c(i,k) \tag{1}$$

Subject to

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

$$\sum_{k} w_{i,k}^{l} = q_i \tag{3}$$

$$\sum_{i} \sum_{k} w_{i,k}^{l} = 1 \text{ for all } l = 1, 2, \ldots, T \tag{4}$$

```
*=============================================================*
         AZNOS MULIT-PERIOD RUN FOR 5-YEAR HORIZON
           (Based on TPM:kw31i.dat and Actual Costs)
     AUTHOR: KELVIN C.P. WANG, PAVEMENT MANAGEMENT BRANCH
         ARIZONA DEPARTMENT OF TRANSPORTATION, 1992
*=============================================================*
```

PARAMETERS & OPTIMIZED COST FROM THE LINEAR OPTIMIZER

```
Reading & Generation of Data: 0.05 min;
Program Started at Tue Oct 20 14:46:11 1992
Optimization Time:  8.82 min
Phase 1 & 2 Iterations:  275, 513;
Convergence Factors: 1.0E-06, 1.0E-06
LP Variables:1200 & Constraints:280;m1=10,m2=10,m3=260
Total Area of the Road Category    = 20082774 SY (16791757 SM)
Unit Cost(Square Yard) for 5 Years = $4.653
Recommended Budget for 5 Years     = $93.436 Million
```

CURRENT CONDITION TABLES BY HIGH & LOW LEVELS

--- Roughness(new Mays) ---		----- Cracking(%) ----	
LOW LEVEL (<75)	HIGH LEVEL (>105)	LOW LEVEL (<6)	HIGH LEVEL (>12)
0.832	0.026	0.756	0.077

PAVEMENT PERFORMANCE TABLE FOR THE MULTI-PERIOD RUN
(The Multiplier Factor, mf, is 0.95)

Year		------- Roughness -------		------- Cracking -----	
		Target	Achieved	Target	Achieved
2	Low	0.857,	0.857	0.731,	0.817
2	High	0.073,	0.024	0.073,	0.038
3	Low	0.902,	0.902	0.770,	0.821
3	High	0.069,	0.007	0.069,	0.026
4	Low	0.950,	0.950	0.810,	0.898
4	High	0.066,	0.000	0.066,	0.014
5	Low	0.950,	0.950	0.810,	0.902
5	High	0.066,	0.000	0.066,	0.013
6	Low	0.950,	0.950	0.810,	0.909
6	High	0.066,	0.000	0.066,	0.013

RECOMMENDED ANNUAL EXPENDITURE
($Million)

Action Year	1	2	3	4	5	6	Total
1	1.942	1.461	3.838	0.000	4.809	0.000	12.050
2	1.971	0.384	5.426	0.000	0.105	3.996	11.882
3	0.891	10.923	9.592	0.000	2.352	0.164	23.923
4	0.784	13.199	8.483	0.000	0.119	0.000	22.585
5	0.687	15.346	6.851	0.000	0.112	0.000	22.996
Total	6.276	41.312	34.190	0.000	7.498	4.160	93.436

FIGURE 2 AZNOS output of a 5-year multiperiod run for high-traffic, desert region Interstates.

$$\sum_{j,k} w^l_{j,k} \le P_l\,(l) \cdot \gamma_i, \text{ for } i \in I, j \in j_1(i), 2 \le l \le T \tag{5}$$

$$\sum_{j,k} w^l_{j,k} \ge p_2(l) \cdot \epsilon_i, \text{ for } i \in I, j \in j_2(i), 2 \le l \le T \tag{6}$$

$$w^l_{j,k} \ge 0, \text{ for all } j,k, \text{ and } 1 \le l \le T \tag{7}$$

where

- $w^l_{i,k}$ = proportion of roads of a given road category at lth time period that are in condition state i and to which kth preservation action is applied,
- I, T = complete set of condition states and total number of analysis periods,
- d_l = present worth of \$1 spent during lth time period,
- $c(i,k)$ = cost matrix of action k for pavements at condition i,
- q_i = current proportion of roads in ith condition state,
- γ_i = maximum proportion of roads in the set of undesirable states denoted by $j_1(i)$,
- $j_1(i)$ = the set of number specifications of undesirable states,
- ϵ_i = minimum proportion of roads in the set of desirable states denoted by $j_2(i)$,
- $j_2(i)$ = the set of number specifications of desirable states, and
- $p_1(l)$ and $p_2(l)$ = two multipliers, ≥ 1 and ≤ 1, to permit a higher than γ_i proportion of roads and a less than ϵ_i proportion of roads in undesirable and desirable states at the lth time period, respectively.

Another observation is that the budget requirement based on AZNOS is higher than that based on the old NOS. The deterioration of pavements under routine maintenance based on the new TPMs is much faster than that based on the old TPMs. There are three major reasons for this discrepancy. First, the new TPMs were based on actual pavement performance data. Because experience indicates that the real budget needs for the ADOT highway preservation program were frequently higher than the recommended budget levels based on the old TPMs, the old TPMs were thus overoptimistic on pavement performance. Second, accessibility rules were applied for the new TPMs, which eliminate the possibilities that a pavement can transition to a better condition state under routine maintenance. It resulted in higher probabilities for pavements to transition to poorer pavements than the probabilities from the old TPMs. Third, the analysis run based on the new TPMs used more stringent classifications of roughness and cracking levels and thus higher performance standards than the mainframe runs. Therefore, the required budget levels based on AZNOS are higher than the budget levels determined by the original mainframe-based NOS.

On the basis of the AZNOS runs the relationship between budget needs and performance standards was revealed. It can be illustrated by an example of both steady-state and 4-year multiperiod runs based on a high-traffic, desert region of Interstates as shown in Figure 3. Because the standard of low-level roughness is usually the critical factor it was used in the example. A range of the roughness standards from 0.900 to 0.990 with an increment of 0.005 was used. The relationships of budget and roughness standard are different for the steady-state run and the multiperiod run in this example. The budget needs for the steady-state run increase rapidly when the roughness standard passes 0.940, whereas the budget needs for the last year of the multiperiod run increase steadily along with the increasing standard. In addition, the budget needs based on steady-state runs were consistently higher than those based on multiperiod runs throughout the entire roughness range. When the standard was set to be 99 percent of the pavements with a low roughness level as shown in Figure 3, the budget to meet the standard became \$120 million for the steady-state run and about \$35 million for the fourth year of the multiperiod run. To meet this stringent standard of steady state, almost all of the pavements in the network need structural overlays every year. However, many fewer pavements need overlays if decisions are made on the basis of the multiperiod run.

AZNOS SENSITIVITY ANALYSIS

Structural improvements were made to the NOS model as described elsewhere (*9*). As a result because of the much more stringent new levels of classifying pavement condition states, the preservation program requires much higher budget expenditures by using the existing pavement condition standards. Therefore, it is necessary that

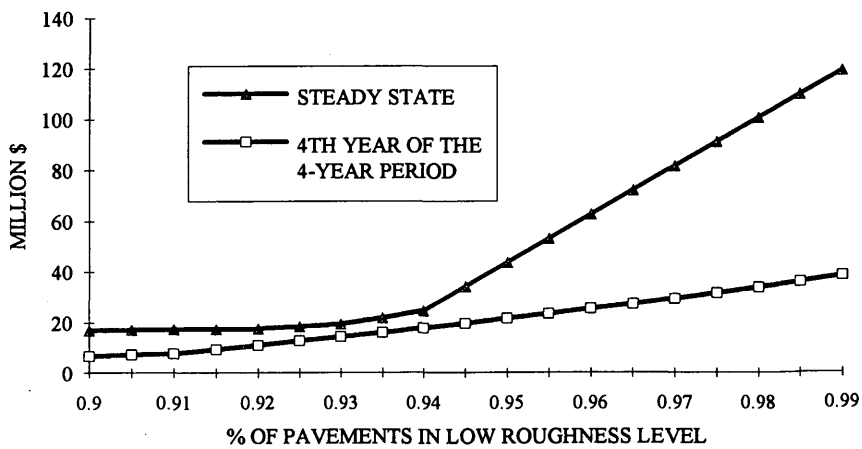

FIGURE 3 Relationship between budget needs and roughness standard on the basis of steady-state and multiperiod runs for high-traffic, desert region Interstates.

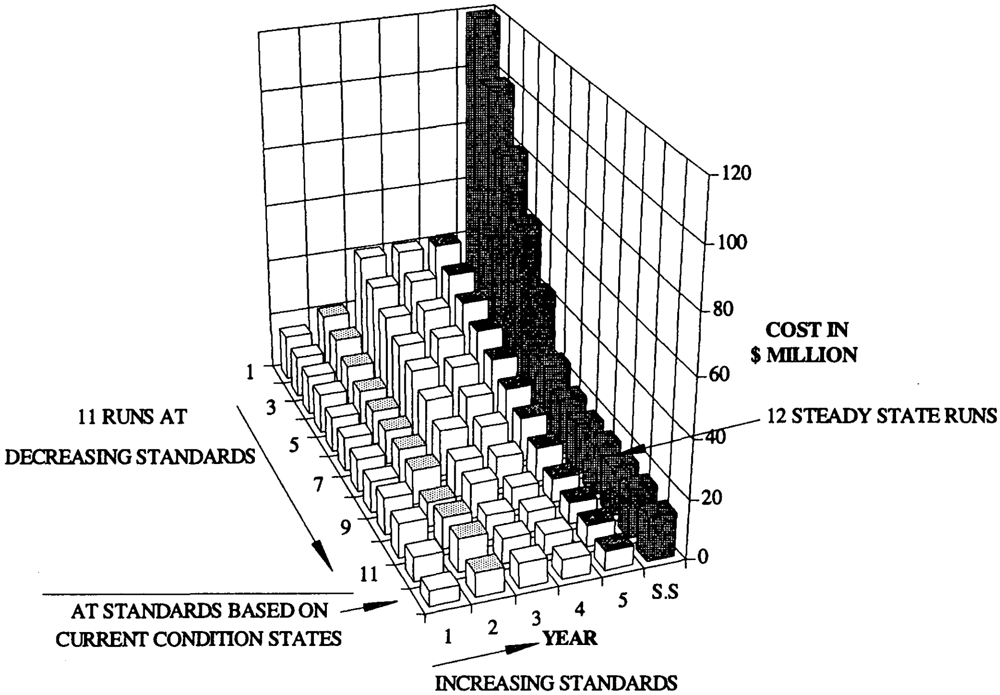

FIGURE 4 Sensitivity analysis chart for high-traffic, desert region Interstates.

a set of new pavement condition standards be determined on the basis of the modifications and the possible budget program. In addition, to validate the model and examine the sensitivities of its parameters, variations of the standards need to be tested against the corresponding budgetary requirements.

The starting point to set up the ranges of performance standards is the present pavement condition. The original performance standards were used for the high end of the sensitivity testing range. A total of 11 runs were conducted for each of the five road categories by using a decreasing standard between each run. A 12th run was conducted by using the existing pavement condition states as the standard for the corresponding road category. The low roughness levels in most AZNOS runs were the critical factors; thus, it was used in all sensitivity analyses.

Because the high-traffic, desert region of Interstates has the largest pavement area in Arizona, it was selected to demonstrate the sensitivity analysis here. Steady-state and multiperiod runs are shown in Figure 4. The vertical axis represents the required budget, in millions of dollars. A 5-year horizon was assumed in all the multiperiod runs. The x-axis represents the year number of the AZNOS runs. The y-axis represents AZNOS run numbers. It should be noted that a special range of roughness standards from 0.90 to 0.99 was used for this road category for the purpose of illustration. The roughness standards for the high-traffic, desert region of Interstates are highlighted in Figure 5. The vertical axis of Figure 5 is the percentage of pavements with a low roughness level. In addition, it was assumed that the pavements of Interstates arrive at standards at Year 3.

Total budget needs for Interstates are shown in Figure 6. The multiplier factor of 0.95 was used for all the multiperiod runs so that lower standards could be used for the interim years to converge to the standards. Figure 6 shows that higher budget needs correspond to higher standards. It can be seen that the budget requirements for Interstate highways are evenly spread out throughout the 5-year horizon for 12 runs. In addition, as discussed previously, budget requirements for steady-state runs are consistently higher than those for multiperiod runs, as shown in Figure 6.

NEW PAVEMENT CONDITION STANDARDS FOR ADOT

On the basis of the 12 AZNOS runs conducted for each road category for both steady-state and multiperiod runs, performance standards were determined, as shown in Table 1. The corresponding AZNOS runs were conducted, and the subsequent budget requirements for Interstate highways are shown in Figure 7. The total network budget is shown in Figure 8 for the fifth year of the planning horizon. The ADOT budget for 1991 was $60 million without considering the safety and other engineering costs, whereas the results from AZNOS show that at the fifth year about $78 million is needed for the pavement network to meet the new standards.

ACHIEVED RESEARCH GOALS

Improvements were made to the ADOT NOS in this study, resulting in the 32-bit microcomputer-based AZNOS. New tools were provided for the analysis of pavement long-term behavior on the basis of the Chapman-Kolmogorov equations. Extensive sensitivity analysis was conducted on the newly developed AZNOS to validate the implementation and set up new pavement condition standards for ADOT. It was revealed in that sensitivity analysis that the budget needs based on steady-state runs are not necessarily less than those based on multiperiod runs. Because it required fewer resources to keep the network at the standards when the solutions

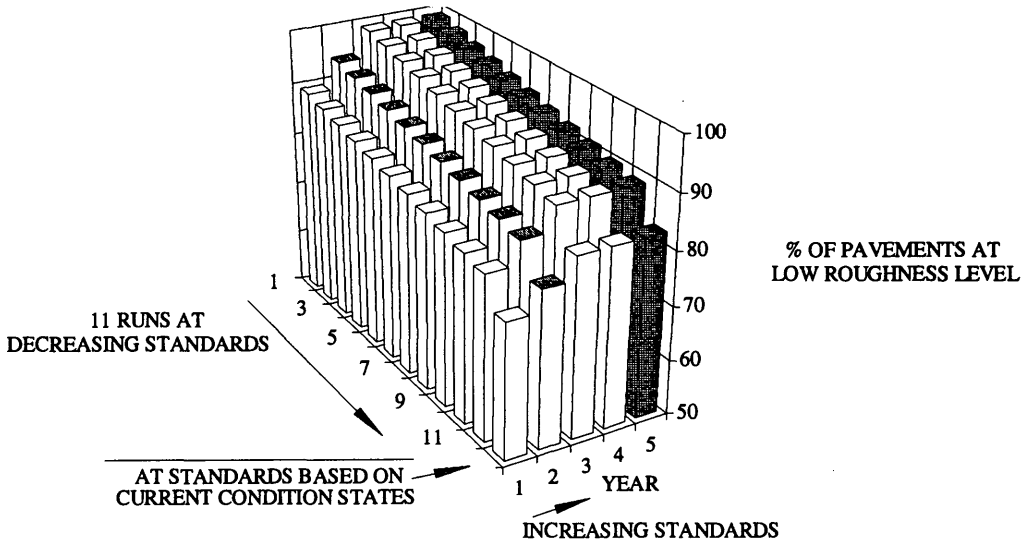

FIGURE 5 Standards of low roughness level for sensitivity analysis for high-traffic, desert region Interstates.

from multiperiod were used, it was recommended that steady-state results not be used for actual pavement preservation program.

On the basis of the results of the present study it was concluded that AZNOS multiperiod runs should be used in an actual pavement preservation program. The ideal situation represented by solutions from steady-state runs is not practical. On the basis of the Chapman-Kolmogorov equations, a methodology was developed for the analysis of long-term pavement behavior by using PBC.

The newly developed 32-bit linear optimizer NOSLIP was used as the optimizer for AZNOS. Very few numerical problems were met in testing and using NOSLIP with other sample problems. A few problems were encountered because of the use of the AZNOS input data instead of the optimizer. For instance, when certain rehabilitation actions were set to be infeasible, such as structural overlay for the best condition state, the optimizer may determine that the problem was infeasible. The reason for this is that when the standards were set very high structural rehabilitation action had to be applied to a portion of the pavements in the best condition to satisfy the standards. Therefore, all actions had to be available to pavements in the best condition state for the optimizer. The new system provides better user interface and lower consumption of user time. Batch run capabilities are also provided. The linear optimizer NOSLIP provides a much faster response time than the mainframe version. The newly developed, microcomputer-based AZNOS pro-

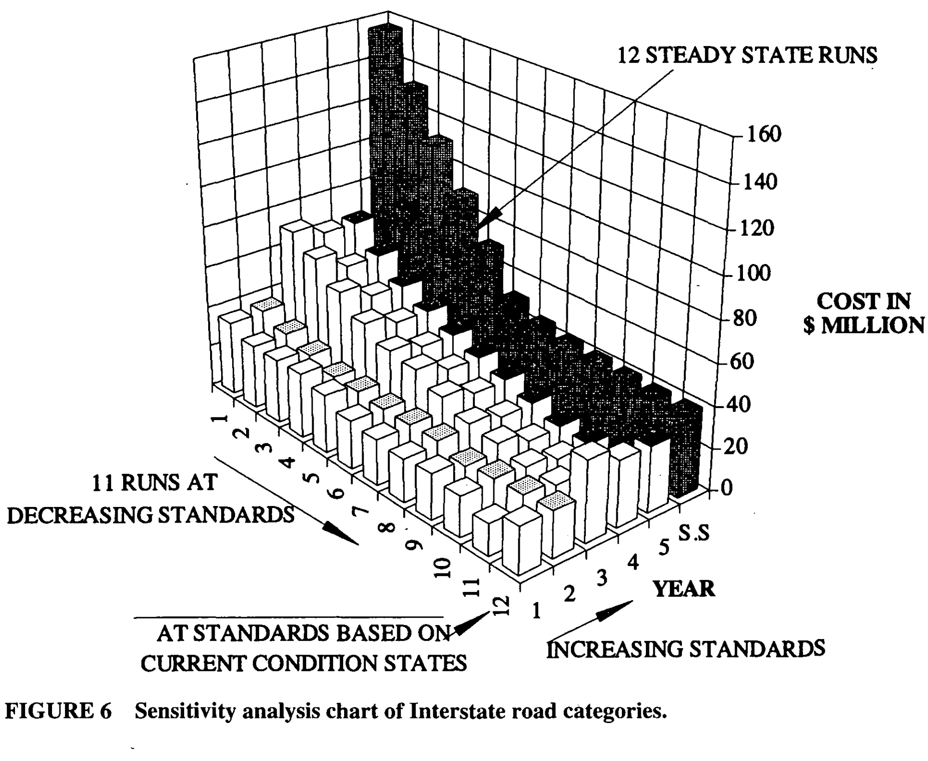

FIGURE 6 Sensitivity analysis chart of Interstate road categories.

TABLE 1 New Performance Standards for Highway Preservation Program of ADOT (Asphalt Concrete)

	TRAFFIC ADT	MIN. % MILES IN SATISFACTORY CONDITION	MAX. % OF MILES IN OBJECTIONABLE CONDITION
INTERSTATE	ROUGHNESS		
	0-2000	NOT APPLICABLE	NOT APPLICABLE
	2001-10,000	85	5
	10,001+	95	5
INTERSTATE	% CRACKING		
	0-2000	NOT APPLICABLE	NOT APPLICABLE
	2001-10,000	80	5
	10,000+	85	5
NON-INTERSTATE	ROUGHNESS		
	0-2000	45	25
	2001-10,000	70	10
	10,000+	80	10
NON-INTERSTATE	% CRACKING		
	0-2000	60	20
	2001-10,000	70	15
	10,000+	80	10

vides an enhanced PMS with improved accessibility and faster response time.

OPTIMUM SOLUTION, SATISFACTORY SOLUTION, AND PMS

In the past 15 years or so a number of institutions have been using techniques of optimization and stochastic process for their pavement preservation programs. Many of them were very successful. However, even larger numbers of agencies have continued using relatively simple PMS based on ranking, prioritization, or semioptimization techniques. Even though optimization provides powerful tools for minimizing agency costs or maximizing benefits, it requires PMS engineers to have a substantial quantitative background and the management and legislature to be supportive in adapting new systems. In addition, the quality of the prediction model and the input data basis will determine the accuracy of the output from the optimizer. Furthermore, the resources needed to develop an optimization-based PMS are substantially higher than those needed to develop a simple data base–based system.

However, NOS has been an important instrument in the ADOT pavement preservation program in the past 12 years. The development of the current annual budget relies heavily on the outputs from the multiperiod AZNOS runs. The continuing reliance on this PMS on the basis of optimization has saved taxpayers tens of millions of dollars in the past decade (*3*). The successful story of the Arizona PMS is the result of three combined efforts: management support, the pavement management engineer's capability of using new technologies and innovations to maintain and update the PMS data bases, and research efforts from the university faculty. This research project was another endeavor from the PMS engineer and the management that new advancements in both hardware and software were used for the enhancement of the existing system. Therefore, to succeed in using an optimization-based PMS, the application of optimization techniques for pavement management should be fully supported and understood by the management. In addition, improving the system by using new technologies must be vigorously pursued by the pavement management engineer.

It should be recognized that the solutions from the optimizer are optimal only with respect to the model being used. Because the model is an idealized instead of an exact interpretation of the real problem, there cannot be a guarantee that the optimal solution from the model will prove to be the best possible solution that could be implemented for the real problem. There are too many imponderables and uncertainties associated with the real problem. However, if the model is well formulated and tested, the resulting solution should be a good approximation of the real problem. Arizona's experience shows that the test for the practical success of an

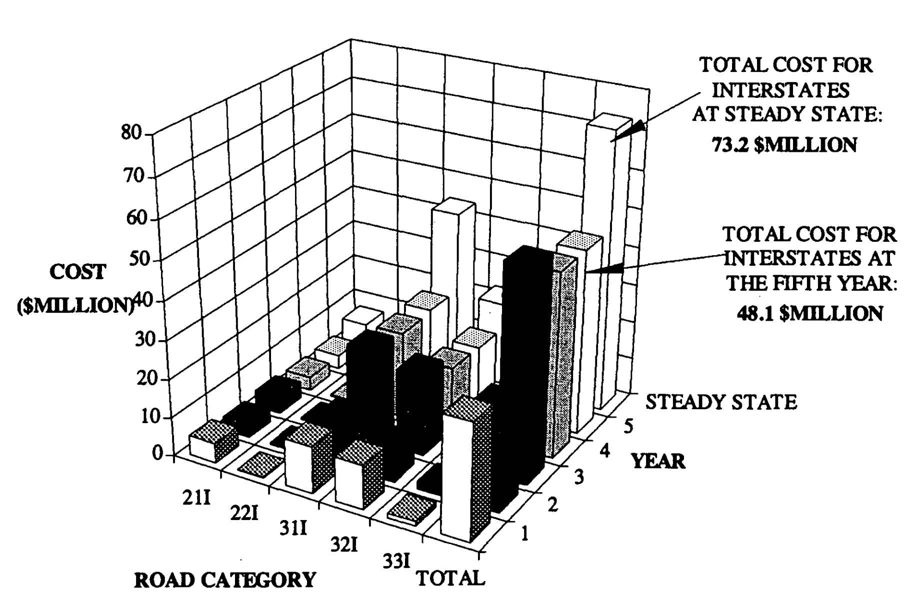

FIGURE 7 Determination of standards for Interstate highways.

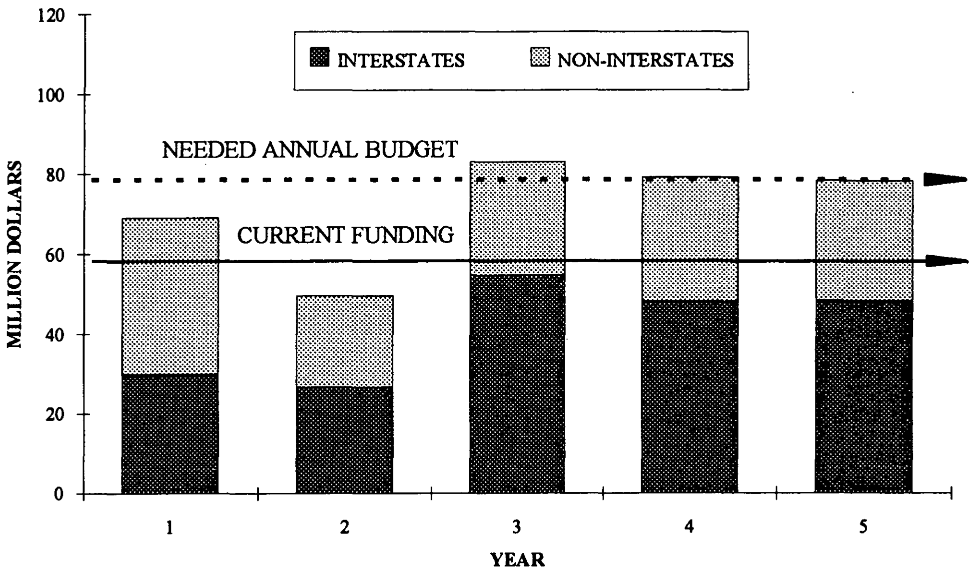

FIGURE 8 Network budget program for fifth year of 5-year planning horizon on the basis of multiperiod AZNOS runs.

optimization-based PMS should be whether the system provides a better guidance for the preservation program than can be obtained by other means. It is impossible to prove that AZNOS is the best possible tool. However, the tremendous savings achieved by the state through the use of true optimization techniques demonstrate that AZNOS is an excellent tool for determining the pavement preservation program's budget requirements.

H. Simon points out (*10*) that *satisficing* is much more prevalent than optimizing in an actual practice. He defines *satisficing* as a combination of *satisfactory* and *optimizing*. Simon describes the tendency of managers to seek a solution that is "good enough" for the problem at hand instead of develop an overall measure of performance to optimally reconcile conflicts between various objectives. The distinction between optimizing and satisficing reflects the difference between theory and the realities frequently faced by many PMS engineers in trying to implement that theory in practice.

FUTURE RESEARCH NEEDS

The application of Markov prediction models in PMS has provided an effective technique in pavement performance prediction. NOS and AZNOS require the infeasible action list as input data to improve the ability of AZNOS to choose cost-effective actions. The requirement that an infeasible action list be used relates directly to the prediction model structure, which needs further studies for possible improvement.

The steady-state model implies that steady-state solutions may recommend longer life actions that require less programming but more expensive projects. Fewer projects in the annual program can mean fewer user delay costs and possibly lower agency administrative costs. The quantification of these factors needs further study. In addition, NOS and AZNOS are not capable of establishing realistic pavement project locations because of the aggregate model structure. Efforts are under way in ADOT to integrate a Knowledge-Based Expert System to the existing PMS so that expert opinions can be stored in the computer to help determine candidate projects, including the locations and timings. Furthermore, sensitivity analyses of the Markov prediction models and their possible calibration are necessary and were never conducted before.

The successful realization of rehabilitation project selection and global optimization in the microcomputer environment will advance pavement network optimization to a new level of sophistication and maturity. A systems approach can be applied to the integration of financial planning, program planning, and pavement design into a single package within the framework of modular design and object-oriented programming. The modern 32-bit desktop operating systems provide tremendous opportunities for the integration of graphical data presentation and query, geographical information systems, and multimedia capabilities along with the existing optimization module into a comprehensive pavement information system in computer networks.

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Pavement Management System for Provinces in Developing Countries: Implementation in Fayoum, Egypt

SAFWAN A. KHEDR AND IBRAHIM A. EL DIMEERY

Almost every road agency is faced with a difficult formula: maintaining its road network to meet the expectations of a minimum level of service, at escalating costs, with ever-constrained budgets. If they are well designed and implemented, suitable pavement management techniques constitute the main answer to this problem. Road agencies of provinces in developing countries are no different; they face even more stringent constraints and limited resources. These agencies also confront social and developmental concerns that may not be of prime concern in other countries. A pavement management system for provinces in developing countries (PMSPDC) geared toward the needs and concerns of these agencies is developed. It features the main development stages of compiling available information, setting the agency's goals, data collection, basic priority criteria, data base design and analysis, execution plans, and determination of the needed resources. A modified pavement condition rating was adopted for paved roads. A developed unpaved road condition rating was used for unpaved roads. The priority factors considered are road condition, population distribution, economic development, regional development, traffic growth, and another factor that was left open for a suitable concern, for example, tourism attraction. PMSPDC is simple, provincial, flexible, and objective. It reflects local values and concerns. It is not a goal in itself, but it is an effective tool that keeps the agency on top of the action and ahead in performance and communication with higher authorities and the general public and that is able to stand firm against unjustified pressures. PMSPDC was successfully implemented in the province of Fayoum, which is a semi-oasis located 105 km southwest of Cairo, Egypt. Analysis of the data in view of PMSPDC priority criteria revealed that the factors considered are mostly independent and significant. The system and the derived justified objective execution plans were well perceived and understood both by the agency's official and by high-ranking officials in the province. Ensuring the optimal nature of these plans made it easier to secure reasonable budgets for road network maintenance and upgrading.

"Local government managers responsible for low-volume roads in the United States are facing a dilemma. On the one hand, there is growing pressure to repair roads and provide an improved level of service. On the other hand, there is public pressure to reduce taxes" (*1*). This dilemma is not confined to the United States; it exists almost everywhere. In addition to the demand for better service with constrained budgets, other factors add to this dilemma in provinces in developing countries. Examples are poor record-keeping on projects, outdated maintenance procedures, lack of quality control facilities and practices, and poorly coordinated decisions regarding network planning. Quite often, these decisions are taken as responses to emergency needs, political pressures, or the complete deterioration of some roads.

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A pavement management system for provinces in developing countries (PMSPDC) was established in the work described here to meet the urgent need for a tool that is effective and efficient in directing the activities involved in providing and sustaining pavements in an acceptable condition at the lowest possible cost. In the United States several pavement management systems for municipalities and agencies with low-volume roadway networks have been developed, for example, MicroPAVER (*2*) and MTC-PMS (*3*). However, in developing countries upgrading of roadway networks not only should seek optimal use of the budget as the primary criterion but also should consider the economic, social, and regional developments of the influence area served by the network. The PMSPDC can be used as part of the transportation planning process.

Most governorates (provinces) in Egypt have short-term plans for maintaining their road networks. Recently they have recognized the significance of establishing pavement management systems for their road networks with the main objectives of putting a priority program according to their available annual budgets and maximizing the positive impacts on the communities and their development. The PMSPDC was developed for that purpose and was implemented in the province (governorate) of Fayoum in Egypt. Presented in this paper are the essence of PMSPDC and the preliminary results of its implementation in Fayoum.

OBJECTIVES

The main objectives of PMSPDC are to

1. Help the province set its goals and improve and upgrade its local road network.
2. Encourage decentralization and the development of local capabilities.
3. Set up an information management system that can aid the decision-making process in assigning investment priorities within the capabilities of the available resources.
4. Develop a general plan for the road network in the province. The plan should
   - Provide a program for construction, maintenance, and improvement of the existing network;
   - Determine if new road links are required to respond to local development needs;
   - Include both paved and unpaved roads;
   - Develop priority criteria to be used in the selection of individual road segments for upgrading; and
   - Initially cover a period of 10 years.
5. Study, assess, and evaluate the existing status of the road agency and road network in a province.

6. Review the plan developed in Objective 4 with various elements of the road agency to achieve an execution program and to determine the resources required to implement it (budget, manpower, equipment, and materials).
7. Identify problematic conditions of roads and review design and construction parameters and considerations on the project level.

FEATURES OF DEVELOPED SYSTEM (PMSPDC)

A successful pavement management system should be tailored to the circumstances of the agency using it. For the scope of PMSPDC several characteristics are of prime concern. The PMSPDC should be

1. Simple. To ensure success and the continuity of implementation the system should be simple enough so that local officials and engineers are able to understand, implement, update, and employ it in their strategic network-level decisions and project-level assessment and design processes. The system, however, should not sacrifice objectivity and completeness in this process and should minimize subjective judgments. Computer involvement should be kept within the available facilities, for example, microcomputers, and be interactive and simple to use.
2. Provincial. When it is implemented the system should answer the agency's concerns: technical, administrative, social, economic, growth, and even political.
3. Flexible. The system should be modifiable and updatable if strategic changes or new data were to become available. It should have the capability of using feedback information regarding the consequences of decisions.
4. Optimal. An optimal priority assessment technique should be included to ensure the best value for investments.
5. Systematic. Periodic data collection, compilation, and updating should be done systematically so that decisions can be checked and reviewed. Information will then be easily retrievable in a useful format.

COMPONENTS AND ACTIVITIES OF PMSPDC

A schematic flow chart of the proposed activities of the PMSPDC is shown in Figure 1. These activities are detailed in the following paragraphs.

Compiling Available Information

Available information for the area under the influence of the road network is collected. This information includes road network data (types, lengths, and functional class); population distribution; economic activities; the district and township locales served by the network; social data, for example, migration movements and education; the administrative structure, policies, programs, and resources of the road agency; and safety issues on the road network.

Setting Agency's Goals

A clear set of goals should be outlined to set the direction in which the agency is heading and how far it wants to go. In this respect general and stage goals may be defined. The general goals should deal with (*a*) the basic needs of people in urban and rural areas and the minimum levels of these needs and (*b*) upgrading of the network to keep up with the expected expansion of economic activities: industrial, agricultural, and so on. Stage goals are more specific and incorporate the time span over which plans are made. These goals should target (*a*) minimum road condition (level of service), (*b*) the continuity of the transportation network, (*c*) servicing of regions with economic activities, (*d*) the level of social development, (*e*) safety level, and (*f*) the performance and productivities of various divisions within the road agency. These stage goals may be revised in view of the outcome of the PMSPDC within realistic constraints and available resources.

Network Data Collection

The decision-making process and the establishment of a pavement management program will not be valid unless they are based on real, detailed, analyzed information on the road network under consideration. Data collection for PMSPDC includes road inventory, a pavement condition survey, and a traffic count.

Road Inventory

The road inventory should include the following for each link and segment:

- Identification and classification of the link or segment. (A proper coding system for the network may have to be established.)
- Land use and the general topography surrounding the road.
- Elements of road geometry: horizontal and vertical curves; effect on sight distance; cross-section dimensions; side slopes; height of embankments, culverts, and bridges; traffic signs; and lighting facilities.
- Available records on road construction, maintenance, and its itemized costs. It is likely that the system is faced with the obstacle of insufficient records. Interviews with engineers and the personnel in charge will temporarily fill that information gap until the system is implemented. From then on information is collected within the system.

Shown in Figure 2 are the proposed forms for collecting geometric characteristics and road segment records.

At this stage preliminary ideas about potential new roads should be developed and the location should be reviewed. The new roads may connect communities that presently do not have access to the road network, or they may serve new economic or regional development areas.

Pavement Condition Survey

Because of the lack of facilities it is not expected that provinces in developing countries will have access to the equipment capable of testing pavement structure or the pavement surface. Therefore, pavement evaluation for network-level management should be made simple and carried out independently of sophisticated equipment. However, it should encompass all of the problems anticipated to occur in pavements in the province. The results that are obtained

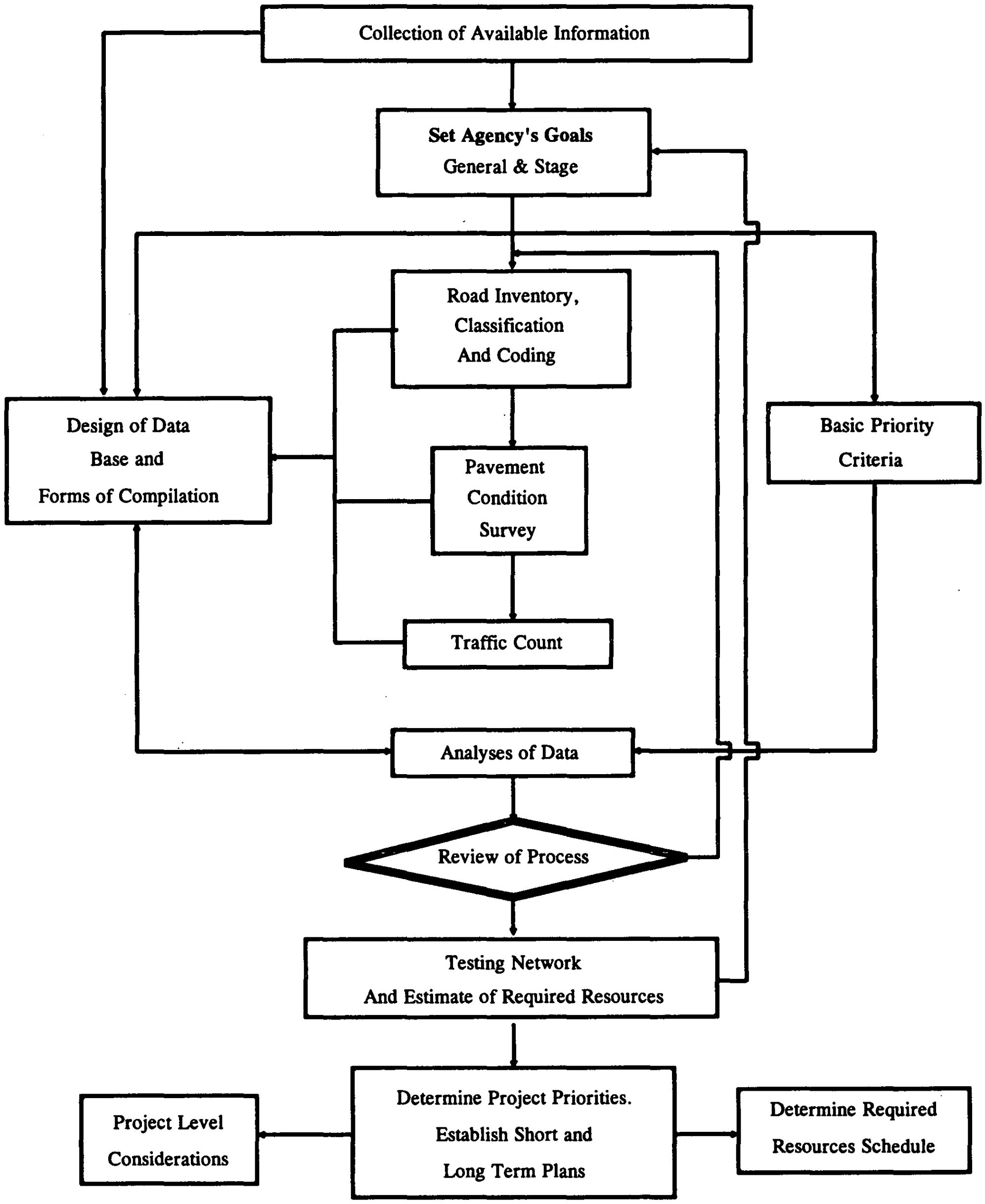

FIGURE 1 PMSPDC development process.

should be reliable and repeatable if evaluation is made by different inspection teams.

Visual inspection of the pavement surface condition is considered the major tool in collecting data for network management. It is important to use uncomplicated, easy-to-understand condition survey methods to ensure consistency and the ability of the agency's personnel to update information for the continuity of PMSPDC. The pavement condition rating (PCR) model (*4*) is adopted and modified to emphasize provincial concerns. A similar evaluation scheme is developed for an unpaved (earth) road condition rating (URCR).

Figure 3 presents the form for survey and calculation of PCR. It is calculated as follows:

$$\text{PCR}\ (\%) = 100 - \sum_{i=1}^{n} w_i t_i d_i$$

where

w_i = relative weight (importance) of distress (i),
t_i = distress severity factor,
d_i = distress extent factor, and
n = number of distresses of concern = 13.

The description of t_i and d_i for the different distresses is given in Table 1. It should be noted that the values for relative weight (w_i) in Figure 3 are set for conditions in which more than two distresses are encountered on the pavement. If only one type of distress exists the corresponding w_i should be doubled. If only two types of distress exist the corresponding w_i should be increased by 50 percent. The emphasis of this rating was placed on cracking because it is believed to be responsible for the fast deterioration of pavements in many regions of the Third World.

Record of Road Link

Road Link: No.: Length:

Compiled By: Date:

Segment		Date of Action	Type of Construction or Maintenance	Base Thickness (cm)	Paving Thickness (cm)		Pavement Width (m)	Shoulder Width & Type (m)		Total Initial Cost	Remarks
From	To				Binder	Surface		Right	Left		

(a)

Geometric Elements

Road Link: No: Length:

Inspected By: Date:

Segment		Average Width (m)	Shoulder Width & Type (m)		Topography & Land Use		Sharp Horizontal Curves*	Steep Vertical Curves*	Remarks
From	To		Right	Left	Right	Left			
Schematic Sketch of Link:									

* Do not satisfy proper sight distance

(b)

FIGURE 2 Forms for: (*a*) road record and (*b*) road geometric elements as constructed.

The URCR is calculated in a way similar to that for PCR. For all distress types the extent factor d_i is designated 0.4, 0.8, and 1.0 for scarce, some, and frequent, respectively. Severity is designated low or high ($t_i = 1.0$). Shown in Table 2 are the values for relative weight and low severity factor for the different types of distress on unpaved roads. Ten types of distress are considered in Table 2. The stability and integrity of the road embankment receive higher weights of importance. Distresses 9 and 10 in Table 2 are related to the improper use of the road's right-of-way, which is not uncommon in developing countries. Also presented in Table 2 is the description of the degree of severity of the distresses. The categories of extent are designated less than 20 percent between 20 and 50 percent, and more than 50 percent for scarce, some, and frequent, respectively, for all distresses except the last two. The improper uses of right-of-way on unpaved roads (Distresses 9 and 10 in Table 2) have scarce, some, and frequent extents if two, two to five, and more than five obstructions are encountered on the road segment, respectively.

The pavement condition survey is performed on every road segment (which normally averages 2 km in length). The survey is done for three randomly sampled sections of 200 m each on the road segment. The pavement condition survey for the network should be performed before every budget planning period.

Traffic Count

The traffic count scheme should be designed so that the average daily traffic is measured or estimated for all road links in the network. This may include 24-, 12-, 6-, and 2-hr counts and classified counts for every group of road links categorized by traffic condition. These measurements or estimates should be adjusted to reflect traffic growth. Traffic growth is mainly attributed to growth in population, changes in standards of living (car ownership), and economic development.

Basic Priority Criteria

On the provincial level in developing countries numerous factors should affect the decision-making process on the network level. Other than the technical evaluation of road condition and perfor-

Pavement Surface Condition Survey

Link:	No.:	Length:
Segment: From	To	Length:
Inspected by:		Date:

Distress	Rel. Wt.	Severity			Extent			Deduction**
		Low	Med	High	Scarce	Some	Freq	
1. Raveling	10	0.3	0.6	1.0	0.5	0.8	1.0	
2. Bleeding	5	0.8	0.8	1.0	0.6	0.9	1.0	
3. Patching & Repair	5	0.3	0.6	1.0	0.6	0.8	1.0	
4. Potholes *	10	0.4	0.7	1.0	0.5	0.8	1.0	
5. Local humps	5	0.3	0.6	1.0	0.5	0.8	1.0	
6. Rutting *	10	0.3	0.7	1.0	0.6	0.8	1.0	
7. Local Depression	10	0.5	0.7	1.0	0.5	0.8	1.0	
8. Corrugation	5	0.4	0.8	1.0	0.5	0.8	1.0	
9. Alligator Cracking *	15	0.4	0.7	1.0	0.5	0.7	1.0	
10. Block & Transverse Cracking	10	0.4	0.7	1.0	0.5	0.7	1.0	
11. Longitudinal Cracks	5	0.4	0.7	1.0	0.5	0.7	1.0	
12. Edge Cracks	5	0.4	0.7	1.0	0.5	0.7	1.0	
13. Other Cracks	5	0.4	0.7	1.0	0.5	0.7	1.0	
**Deduction = Rel. Wt. * Severity * Extent				Total Deduction				
PCR = 100 - Total Deduction				Total Structural Deduction*				

FIGURE 3 PCR survey and calculation.

mance, there are considerations pertaining to regional development, social development, economic activities and development, safety, and political pressure. To simplify the process in the PMSPDC six factors were considered to assign a priority ranking to road segments in the provincial network. Listed in Table 3 are these factors and their relative weights in priority designation. Discussion and the measurement of each factor are presented below.

Road Condition

The relative weight of road condition is the highest among the considered factors; however, it amounts to about one-third of the priority criteria. From a technical point of view this is the only factor that focuses on pavement performance. PCR and URCR are used to reflect pavement condition. The geometric features of the road should be investigated to study the need for road realignment and the effects of these needs on safety. Urgent safety-related geometric problems are either flagged (on the computer program) or reflected in the values of PCR or URCR (by reducing them). Presented in the Table 4 are relative weights for various road conditions. Emphasis is given to paved roads whose condition is between good and failed (or very poor), on which pavement performance deteriorates faster in the case of delayed maintenance actions. Poor and failed paved roads receive a higher priority than unpaved roads with similar rankings. This contemplates the higher expectation versus severe roughness on pavements under such conditions. There is no "excellent" unpaved road, and it warrants greater attention at the "very good" level than paved roads.

Population Distribution

The population distribution factor focuses on the distributions of the urban and rural populations in the influence area of the road. A population distribution ratio for a road link is calculated as follows:

$$\text{Population distribution ratio} = \frac{\text{Population served by the road link}}{\text{provincial population}} \times \text{urbanization factor} \times \frac{\text{number of municipal division served by link}}{\text{total number of municipal divisions in province}}$$

TABLE 1 Degrees of Severity and Extent of Distresses for Paved Roads

Distress	Severity			Extent (%)		
	Low	Medium	High	Scarce	Some	Frequent
1. Raveling	Loss of some particles	Noticeable pitting and roughness	Very rough, full of pitting	< 20	20-50	> 50
2. Bleeding	------	Asphalt fill voids between aggregates	Covered with asphalt film	< 10	10-30	> 30
3. Patching	Some deterioration	Partial failure	Needs replacement	< 10	10-30	> 30
4. Potholes	Diameter < 15 cm Depth < 3 cm	Diameter > 15 cm Depth 3-5 cm	Diameter > 15 cm Depth > 5 cm	< 20	20-50	> 50
5. Local Humps	Diameter < 15 cm Height < 3 cm	Diameter > 15 cm Height 3-5 cm	Diameter > 15 cm Height > 5 cm	< 10	10-30	> 30
6. Rutting	< 6 mm	6-25 mm	> 25 mm	< 20	20-50	> 50
7. Local Depression	Some, with negligible effect of driving	Discomfort in controlling vehicle	Difficult to control vehicle	1/km	2-4/km	> 4/km
8. Corrugation	Some, with some discomfort in driving	Discomfortable driving but good control	Severe vibration and reduced speed	< 10	10-30	> 50
9. Alligator Cracks	< 3 mm single or cross cracks	3-6 mm multiple or alligator pattern forming	> 6 mm alligator pattern established	< 20	20-50	> 50
10. Block & Transverse Cracks	< 3 mm	3-25 mm width spalling with less than half length	> 25 mm with common spalling	< 20	20-50	> 50
11. Longitudinal Cracks	< 3 mm single	> 3 mm single or multiple	multiple with spalling	< 20	20-50	> 50
12. Edge Cracks	< 6 mm without spalling	> 6 mm with spalling	multiple with spalling	< 20	20-50	> 50
+13. Other Cracks	< 3 mm without spalling	3-25 mm with spalling	> 25 mm with spalling	< 20	20-50	> 50

TABLE 2 Severity of Distresses on Unpaved Roads

Distress	Rel.Weight	Severity		
		Low		High
1. Failure & Erosion of Embankment	15	Width < 2 m Depth < 5 cm	0.5	Width > 2 m Depth > 5 cm
2. Failure of Side Slopes	15	Does not effect right-of-way	0.3	Reduce right-of-way
3. Corrugation	15	< 3 cm	0.5	> 3 cm
4. Uneven Cross Section	10	Depressions are noticed with not obvious cross slope	0.5	Severe depressions which affect comfort and safety of driving
5. Rutting	10	< 3 cm	0.5	> 3 cm
6. Local Depressions	8	Diameter < 30 cm Depth < 5 cm	0.5	Diameter > 30 cm Depth > 5 cm
7. Intrusion of Irrigation Water	8	Minor with width < 1 m	0.4	Water affects traffic and reduces speed
8. Vegetation of Road Surface	7	Area < 1 m^2	0.5	Area > 1 m^2
9. Obstacles	7	Length < 1 m	0.5	Length > 1 m
10. Illegal Use	5	Length < 1 m	0.4	Length > 1 m

TABLE 3 Relative Weights of Priority Factors

Factor	Relative Weight
Road Condition	35
Population Distribution	20
Regional Development	15
Economic Development	10
Traffic Growth	10
Other (Tourism Attractions)	10
Total	100

The "urbanization factor" depends on the type of residence: 1.2 for urban and 1.0 for rural. The population distribution factor is determined from 5 to 20 in proportion to the population distribution ratio. This relationship will differ from one province to another, depending on the actual population and the number of municipal divisions in the province.

Regional Development

If the road link serves new or developing existing regional or urban developments, it is given a higher priority. The regional development factors for three conditions are provided in Table 5.

Economic Development

The role of highways in economic progress is indisputable. If a road link has a direct impact in serving economic activities, for example, industrial and agricultural activities, it is designated a higher-priority road. Road links in this category are expected to carry traffic with a higher ratio of heavy vehicles. This is used as an indicator. The economic development factors for three activity levels are provided in Table 5.

Traffic Growth

In general, most roads on provincial networks in developing countries can be categorized as low volume. Therefore, traffic volume may not be regarded as a major factor in the priority measure because most traffic volumes on roads will be too low anyway. This factor is given a relative weight of 10 out of 100. It is measured in reference to the average daily traffic, as shown in Table 5.

Other Factors

Provinces may differ somewhat in their consideration of factors that affect ranking of roads in a management scheme. A relative weight of 10 is left to be filled with a factor of particular interest. This factor may regard, for instance, strategic defense or political, environmental, or tourism considerations. In Egypt, for example, tourism attraction is considered important in most provincial activities. The relation of a road link to these activities will be a criterion in deciding its relative priority.

The overall priority index (PI) is the sum of all six criteria factors discussed above. A higher PI for a road segment will place it higher on the construction, rehabilitation, or maintenance list.

Political influence is usually an important issue. It can be handled under one of two different scenarios. First it may be assigned a designated relative weight and given a value decided subjectively in relation to the prevailing political directions. The main advantage of this alternative is that it gives a predecided weight to political pressure and therefore puts it within predetermined limits. On the other hand it may be difficult to get politicians to agree to the limitation and acknowledge the existence of such pressure. The second scenario is to continue with the system and then review the final execution program with the political domain, having the tool to assess the consequences of altering the program.

Data Base Design

Data storage, organization, analyses, and retrieval are done best by using computer facilities. However, on the provincial level this should be made simple and friendly for use by local officials and engineers. In most provinces personal computers have recently become available. In PMSPDC a program is developed to satisfy these requirements in a DBASE IV software environment. Facilities are available to display the user program interaction in the user's native language. This removes major language obstacles in the usage of PMSPDC.

The program handles the following for the road division and road network in the province:

1. Information pertaining to administrative affairs;
2. Construction and maintenance cost items;
3. Information on resources: manpower, equipment, materials, and budget;
4. Road identification, condition, and traffic for every link;
5. Pavement condition for every segment;
6. Priority criteria factors and PI for every link and segment; and
7. Priority list and program alternatives on the provincial and district levels.

TABLE 4 Pavement Condition Priority Factor (PCPF)

PCR	PCPF	Condition	URCR	PCPF
0-20	35	Failed	0-40	30
21-40	30	Poor	41-65	25
41-60	20	Fair	66-80	20
61-75	10	Good	81-90	15
76-90	5	Very Good	91-100	10
91-100	0	Excellent	—	—

TABLE 5 Priority Factors of Regional Development, Economic Development, and Traffic Growth

Purpose of Link	Regional Development Factor	Level of Economic Activities (measured by heavy vehicle traffic)	Economic Development Factor	Average Daily Traffic	Traffic Growth Factor
Evolution of new settlement(s)	15	Heavy traffic loads	10	> 1500	10
Help developing existing communities	10	Medium traffic loads	6	500-1500	5
Connection of existing communities	5	Light traffic loads	2	< 500	2

Analysis and Plans

On the basis of the available information and priority criteria a priority list is compiled in a computer data base for all road segments. Each segment is preliminarily reviewed and initially categorized in one of three stages: routine maintenance, rehabilitation, or construction. A cost estimate is used to provide an estimate of the needed resources in the short and long term. The agency's goals are then reviewed in light of the available information. The goals may be adjusted to be more realistic if they are not.

A weighed average of the PIs over all sections of the road network is called the network index (NI). The NI can be used as an indicator of the overall condition of the network. For the purpose of comparison a lower NI indicates a better condition of the network.

The final year-to-year plans are made in the following steps:

1. Analyze each road segment at the project level to assess the expected performance under alternate actions (usually limited to two to three alternatives).
2. Estimate the initial cost of the action for each segment by using current itemized costs.
3. Provide alternative plans of action for the first year within the constraints of the resources and check the consequences of each plan in the form of expected network performance (NI). Select the plan with the best performance.
4. Assuming that the selected plan in Step 3 is carried out, run the same procedure for the second year, and so on.

A linear pavement performance model was used in this procedure. This is considered a preliminary assumption until adequate data are collected over the years in the province to develop a more realistic model and update the procedure.

It is common practice among provincial road agencies for the budget for routine maintenance to be handled separately from the rehabilitation and construction budget. The sources of each budget may even be different. Therefore a provision is made to provide separate plans for each set of actions.

Resources

According to the developed plans, the resources required for short- and long-term implementation are determined on the basis of the local rates of productivity, techniques, costs, and performance. At this point it is appropriate to review the administrative structure, policies, and practices of maintenance and construction.

IMPLEMENTATION OF PMSPDC IN FAYOUM, EGYPT

Fayoum is a semi-oasis province (governorate) located 105 km southwest of Cairo. In 1986 the province had a population of 1.55 million, and its total area is 4,550 km^2. Administratively it is divided into five districts with five cities, 157 villages, and 1,450 small villages. The province has a mix of rural and urban communities, with the urban area population representing 23.3 percent of the total population. Agriculture is the main economic activity in the province, where 315,000 *fedans* (29 percent of the total area) are cultivated by using canal irrigation systems (*5*).

The roads in Fayoum Province are divided into three categories: national, regional provincial, and local urban. The first is managed by the national Authority of Roads and Bridges of Egypt. The third category is administered by local city or village authorities, sometimes helped by the province. The second category of regional provincial roads is the subject of the present PMSPDC implementation. The total length of this road network in 1992 was 1,099 km, of which 652 km was unpaved (earth) roads. All routine maintenance is executed by the road agency's crews through its five main district offices. Rehabilitation and construction are executed by contractors under the supervision of the agency. The annual budget level in previous years ranged between £E 3 million to 4 million (US$1 = about £E 3.4). The total manpower of the agency is 1,380, of which there are 13 engineers, 252 technicians, 244 administrative staff, and 870 laborers.

The general goals of the province's road agency were stated to emphasize social development by extending road services to every small community, serving developing economic activities, and enhancing road services to tourism activities in the province. The main development activity was identified as the cultivation of new lands in the adjacent desert.

The stage goals were set to maintain road pavements at a performance level of at least "good," complete road sections vital to the continuity of traffic flow, extend road services to at least 25 percent of the communities that did not enjoy them at present, construct new roads to serve tourism and the cultivation of new desert land, improve the safety characteristics of roads, and upgrade the performance levels of various elements in the agency.

A new consistent coding system was developed for the road network. The network was divided into links, with each link subdivided into segments that are similar in character. The average length of the segments was 2.5 km. The new coding system identified 142 roads, divided into 215 links, subdivided into 471 segments. Approximately 100 km was designated candidate new roads that serve the stage goals.

All paved roads were two-lane with an average width of 6.8 m and unpaved shoulders with an average width of 1.25 m. The average width of earth was 4.5 m. Most roads were run in agricultural land, with only 12 percent run in desert surroundings.

The road surface condition survey revealed the results shown in Figure 4. The most common distresses on paved roads were rutting and "other" cracks. On unpaved roads uneven cross sections and illegal use were most commonly found.

The average daily traffic measured and estimated ranged between 200 and 2,800 vehicles per day. The proportion of heavy vehicles ranged between 0 and 25 percent. Slow-moving vehicles constituted 21 to 70 percent of the traffic.

In the implementation of PMSPDC in Fayoum it was decided to adopt tourism attraction as a priority factor in calculating the priority index. The relative weights of 0, 6, and 10 were given to road links that have no, an indirect, and a direct influence on tourism attraction, respectively. The population distribution factor was correlated to the population distribution ratio as shown in Table 6.

Priority factors were calculated or determined for each road segment in the network. Statistical analyses were carried out to study the interrelationship among and the significance of these factors in the case of Fayoum. A sensitivity analysis was performed to assess the effect of excluding any of the factors and its influence on the priority program. The correlation coefficients are shown in Table 7. A relationship was suggested by the correlation coefficient of 0.83 between economic development and traffic growth factors. It was normal to expect heavier traffic where there was stronger economic development. However it was not the only variable that might affect traffic volume and traffic growth. There were lesser significant interrelationships between economic development and tourism attraction factors on the one hand and between traffic growth and population distribution factors on the other. These relationships might suggest that the economic development could be expressed by some of the other factors considered in Fayoum.

The significance of each priority factor was tested by investigating the effect of its absence on the priority list. It was found that the absence of any of the factors significantly affected the priority scheme of the road segments. They ranked from most to least influential as follows: surface condition, tourism attraction, population distribution, traffic growth, and economic development. Regional development had the least effect on the priority scheme. This might be attributed to the more settled nature of Fayoum Province, which is considered an older, more established province with limited potential for new regional development.

The upgrading operations were divided into five categories:

1. Routine maintenance for both paved and unpaved roads.
2. Maintenance of paved roads.
3. Overlay of paved roads.
4. Reconstruction of paved roads.
5. Paving of earth roads and new construction.

Two 10-year plans were established: one for routine maintenance, handled by the agency's crews, and the other for the other actions, to be handled by contractors under control of the road agency. Budgets were estimated for each plan by using local cost information. The plans were detailed year to year. The plans and budgets were further scheduled for each of the five districts in Fayoum.

On the basis of the 10-year plans, local rates of productivity of the workforce and equipment, and a review of the structure of administration and activities, a schedule of needed resources was organized. The needed budgets were found to be higher than originally allocated by approximately 20 percent. However, the present workforce was much higher than needed, especially in the categories of administrative staff and labor. More engineers were needed to accomplish the target plans. The excess of labor was attributed to the outdated routine maintenance practices, which were labor-intensive with poor efficiency. Surprisingly, the equipment owned by the province was found to be adequate, if it was used efficiently, for routine maintenance. Finally, it was found that there was a risk of an inadequate supply of aggregates that satisfied proper quality standards.

During the course of implementing PMSPDC in Fayoum, a general investigation of common problems that were found to face road engineers on the project level was carried out. Structural failure of pavement under medium truck loads was a main concern.

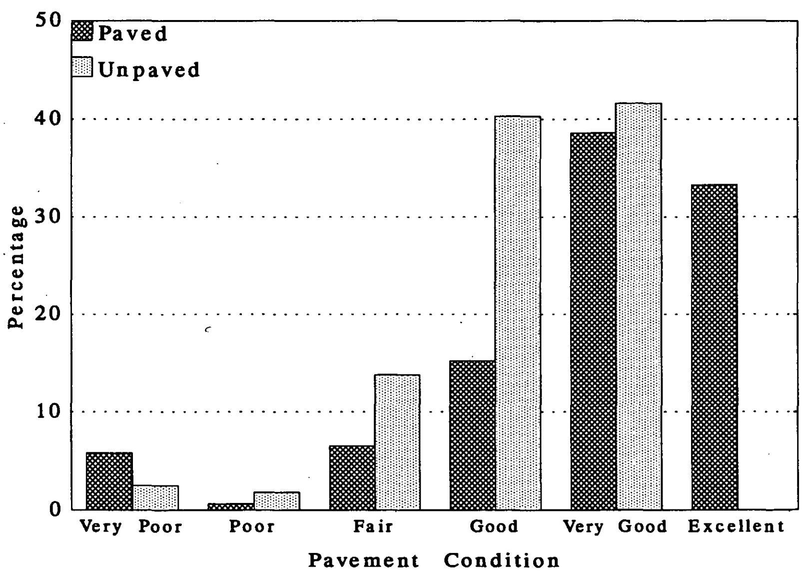

FIGURE 4 General road surface conditions in province of Fayoum.

TABLE 6 Population Distribution Factor for Fayoum

Population Distribution Ratio (10^{-5})	Population Distribution Factor
0-6	5
6-12	10
12-31	15
> 31	20

TABLE 7 Interrelation Correlation Coefficient Among Priority Factors

	f_1	f_2	f_3	f_4	f_5	f_6
Surface Conditions (f_1)	1.0	0.06	0.52	0.16	0.16	0.18
Population Distribution (f_2)		1.0	0.65	0.60	0.71	0.54
Regional Development (f_3)			1.0	0.21	0.66	0.43
Economic Development (f_4)				1.0	0.83	0.76
Traffic Growth (f_5)					1.0	0.6
Tourism Attractions (f_6)						1.0

Narrow pavements, which forced trucks in both directions to take almost the same wheelpath, were found to be responsible for double loading on pavements. This led to early failures of those pavements. The second problem was deep failure of road foundations. This was attributed to the seepage of irrigation water under the road from agricultural fields to water canals. A lack of proper subsurface drainage systems to control water movement was the reason behind the considerable loss of strength of silty soils in road foundations. Finally, slope failures of embankments were found to be caused by water seepage and encroaching agricultural fields, which forced steep side slopes. General guidelines were established to deal with these problems.

CONCLUSIONS

1. A pavement management system geared toward the needs and concerns of provinces in developing countries (PMSPDC) was developed. It is expected to be effective in reducing road maintenance costs and improving road conditions.
2. A simplified scheme for pavement surface condition survey and rating was developed.
3. Priority criteria were established to exemplify the concerns of a province in developing countries.
4. Language represents a great barrier that should be overcome before expecting an adequate response from local officials and engineers. PMSPDC was presented in native languages as a system and computer program.
5. Successful implementation of PMSPDC in Fayoum, Egypt, uncovered important facts and was well received by the road agency's officials and the high-ranking provincial officials.

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Optimality of Highway Pavement Strategies in Canada

BRUCE HUTCHINSON, FRED P. NIX, AND RALPH HAAS

The flexible pavement performance predictions of a deterioration model proposed by Small, Winston, and Evans are compared with those estimated from a deterioration model developed for Ontario conditions. The proposed model by Small et al., incorporating even a very modest rate of annual environmentally induced degradation, severely underestimates the initial service lives of Ontario pavements. The life-cycle cost characteristics of Ontario pavements are illustrated, and it is concluded that the optimal pavement strategy is insensitive to changes in the initial pavement thickness. This conclusion is in contrast to the "underbuilt" conclusions by Small et al. Typical average and marginal cost functions that must form the basis of any rational axle weight-based road user charge system are also presented.

Several years ago, Small and Winston (*1*) and Small et al. (*2*) presented an elegant analysis of the optimality of highway pavement structural designs on the U.S. primary highway system. They concluded with respect to optimal pavement durability that

> Our analysis of pavement durability, using standard economic techniques and a new statistical analysis of road test data, suggests that a substantial increase in durability could be achieved at modest cost and would lower the total costs of building and maintaining pavements over their life cycle.

Their conclusion about the optimality of pavement designs was based heavily on their reanalysis of the AASHO Road Test data. They argued that "AASHO's functional specifications and statistical estimation of the coefficients . . . were seriously flawed."

Hudson et al. (*3*) reviewed the analyses conducted by Small et al. (*2*) and performed additional analyses of the performance of the Road Test rigid pavement sections. These new analyses by Hudson et al. included data obtained from the subsequent long-term monitoring of the surviving Road Test sections that had been incorporated into the Illinois Interstate highway network. They concluded that

> the Small/Winston analysis significantly underestimated the life of thick rigid pavements. The original AASHO method overestimated the life of thick rigid pavements, but not as significantly as the Small and Winston survival analysis underestimated the life. Because of the lack of distress in the Road Test for thicker rigid pavements, the Small and Winston survival analysis of AASHO Road Test rigid sections is not valid. (*3*)

Many more of the flexible pavement sections failed at the AASHO Road Test, and the Small and Winston reanalysis of the flexible pavement performance data suggested only relatively small increases in pavement thickness to achieve optimal durability. Small and Winston noted that

> use of the AASHO equations accounts for only about one-third of the difference between optimal and current design (thicknesses); the rest must be due to failure to incorporate economic optimization into design procedure. (*1*)

The purpose of this paper is to highlight some of the findings of a study on road costs conducted for the recently completed Royal Commission on National Passenger Transportation in Canada on road costs (*4*). One of the major questions addressed in this study of Canadian road costs was the "underbuilt thesis" advanced by Small et al. (*2*).

The paper first addresses the issue of pavement deterioration models and compares the behaviors of flexible pavements estimated by a model developed for Ontario conditions with that developed by Small et al. from the AASHO Road Test, which contained some modifications for environmentally introduced pavement deterioration. The second part of the paper analyzes the life-cycle cost characteristics of pavements and identifies optimal pavement strategies. It also partitions the life cycle costs into those caused by vehicle damage and those caused by environmental degradation. The final part of the paper presents representative average and marginal cost functions along with their implications for heavy vehicle pricing.

TYPICAL FLEXIBLE PAVEMENT DESIGNS

Most of the pavements on the major highway system in Canada consist of flexible pavements. Summarized in Table 1 are the characteristics of representative flexible pavement designs for primary highways in three provinces of Canada. The thicker pavements required in New Brunswick reflect the poorer-quality subgrades in that province. Alberta has the smallest range of thicknesses because most of the pavement deterioration is caused by the harsh winter climate rather than by traffic. The thinner base course thickness in Alberta reflects the use of asphalt-stabilized base courses because of the unavailability of good granular base course material. The last row of Table 1 shows the range of equivalent granular thicknesses of the pavement using granular thickness equivalencies for the surface of 2 and for the subbase of 0.67.

PAVEMENT DETERIORATION

In Canada pavement surface quality is usually expressed in terms of the riding comfort index (RCI) rated on a 10-point scale instead of the 5-point scale of the U.S. present serviceability index (PSI). New pavements typically have an initial RCI of 8.5, and first-class highway pavements are normally considered to have deteriorated to an unacceptable condition when the RCI has decreased to 4.5.

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TABLE 1 Typical Flexible Pavement Structures

Layer	New Brunswick	Ontario	Alberta
asphaltic concrete	140-200	50-130	80-100
base course (granular or asphalt stabilized*)	150	150	50*
granular sub-base	455-760	150-450	180-330
equivalent granular thickness	750-1090	350-700	420-615

entries are in mm

Deterioration Model of Small and Colleagues

Small et al. (2) proposed the following pavement deterioration model on the basis of their reanalysis of the AASHO Road Test data and the incorporation of a term to account for environmental degradation:

$$\mathrm{RCI}(t) = \mathrm{RCI}(0) - [\mathrm{RCI}(0) - \mathrm{RCI}(f)]\left(\frac{Qt}{N}\right)e^{mt} \tag{1}$$

where

$\mathrm{RCI}(t)$ = RCI at time t,
$\mathrm{RCI}(0)$ = initial RCI magnitude,
$\mathrm{RCI}(f)$ = the RCI magnitude at failure,
Q = the annual number of ESAL coverages;
N = the total equivalent single axle load (ESAL) coverages expected from a pavement before it deteriorates to the terminal serviceability magnitude RCI (f), and
m = a parameter to account for the annual decrease in serviceability caused by climatic factors.

The Small et al. (2) reanalysis of the AASHO Road Test data for flexible pavements resulted in the following equation for predicting N, the cumulative ESAL coverages to failure:

$$N = e^{12.062}(D + 1)^{7.761}(L_1 + L_2)^{-3.652}(L_2)^{3.238} \tag{2}$$

where

D = the structural number of a pavement,
L_1 = the axle load (in thousands of lbs), and
L_2 = 1 for single axles and 2 for tandem axles.

Setting L_1 equal to 18 (i.e., the 18,000-lb standard axle load) and L_2 equal to 1, the pavement deterioration model in Equation 1 may be rewritten as

$$\mathrm{RCI}(t) = 8.5 - \left[\frac{4Qt}{3.7021(D + 1)^{7.761}}\right]e^{mt} \tag{3}$$

The value 4 in Equation 3 is the difference between RCI(0) equal to 8.5 and RCI(f) equal to 4.5.

Equation 3 can be used to develop the RCI profile of a representative flexible pavement with a structural number D of 4.9. The magnitude of m equal to 2.3 appears to be representative of the climatic-induced deterioration in Canada from the comments made by Small et al. (2). Small et al. (2) used m equal to 4.0 in most of their calculations, and Paterson (5) suggests that an m in the range of 5.0 to 10.0 might be appropriate for severe climates. There is considerable uncertainty about the appropriate m magnitude, but a value of m of 2.3 is adequate to illustrate the important features of Equation 3.

Illustrated in Figure 1 are the RCI profiles calculated from Equation 3 for D equal to 4.9 for a range of annual ESAL loadings. The diagram shows that the model does not allow for deterioration in the absence of axle loads. The RCI profiles show that the model of Small et al. (2) predicts failure in about 21 years, when the pavement experiences 100,000 ESALs per year, and failure in about 7 years, for an annual ESAL loading of 500,000.

If m magnitudes are used in Equation 3 such as those suggested for harsh Canadian winter conditions by Paterson, say m equal to 7.0, then the terminal serviceability of 4.5 would be reached in about 13 years instead of 21 years for an annual ESAL loading of 100,000.

OPAC Deterioration Model

The Ontario flexible pavement deterioration model (OPAC) provides one of the few models that separates load- from climate-induced deterioration. It was developed from the behavior observed at the AASHO Road Test, the theoretical behavior of layered elastic systems, and the longer-run Brampton Test Road in Ontario (6) Rilett et al. (7) have discussed some of the characteristics of OPAC with respect to cost allocation studies.

The RCI profiles predicted for a flexible pavement with D equal to 4.9 by using the OPAC model are illustrated in Figure 2 for the same range of annual ESAL loadings used in Figure 1. The diagram illustrates that OPAC predicts that pavements deteriorate to an unacceptable condition in about 40 years in the absence of axle loads. With annual ESAL loadings of 1 million, an RCI of 4.5 is reached in about 18 years [versus 4 years for the model of Small et al. (2)];

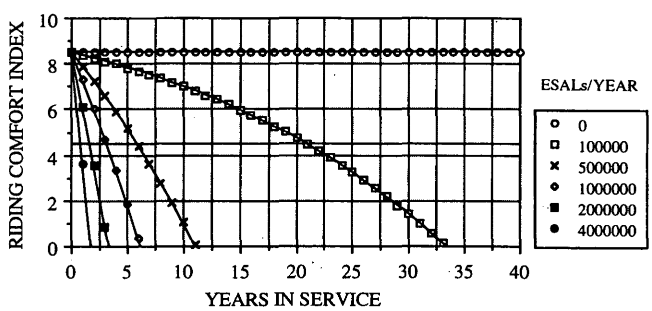

FIGURE 1 RCI versus age profiles from the model of Small et al. (2).

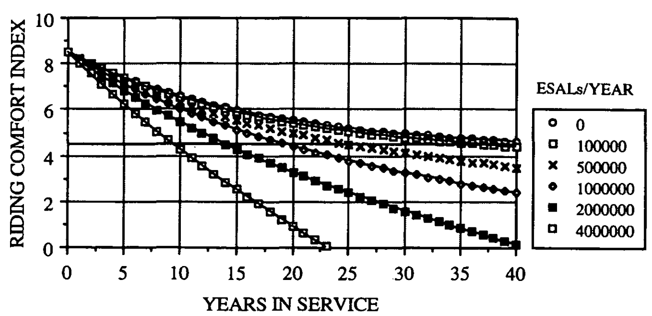

FIGURE 2 RCI versus age profiles from OPAC model.

with an annual ESAL loading of 2 million the terminal serviceability of 4.5 is reached in 14 years [versus about 2 years for the model of Small, et al. (*2*)].

Rilett et al. (*7*) have pointed out that the relative importance of axle loads and environment as sources of pavement deterioration change with different intensities of loads. For example, the environment may account for as much as 80 percent of the deterioration on low-volume roads (fewer than 250,000 ESALS per year) and for only about one-half of the deterioration on heavily loaded roads.

Differences in Failure Life Prediction

Demonstrated in the Table 2 are the differences in pavement lives predicted by the OPAC model and that proposed by Small et al. (*2*). The entries in Table 2 show the initial years to failure for a variety of annual axle load intensities by using both models. The entries confirm the differences between the two models illustrated previously; the model of Small et al. (*2*) forecasts are very sensitive to ESAL loading changes. Clearly, the economically optimum pavement strategy would be much more sensitive to the key design variable, the initial pavement life, if the model of Small et al. (*2*) rather than the OPAC model were used. Small increases in pavement thickness would have substantial payoffs if the model of Small et al. (*2*) were used to predict performance.

ECONOMIC CHARACTERISTICS

The economic characteristics of flexible pavements discussed in this section are based on pavement performance behavior forecast by the OPAC model. The pavement costs are based on unit costs for the surface course of $Can950/mm/two-lane km (US$1 = about $Can 1.35) and for the base and subbase courses of $Can200/mm/two-lane-km.

Illustrated in Figure 3 are the well-known scale economies that exist for highway pavements, in which the present worths of total costs required to achieve a 15-year and a 20-year initial service life to failure are shown as a function of the magnitude of the annual ESAL loading. The pavement performance estimates that form the basis of Figure 3 were obtained from OPAC. Small increases in pavement construction costs can accommodate the substantial increases in ESAL loadings.

Illustrated in the Figure 4 is present value of the construction, resurfacing, and routine maintenance costs for two magnitudes of annual ESAL loadings by using a discount rate of 5 percent. The life-cycle curves are derived from the initial pavement costs, the resurfacing costs required to achieve a 40-year service life (usually two cycles are required), and the routine pavement maintenance costs. The life-cycle cost curves for the high annual axle loadings are quite flat, suggesting that the choice of an initial pavement life is not critical in establishing minimum life-cycle cost thicknesses. For example, in the case of an annual ESAL loading of 2 million, there is only a 2.2 percent difference between the highest and the lowest costs. The life-cycle cost curve for annual ESALs of 250,000 does rise continually, suggesting that the optimal strategy may be to build pavements with short lives. However it should be noted that the costs used in Figure 4 do not include the delay costs to traffic induced by pavement maintenance and reconstruction operations.

Nix et al. (*4*) provide additional analyses of the life-cycle costs of flexible pavements in Canada. They concluded that for the purpose of developing costing procedures for Canadian roads there is no evidence that pavements are being built with less than optimum

TABLE 2 Years in Service Estimated by Two Performance Models

Annual ESAL Load	OPAC Model	Small, Winston & Evans Model		
		m = 0.0	m = 0.023	m = 0.07
100,000	37	34	21	13
500,000	26	7	6	5
1,000,000	20	3	3	3
2,000,000	14	2	2	1
4,000,000	10	1	1	1

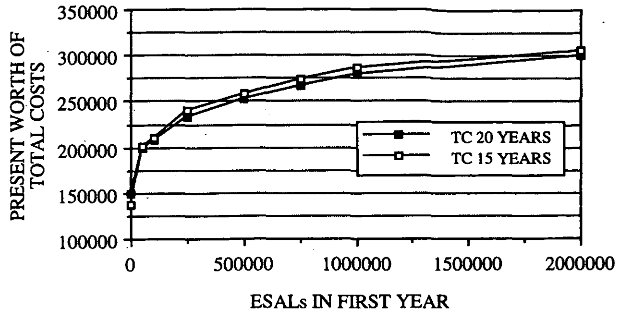

FIGURE 3 Initial construction cost versus annual ESAL loadings.

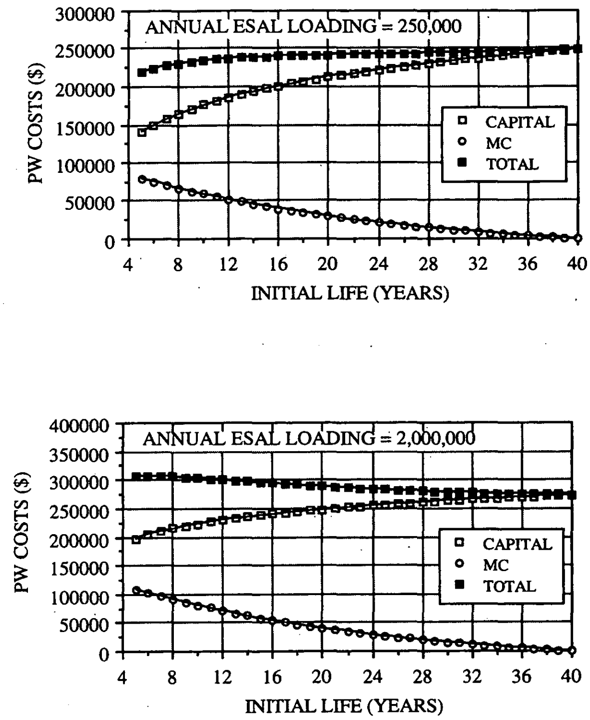

FIGURE 4 Present worth (PW) life-cycle costs versus initial pavement life.

durability. Any initial pavement life of about 15 years seems to be optimal in minimizing total life-cycle costs.

Use of the deterioration model of Small et al. (*2*) as the basis for the economic analysis would yield very different conclusions about the optimality of flexible pavement designs in Ontario. Total life-cycle costs would be higher, and the rate of change of costs with initial pavement costs with changes in initial pavement life for each level of ESAL loading would be higher.

Illustrated in Figure 5 is the present worth of the life-cycle pavement costs for an initial pavement life of 15 years for different annual ESAL loading magnitudes. The following function has been fitted to the data:

$$C = 89{,}969 + 23{,}214 \log (\text{ESALs}) \quad (4)$$

where *C* is the present worth of the life cycle costs.

Illustrated in Figure 6 is the marginal pavement cost (*MC*) function derived from Equation 4, which has the following equation:

$$\text{MC} = \frac{23{,}214}{\text{ESALs}} \times \frac{1}{\ln 10} \quad (5)$$

The average cost function is also shown in Figure 6, and because of the scale economies it falls above the marginal cost function.

The implications of these marginal cost functions for three representative Ontario truck types and for three ESAL volumes by using two methods for converting axle loads to ESALs are illustrated in Table 3. The upper number in each cell uses the AASHTO ESAL equivalencies, whereas the lower number uses the functions

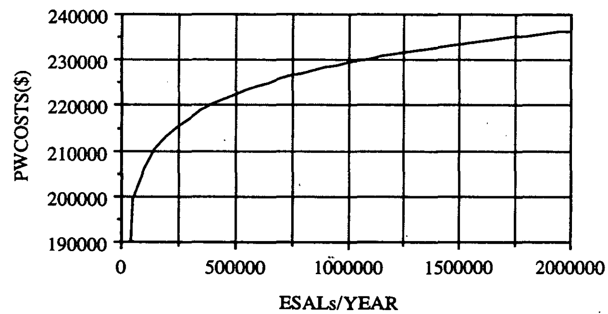

FIGURE 5 Present worth of life-cycle costs versus annual ESAL loadings.

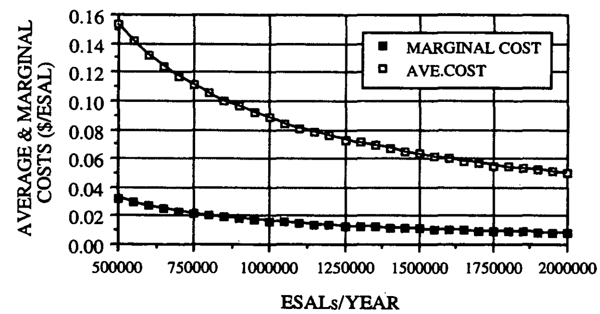

FIGURE 6 Marginal and average pavement damage costs per ESAL versus annual ESAL loadings.

TABLE 3 Marginal Pavement Costs for Several Ontario Truck Types

Truck Type	Annual ESAL Load Intensity		
	Low (50,000)	Medium (500,000)	High (2,000,000)
3 axle (25 t)	10.3 19.5	2.6 4.9	1.3 2.4
5 axle tractor-semi (39 t)	13.5 23.4	3.4 5.8	1.7 2.9
8 axle B-train (62 t)	18.6 33.1	4.6 8.3	2.3 4.1

entries are cents per kilometre

reported by Rilett and Hutchinson (*8*). The marginal costs listed in Table 3 are for the traffic volumes listed at the head of each traffic volume column and are calculated from Equation 5.

The Table 3 entries show the large variations in the marginal costs per kilometer between truck types and across road types, for example, \$0.186/km for a 3-S3-S2 operating on a road with low annual ESAL loadings versus \$0.023/km for the truck when it operates on a road with high annual ESAL loadings. Charging trucks for load-associated pavement damage at the long-run marginal cost would not recover total pavement damage costs, and a second charge would be required to recover fully the remainder of the damage costs, the nonload-associated costs, and the other costs associated with the provision of highway infrastructures.

CONCLUSIONS

The initial service lives of flexible pavements forecast by the model of Small et al. (*2*) containing a modest annual environment-induced degradation are much shorter than those observed in Ontario. The more rapid decrease in RCI estimated by the model of Small et al. (*2*) is probably caused by its use of the AASHO Road Test data and the accelerated loading of the pavements in this Road Test.

Analyses of the life-cycle costs of flexible pavements by the OPAC degradation model for a range of annual ESAL loadings showed that weak minima existed. This means that optimal pavement strategies in Ontario are not sensitive to the issue of initial pavement durability (thickness) as suggested by Small et al. (*2*).

The strong economies of scale present in flexible pavements produce long-run marginal pavement costs per ESAL that are much lower than the long-run average costs. This means that ESAL-kilometer charges for loaded trucks based on long-run marginal costs would not fully recover life-cycle costs. Other mechanisms such as Ramsey pricing, higher vehicle registration fees, charges for externalities, and charges for capacity consumption would be required to recover the additional costs of providing highways.

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Forecasting Pavement Rehabilitation Needs for Illinois Interstate Highway System

KATHLEEN T. HALL, YING-HAUR LEE, MICHAEL I. DARTER, AND DAVID L. LIPPERT

The Illinois Interstate highway network is deteriorating rapidly because of its age and heavy truck loadings. Unfortunately, the funds required for rehabilitation far exceed the available funds. The Illinois Department of Transportation (IDOT) faces many difficult decisions concerning ranking rehabilitation projects in order of priority and anticipating future pavement conditions and rehabilitation needs. To assist IDOT in making these decisions, three analyses were conducted by using the ILLINET pavement network rehabilitation management program. The first of these was an analysis of the accuracy of ILLINET's pavement condition prediction models. The second was an analysis of the remaining life of each of the more than 1,200 pavement sections in the Illinois Interstate network. The third was a comparison of the rehabilitation needs predicted by ILLINET with those in IDOT's latest multiyear program. The results of these analyses are of immediate practical use to IDOT in forecasting pavement rehabilitation needs for individual pavement sections, Interstate routes, and the entire Interstate network.

The Illinois Interstate highway system consists of about 1,750 two-directional miles of heavily trafficked multiple-lane pavements that were constructed largely between 1957 and 1980. About one-third of these pavements were originally constructed as 10-in. (25.4-cm) jointed reinforced concrete pavement (JRCP), and about two-thirds were originally constructed as continuously reinforced concrete pavement (CRCP) ranging in thickness from 7 to 10 in. (17.8 to 25.4 cm).

These pavements have performed well, despite Illinois' wet-freeze climate, poor subgrade soils, the prevalence of nondurable aggregates, and an unexpectedly high volume of heavy truck loadings. A recent survival analysis indicates that the mean life (years from construction to first major rehabilitation) of these pavements was about equal to the design life of 20 years, whereas the mean 18-kip (8.1-metric-ton) equivalent single axle loadings (ESALs) carried was three to four times higher than the design traffic (*1*).

The Illinois Interstate system is now deteriorating rapidly because of its age and the high volume of heavy truck loadings. As of 1991 about 60 percent of the system had been resurfaced, and much of the rest either is currently in need of rehabilitation or will be within the next 10 years. Unfortunately, the funds required for rehabilitation far exceed the available funds. The Illinois Department of Transportation (IDOT) faces many difficult decisions concerning ranking rehabilitation projects in priority order and anticipating future pavement conditions and rehabilitation needs.

In 1985 IDOT began working together with the University of Illinois to develop the Illinois Pavement Feedback System (IPFS). A major part of the IPFS project has been the development of the IPFS data base, which provides IDOT districts and central offices with data on design, construction, traffic, and condition of 1,263 Interstate highway sections. Although the IPFS data base is neither error-free nor complete, it is sufficiently developed for use in analyses that will provide useful answers to many of IDOT's questions. In addition to the survival analysis already mentioned, other analyses conducted with the IPFS data base include assessment of truck traffic growth rates and the development of performance prediction models.

Another major component of IPFS is the ILLINET pavement rehabilitation network management program. ILLINET uses data from the IPFS data base, decision trees, performance prediction models, and a variety of project-level and network-level management algorithms to generate feasible rehabilitation strategies (treatments and timing) for each pavement section in the Illinois Interstate network for a period of up to 10 years. The network management algorithm options available in ILLINET include analysis of needs (assuming an unconstrained budget), ranking, benefit-cost ratio, incremental benefit-cost ratio, and long-range optimization. The development of ILLINET and its capabilities have been described previously (*2,3*).

Because of the large mileage of Illinois Interstates that will need rehabilitation in the coming years and the expectation that funding for rehabilitation will be inadequate, IDOT is concerned about being able to anticipate the potential impact of insufficient rehabilitation funding on the overall condition of the network. Among the specific questions IDOT would like to answer are the following:

- How accurately can we predict the future condition of individual pavement sections and the future condition of the network as a whole?
- How uniform are the various Interstate routes in condition? Is it feasible to manage long corridors of Interstate as units, or must we continue piecemeal rehabilitation of more than a thousand short highway sections?
- How well are our rehabilitation needs met by the funds available? What will be the effect of the programmed funding level on the overall condition of the network?

Three analyses recently conducted to assist IDOT in answering these questions are described in this paper. The first of these was an analysis of the accuracy of ILLINET's pavement condition prediction models. The second was an analysis of the remaining life of each of the 1,263 pavement sections in the Illinois Interstate network. The third was a comparison of the rehabilitation needs predicted by ILLINET with those in IDOT's latest multiyear rehabilitation program. The purpose of these analyses is to demon-

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strate the practical benefit that a network rehabilitation program with ILLINET's capabilities can provide a state highway agency in quantifying rehabilitation needs and ranking rehabilitation projects in priority order.

ACCURACY OF PAVEMENT CONDITION PREDICTION MODELS

DOT evaluates pavement condition by using condition rating survey (CRS) values, which are assigned by panels of expert raters in field inspections conducted in even-numbered years. CRS is the key pavement condition indicator that is used for planning, programming, and scheduling highway pavement improvement projects. Pavements are rated on a 1 to 9 scale on the basis of the distress observed. The best rating is 9, which is assigned to a newly constructed or resurfaced pavement. For guidance in assigning CRS ratings, panel members consult a manual that illustrates various pavement types and conditions with photographs accompanied by distress descriptions and CRS ratings.

In general, a pavement with a CRS value that falls below 6 would be programmed by IDOT for rehabilitation within the next 5 years. However, many sections have CRS ratings below 6 because their rehabilitation must be deferred because of a lack of funds. Some pavements require considerable maintenance to keep the CRS above 5; below this level ride quality is generally very poor, and maintenance needs become more extensive.

CRS Models

ILLINET contains models to predict CRS for the following pavement types:

- JRCP,
- CRCP, and
- Asphalt concrete (AC) overlay of JRCP (JROL) and CRCP (CROL).

Each predictive model was developed from in-service pavement condition data. After considerable evaluation of different possible model forms, the following functional form was selected for the CRS models:

$$CRS = 9 - 2 \cdot a \cdot THICK^{b} \cdot AGE^{c} \cdot CESAL^{d} \quad (1)$$

This nonlinear model form may also be expressed in the following linear form by logarithmic transformation:

$$\log_{10}(9 - CRS) = 0.301 + \log_{10} a + b \cdot \log_{10}THICK + c \cdot \log_{10}AGE + d \cdot \log_{10}CESAL \quad (2)$$

where

CRS = panel condition survey rating (1 to 9),
THICK = slab thickness for JRCP or CRCP and overlay thickness for AC overlay,
AGE = years since construction or overlay,
CESAL = accumulated million ESALs in outer lane since construction or overlay, and
a, b, c, d = constants for each pavement type (Table 1).

TABLE 1 Constants for CRS Model Prediction

MODEL	CRS MODEL CONSTANTS			
	$\log_{10} a$	b	c	d
JROL, CROL	-0.4185	-0.1458	0.5732	0.1431
JRCP	1.7241	-2.7359	0.3800	0.6212
CRCP	0.7900	-1.3121	0.1849	0.2634

CRS Model Calibration

Within a certain climatic range (i.e., Illinois conditions) pavements of a certain type and design can be expected to exhibit a general trend in condition as a function of time and traffic loadings. However, even pavements of a single type and design can exhibit highly variable performances. Therefore, the prediction model must be calibrated to the observed condition of a specific section to accurately predict the performance of that section.

In other words if the actual current condition of a given section differs from the CRS predicted by the model (as it almost certainly will, because the model describes the mean performance of all sections of that pavement type), then the prediction curve must be adjusted to match the actual value. If this calibration is not done future conditions predicted by the model for that section will not be reasonable.

Two different methods for prediction model calibration are available. The first method basically involves shifting the prediction curve upward or downward so that it passes through and extrapolates from the actual known pavement condition (e.g., CRS). The extrapolated curve is parallel to (and thus predicts the same rate of deterioration as) the mean curve. This approach inherently assumes that the data on age and past accumulated traffic are accurate but that the specific section's performance differs from the predicted mean performance.

The second calibration method uses the actual current condition (e.g., CRS) and the current annual traffic level to "backcast" values for the age or past accumulated traffic inputs, which will predict a condition level matching the actual value. This method, which shifts the mean curve horizontally forward or backward until it passes through the actual known condition level, is particularly appropriate when the accuracy of the age or past traffic data are questionable.

This latter calibration method is currently used in ILLINET because of the uncertainty associated with estimating accumulated ESALs. The current annual ESALs in the outer traffic lane may be estimated more reliably from current or recent counts of the average daily traffic, single-unit trucks, and multiple-unit trucks. A direct relationship is assumed to exist between pavement age, annual ESALs (ESALPYR), and cumulative ESALs:

$$CESAL = AGE \cdot ESALPYR \quad (3)$$

The CRS model for a given pavement type may be calibrated to the current condition of any given section of that type in any year by calculating the following two calibration constants:

$$C_1 = \left(\frac{9 - CRS}{2 \cdot a \cdot THICK^{b} \cdot ESALPYR^{d}} \right)^{\frac{1}{c+d}} \quad (4)$$

$$C_2 = C_1 \cdot \text{ESALPYR} \qquad (5)$$

Once the model has been calibrated to the current condition of the section, the condition of the section in any future year may be predicted as a function of the change in the age of the pavement in years (ΔYEAR) and the change in millions of accumulated ESALs (ΔCESAL) over that time period:

$$\text{CRS}_{\text{future}} = 9 - 2 \cdot \text{a} \cdot \text{THICK}^{b} \cdot (C_1 + \Delta\text{YEAR})^{c} \cdot (C_2 + \Delta\text{CESAL})^{d} \qquad (6)$$

The increase in millions of accumulated ESALs over some future time period is computed by using the current annual ESALs (ESALPYR), the length of time (ΔYEAR), and an assumed annual ESAL growth rate. A compound growth rate of 6 percent is used as a default in ILLINET, although this value may be changed at the user's discretion.

Accuracy of CRS Prediction for Pavements Without D-Cracking

The first step in assessing the accuracy of the CRS prediction models was a comparison of the 1992 CRS values predicted by the models with the actual 1992 CRS values assigned by the expert rating panels. This was done by using CRS history, pavement design, and traffic information retrieved for each of the 1,263 Interstate sections in the IPFS data base.

For each section the appropriate model for the pavement type was calibrated to the actual 1990 CRS, and the CRS was projected from that point assuming a 6 percent compound growth rate in ESALs. This comparison showed that the models predicted CRS well from 1990 to 1992 for bare CRCP, bare JRCP, AC-overlaid CRCP, and AC-overlaid JRCP without D-cracking. The results are shown in Figures 1, 2, 3, and 4, respectively.

To assess how many years into the future the CRS models could provide accurate predictions, the comparison of predicted and actual 1992 CRS values was repeated with models calibrated to 1988 CRS data and then to 1986 CRS data. Sections that were rehabilitated between the starting year and 1992 were excluded from the analysis. The results for pavements without D-cracking indicate that the models' predictive accuracies are good even for 6 years into the future. Analysis of the models' accuracies for longer time periods could be done, but there is a limitation: the predicted and actual

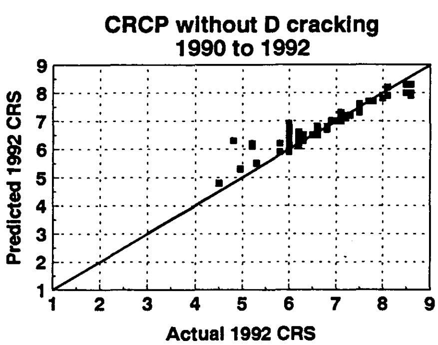

FIGURE 1 Predicted versus actual 1992 CRS for CRCP without D-cracking.

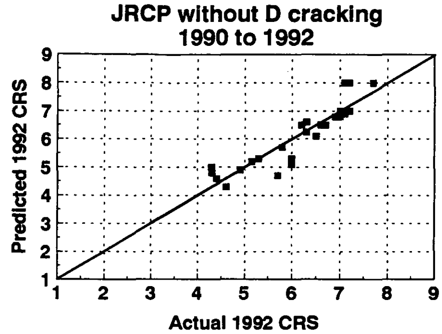

FIGURE 2 Predicted versus actual 1992 CRS for JRCP without D-cracking.

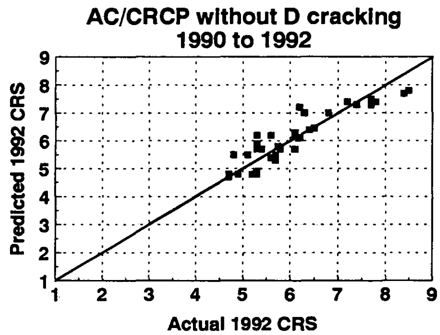

FIGURE 3 Predicted versus actual 1992 CRS for AC-overlaid CRCP without D-cracking.

CRS values can be compared only for sections that do not receive any rehabilitation during the time period considered. For periods of 8 years or more the number of sections available for use in the analysis becomes considerably smaller.

Accuracy of CRS Prediction for Pavements with D-Cracking

The drop in CRS from 1990 to 1992 was generally greater for D-cracked pavements than the models predicted. When the CRS models were developed in 1986 a D-cracking variable was not in-

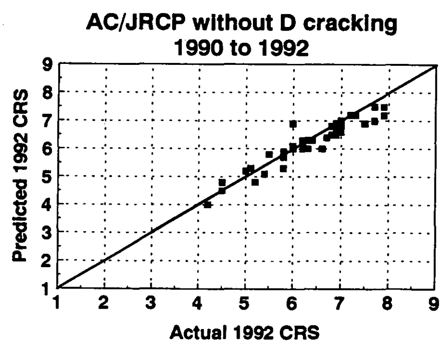

FIGURE 4 Predicted versus actual 1992 CRS for AC-overlaid JRCP without D-cracking.

cluded, primarily because the D-cracking data contained in the IPFS data base at that time were not considered sufficiently reliable.

In 1991 a thorough review of the D-cracking data in the data base was conducted by using distress survey results, materials records, and previous research results. That review was done to conduct survival analyses of bare and resurfaced concrete pavements in Illinois with and without D-cracking (*4*). One finding of the survival analysis was that both bare and overlaid pavements without D-cracking lasted longer and carried more truck traffic than D-cracked pavements of the same type and thickness. The mean life (age and accumulated ESALs) was 20 to 50 percent higher for non-D-cracked pavements than for D-cracked pavements of the same type and thickness.

To account for the more rapid deterioration of D-cracked pavements, an analysis was conducted to determine an appropriate adjustment that could be applied to the predicted rate of loss in CRS. This was done for four pavement categories (bare JRCP, bare CRCP, AC-overlaid JRCP, and AC-overlaid CRCP, all with D cracking) by comparing the predicted with the actual 1992 CRS by using CRS data sets from 1990, 1988, and 1986. The following adjustment factors were found to give the best fit over the time ranges considered:

Adjustment Factor	*Pavement Category*
1.2	Bare JRCP
1.2	AC-overlaid JRCP
1.2	AC-overlaid CRCP
1.5	Bare CRCP

An alternative to applying these adjustment factors to the rate of CRS loss for D-cracked pavements would be to repeat the regression of the CRS models with an additional term for D-cracking. However, the use of adjustment factors may be preferable because IDOT personnel will be able to modify the factors as needed in future years to maintain a good fit of predicted to actual CRS without having to conduct nonlinear regression analyses to modify the CRS models themselves.

REMAINING LIFE ANALYSIS

ILLINET was also used to predict the remaining life of each section of the Illinois Interstate network. The purposes of this analysis were to assess the overall health of the network and to examine the variabilities in the remaining lives of pavements along the various Interstate routes. This knowledge would be useful to IDOT in assessing the feasibility of identifying corridors of multiple sections that could be brought up to uniform condition and subsequently managed as units in terms of future rehabilitation decisions.

Selection of Critical CRS

The "remaining life" of each Interstate section, defined as the number of years from 1993 until the section reached a CRS of 6.0, was predicted by using the CRS models, calibrated to the 1992 CRS and adjusted for D-cracking as described before, and assuming a 6 percent compound ESAL growth rate. This analysis was then repeated by using a CRS of 5.1, which IDOT personnel believed might represent more realistically the level at which a pavement was likely to be rehabilitated (considering the typical budget limitations), even though a CRS of 6.0 was the level at which rehabilitation would be desirable. Of course, the estimate of remaining life depends on the critical CRS selected.

Effect of Maintenance on CRS Prediction

The prediction of the number of years remaining to a CRS of 6.0 is reasonable in most cases; however, the prediction to lower CRS levels for any given section is highly dependent on the level of maintenance applied. Many sections of Interstate highway receive extensive maintenance to keep the pavement in service until rehabilitation can be done. The CRS histories of such sections fluctuate between about 5 and 6 for several years, despite a previous steady decline from 9 to about 6. Of course it is difficult to predict accurately the rate of deterioration for such sections.

Remaining Life of Interstate Routes

The results of the remaining life analysis were plotted by Interstate route and direction. The results for portions of I-55 and I-70 are shown in Figures 5 and 6, respectively, as examples. The heights of the bars indicate the remaining life in years. The numbers on the horizontal axis are mileposts, rounded to the nearest mile, given for reference.

Some Interstate routes show reasonable uniformity in remaining life, whereas others show large variations. I-55 is an example of a route with large variations in remaining life. The nonoverlaid pave-

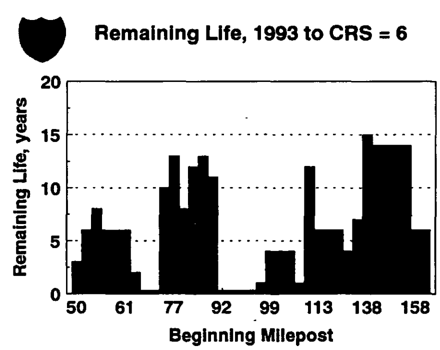

FIGURE 5 Remaining life of pavement sections along portion of Interstate 55.

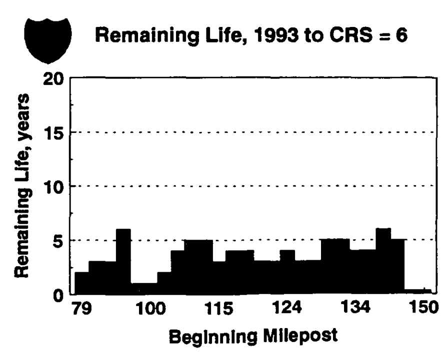

FIGURE 6 Remaining life of pavement sections along portion of Interstate 70.

ment sections represented in Figure 5 range in age from about 15 to 30 years, and the overlays on some sections range in age from about 3 to 12 years. About half of the sections have D-cracking, and thus have shorter predicted remaining lives than sections of similar design and traffic that do not have D-cracking. Some large differences in remaining life by direction are also evident for some sections.

Among the routes with more uniform remaining lives, some have fairly long and others have fairly short remaining lives. I-70 is an example of a route with a uniformly short remaining life: the sections illustrated by Figure 6 are primarily 8-in. (20.3-cm) CRCP with some 10-in. (25.4-cm) JRCP, constructed between 1960 and 1972. Nearly all of these pavements have D-cracking, which, combined with the heavy truck traffic on I-70, has resulted in considerable deterioration of the concrete. All of these sections have been overlaid at least once since 1980, and some have been overlaid three times. It is understandably discouraging to IDOT planners and district engineers to contemplate the future rehabilitation needs of such a long stretch of a heavily trafficked Interstate that, despite frequent rehabilitation and nearly constant maintenance, has only a few more years of remaining life.

Future Analyses of Remaining Life by IDOT

The remaining life analysis capability was added to the ILLINET program so that in future years this analysis can be repeated easily by IDOT personnel for the entire network or specific routes. The user needs only to select an ESAL growth rate and a critical CRS. The standard keyboard "page up" and "page down" keys are used to move through the Interstate route graphs displayed on the computer screen, and once a printer has been selected, the "shift" and "print screen" keys are used to print the displayed graph.

ANALYSIS OF REHABILITATION NEEDS VERSUS IDOT PROGRAMMING

The third analysis conducted was a comparison of the rehabilitation needs predicted by ILLINET and IDOT's proposed multiyear rehabilitation program. This analysis has actually been conducted four times: first with IDOT's improvement program for fiscal years 1991 to 1995 and then for 1992 to 1996, 1993 to 1997, and most recently with the 1994 to 1998 program.

Proposed Highway Improvement Program

The multiyear program itemizes IDOT's proposed expenditures for Interstate highways, state highways, and other facilities in several areas, including pavement rehabilitation, bridge rehabilitation or replacement, major highway construction, and safety improvements. The programmed expenditures considered in this analysis were those for resurfacing and reconstruction of Interstate pavement sections. Programmed expenditures for patching, interchange reconstruction, and bridge reconstruction were excluded.

Rehabilitation Needs Analysis with ILLINET

One of several pavement network management algorithms programmed in ILLINET is the needs algorithm, which estimates the rehabilitation needs for up to 10 years into the future, assuming no yearly budget constraint. Every section in the network whose condition falls below a user-defined minimum CRS is a candidate for rehabilitation. The type of rehabilitation is determined by selection of one of several available options for project-level rehabilitation (2). For this analysis the needs algorithm was run by using a single thickness of asphalt resurfacing as the sole rehabilitation strategy. In fact the rehabilitation type is not significant to this analysis, the purpose of which is to predict the timing of rehabilitation, not the cost. The analysis was run for three critical CRS levels: 6.0, 5.5, and 5.1.

Comparison of Rehabilitation Needs with Program by Route

The sections with rehabilitation needs identified by ILLINET and the sections programmed for rehabilitation by IDOT were graphically displayed by Interstate route and direction. A comparison for portions of I-74 and I-80 are shown in Figures 7 and 8, respectively, as examples. For each direction the sections needing rehabilitation according to ILLINET are represented by the bars above the line representing the route, and the sections actually programmed by IDOT for rehabilitation are represented by the bars below the line. The numbers next to the bars indicate beginning and ending mileposts; these are followed in parentheses by the year that rehabilitation is needed or programmed.

A summary of the mileage of rehabilitation needs identified by ILLINET and the programmed rehabilitation mileage is provided in Table 2. This summary indicates that the rehabilitation work pro-

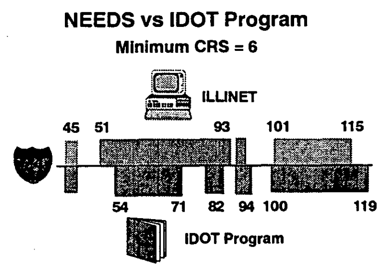

FIGURE 7 Rehabilitation needs (from ILLINET) versus rehabilitation programmed (from IDOT 1994–1998 program) for portion of Interstate 74.

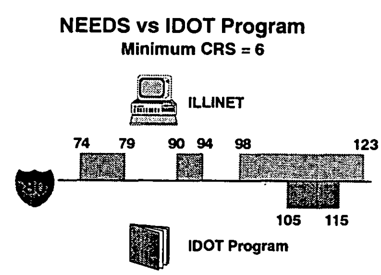

FIGURE 8 Rehabilitation needs (from ILLINET) versus rehabilitation programmed (from IDOT 1994–1998 program) for portion of Interstate 80.

TABLE 2 Summary of Rehabilitation Needs Versus Rehabilitation Program

District	ILLINET Needs			IDOT Programmed Miles	Total Miles Analyzed
	CRS = 6.0	CRS = 5.5	CRS = 5.1		
1	237.17	170.31	153.14	153.81	412.59
2	160.10	114.71	51.55	95.46	320.10
3	196.55	123.85	93.73	117.73	476.63
4	122.83	91.09	72.90	107.27	207.26
5	256.03	146.81	112.90	172.49	510.82
6	106.01	43.57	32.78	62.54	246.56
7	263.95	143.98	113.28	131.63	405.93
8	117.78	86.77	37.62	37.62	352.16
9	110.17	53.82	7.91	7.91	229.88
Total	1570.59	974.91	675.81	939.23	3161.93

Notes:

1. All miles are one-directional.
2. Ratio of miles programmed by miles needed (for critical CRS = 6.0) is 939.23 / 1570.59 = 0.60, or 60 percent.
3. District 2 has one resurfacing project programmed on I-180 (mileposts 5.43 to 9.76, both directions), which was not included in this comparison because I-180 is not currently in the IPFS database.
4. Only resurfacing and reconstruction projects programmed for 1994-1998 were considered in this comparison. Patching, interchange reconstruction, bridge reconstruction, etc. were excluded. Some projects let for bids recently may not be included. The latest bid letting information available was December 1992.

grammed by IDOT with the anticipated available funds is only about 60 percent [939 versus 1,570 mi (1502 versus 2512 km)] of the needs identified by ILLINET to keep all sections of the Interstate above a CRS of 6.

If additional funding is not available a large percentage of Interstate sections are predicted to fall below a CRS of 6.0 over the next 5 years. If the funds available for rehabilitation continue to fall short of the amount required to keep the pavements in acceptable condition the backlog of deficient pavements will continue to grow. This will result in substantial maintenance expenditures and probably more costly rehabilitations as well. Of course what constitutes an acceptable pavement or a deficient pavement depends on the target CRS level selected.

At a critical CRS of 5.5 the ratio is about 96 percent [939 versus 975 mi (1502 versus 1560 km)], and at a critical CRS of 5.1 the programmed mileage exceeds the needs indicated by ILLINET by about 39 percent [(939 versus 676 mi (1502 versus 1082 km)]. These results suggest that the rehabilitation funds programmed over the next 5 years should be sufficient to keep nearly all sections of the Interstate network above a CRS of 5.5 over that time period.

Limitations of Needs Algorithm

ILLINET's needs algorithm was used in the present analysis to identify projects that will reach the selected critical CRS and determine the total mileage of these projects. This algorithm was run by using resurfacing as the single rehabilitation strategy. Hypothetically the budget for rehabilitation is unlimited, so a section is resurfaced as soon as it reaches the critical CRS. This algorithm, particularly when it is run with a single rehabilitation strategy, does not necessarily develop the optimum rehabilitation plan for the network.

Indeed, what is an "optimum" plan depends on what benefit one chooses to maximize or what cost one chooses to minimize. The needs algorithm seeks to eliminate the mileage of deficient pavements. It may do this in a way that is not the most cost-effective for particular sections or for the network as a whole. For example, a severely deteriorated pavement that continues to deteriorate rapidly probably should not be resurfaced every few years; some longer-lasting rehabilitation strategy would be more cost-effective. Other analyses conducted for this research study and described in a separate paper indicate that very different network rehabilitation programs may be developed depending on the network-level management algorithm selected (*5*). For example, in another analysis conducted by using ILLINET, the incremental benefit-cost ratio algorithm produced a network rehabilitation program with the same total cost (in millions of dollars) as the needs algorithm, but with a 50 percent improvement over the needs analysis in vehicle-miles traveled on good pavements. This is because the incremental benefit-cost algorithm may pick more costly rehabilitation strategies for some sections if they are more cost-effective for the network as a whole and also will favor rehabilitation of higher-volume routes, because the benefit that it seeks to maximize is vehicle-miles traveled on good roads.

Future Program-Versus-Needs Analyses by IDOT

The capability of comparing IDOT's multiyear improvement program with the results of the needs analysis was added to the ILLINET program so that in future years this analysis can be repeated easily by IDOT personnel for the entire network or for specific routes. The multiyear program of pavement rehabilitation and reconstruction projects simply needs to be entered into an ASCII input file with route, direction, and beginning and ending milepost data. The user has only to select an ESAL growth rate and a critical CRS.

CONCLUSIONS

The Illinois Interstate highway network is deteriorating rapidly because of its age and heavy truck loadings. Unfortunately, the funds required for rehabilitation far exceed the available funds. IDOT faces many difficult decisions concerning the ranking of rehabilitation projects in priority order and anticipating future pavement conditions and rehabilitation needs.

To assist IDOT in making these decisions three analyses were conducted by using the ILLINET pavement network rehabilitation management program. The first of these was an analysis of the accuracy of ILLINET's pavement condition prediction models. The second was an analysis of the remaining life of each of the more than 1,200 pavement sections in the Illinois Interstate network. The third was a comparison of the rehabilitation needs predicted by ILLINET with those in IDOT's multiyear program.

The analysis of the CRS prediction models showed that future pavement conditions could be predicted with acceptable accuracy for several years into the future. The rate of deterioration for bare and overlaid concrete pavements with D-cracking, which is more rapid than for pavements without D-cracking, could be more accurately predicted by using the adjustment factors determined in the present analysis. However, the effect of maintenance on pavement condition is difficult to predict.

The analysis of the remaining life of the Interstate routes demonstrated considerable variability along some routes and more uniform remaining life along others. This type of information is needed to assess the feasibility of identifying corridors of entire routes or major components of routes that could be brought up to uniform condition and subsequently managed as units in terms of future rehabilitation decisions.

The comparison of rehabilitation needs indicated by the ILLINET software with those in IDOT's multiyear improvement program demonstrated that for any selected critical CRS level a section-by-section and route-by-route comparison of rehabilitation needs and rehabilitation funding could be made. In that analysis the IDOT program met only about 60 percent of the indicated needs when the critical CRS was set at a level below which IDOT personnel generally consider rehabilitation desirable. What constitutes an acceptable or a deficient pavement depends on the critical CRS selected. However, even when rehabilitation costs are deferred because of budget limitations, maintenance costs continue to accrue and increase greatly as the pavement deteriorates.

The purpose of these analyses is to demonstrate the practical benefit that a network rehabilitation program with ILLINET's capabilities can provide a state highway agency in quantifying rehabilitation needs and ranking rehabilitation projects. The graphical displays and graphical printed outputs are useful in communicating the analysis results to the central office and district personnel responsible for rehabilitation planning and programming.

The ILLINET software has also been modified to facilitate these analyses being repeated in the future by IDOT personnel. This represents another step in development of IPFS: after development of the data base, after the retrieval of data for specific analysis demonstrations, and after demonstrating the practical value of the analysis results, user-friendly tools to do those analyses should be put into the hands of the IDOT planners and engineers responsible for pavement rehabilitation decision making. A reliable and accessible data base, reliable performance prediction models, and the tools required to do the analyses needed to support decisions are the essential elements of a dynamic feedback system for continuously improved pavement performance and efficient, cost-effective pavement network management.

ACKNOWLEDGMENTS

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Maintenance Planning Methodology for Statewide Pavement Management

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After the successful implementation of a pavement management information system (PMIS) for a district road network the Mississippi Department of Transportation undertook a project to extend the system to the state network of more than 12,000 mi. An overview of systems development and the major products is presented. The system incorporates a number of microcomputer programs for data management functions, user interface, analysis, and reporting. The system was developed by using the FOXPRO data base software. Four major subsystems are identified: (*a*) a pavement management system inventory and monitoring data base, (*b*) condition data analysis accomplished through interface program, (*c*) a maintenance, planning, and budgeting (MPB) program, and (*d*) a priority ranking of rehabilitation projects. The condition data are collected with automated equipment. The raw condition data are used to calculate (*a*) distress quantities by severity in each section and (*b*) two composite measures, visual pavement condition evaluation rating and pavement condition rating. The MPB program selects two alternative strategies if the road condition warrants major maintenance activity. It also calculates the agency cost and vehicle operating cost. The fourth subsystem ranks the pavement network, relying on traffic volume, condition, and functional classification, to produce a list of projects for the annual work program. Pavement management reports can be summarized for the network as a whole, each district, each county, or individual sections.

Mississippi has a state-maintained highway network of more than 12,000 mi. It is essential to maintain the network in a serviceable and safe condition at a minimum cost to both the agency and the road users. To adequately meet this responsibility management requires well-documented information to make defensible decisions on the basis of sound principles of management and engineering. A pavement management system (PMS) helps engineers in making informed decisions. PMS is defined (*1*) as "a set of tools or methods that (can) assist decision makers in finding cost-effective strategies for providing, evaluating and maintaining pavements in a serviceable condition." Pavement management should be viewed as a part of an overall highway management system comprising a highway information system and several application modules or decision support models. The development and implementation of a comprehensive pavement management system suitable for the Mississippi Department of Transportation (MDOT) are described in this paper.

The development and simultaneous implementation of PMS were planned with the following objectives:

- Establish and maintain a computerized data base of inventory, traffic, and other field data for analysis;
- Maintain a data base of the conditions and histories of pavements;
- Integrate inventory, traffic, and condition data to form a PMS data base for easy and efficient use, with a provision to update them with new data as and when they are available;
- Develop an interface program to access and convert the raw data from the field and store them in a suitable format for easy access in all maintenance and budget analyses;
- Develop, with a provision to update, guidelines for triggering maintenance action and policies to remedy the distresses in each type of pavement;
- Evaluate vehicle operating cost;
- Establish a ranked list of sections requiring maintenance treatment annually;
- Estimate budget requirements for implementing the annual maintenance work program; and
- Set up a feedback mechanism from field personnel to modify/improve the overall PMS activities.

The MDOT PMS is a customized system that assists the department in preparing network-level highway maintenance programs and budget analysis reports. The modules of MDOT PMS and their logical structure are shown in Figure 1. Not all modules shown in Figure 1 are integrated in this phase of the study; those marked by broken lines will be pursued in a subsequent project. For discussion purposes the basic activities may be categorized under four "building blocks" on which the entire system rests. These basic blocks include

- Data base, inventory, and monitoring data;
- A condition data analysis/interface program;
- A PMS analysis and maintenance planning and budgeting (MPB) program; and
- A priority ranking and an annual work program.

In the following sections each of these topics is described giving the specifics, for example, the data required, the analysis routines, and anticipated output reports.

PMS DATA BASE

The data base, an organized collection of information or data, is an important element of a PMS and the core of the system in which all the inputs are loaded in a suitable and ready-to-use format. Quick access to the data base is crucial to the efficiency of pavement management. Easy updating, fast retrieval, the capability of expanding the data base for new data categories, and the capability of combin-

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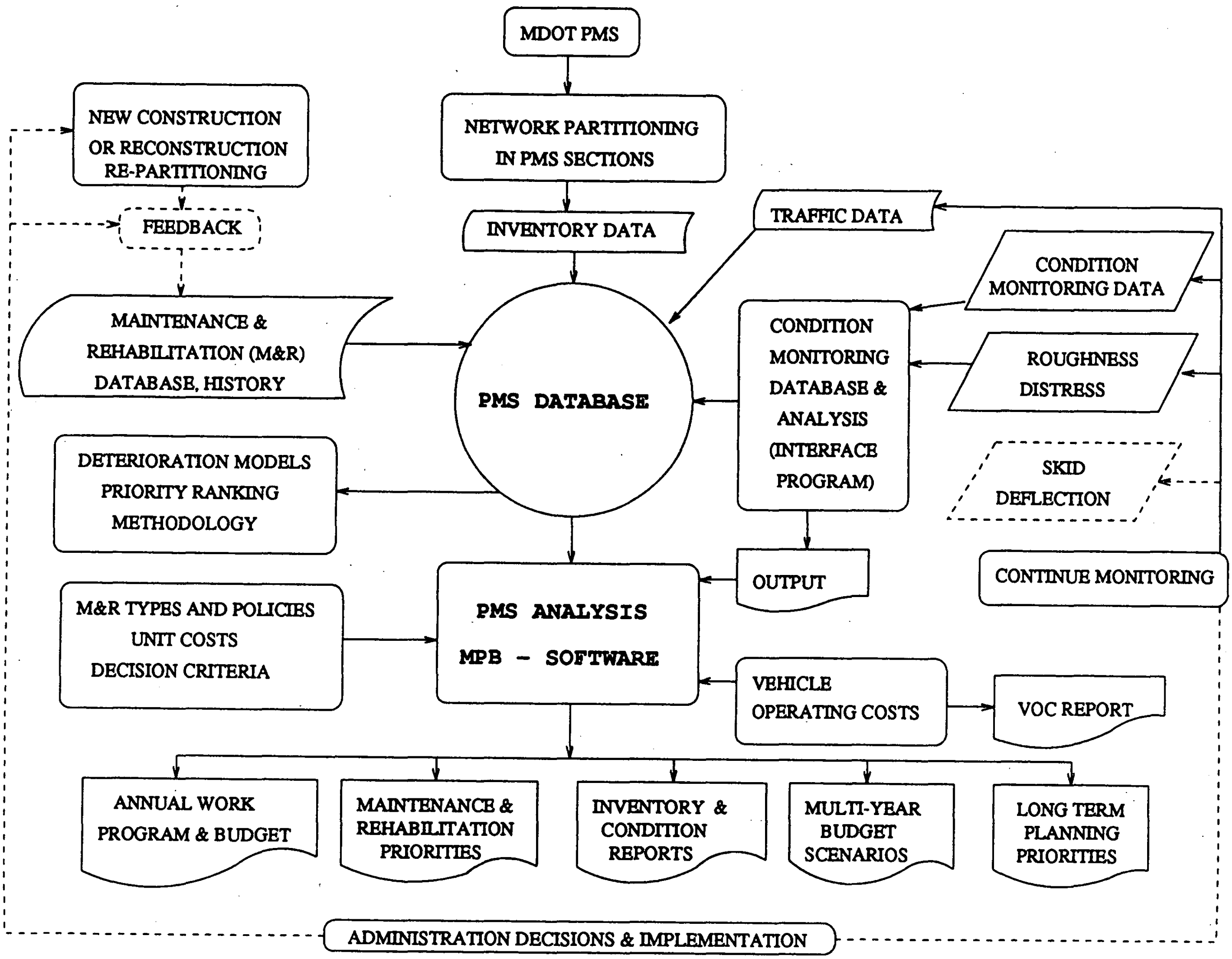


FIGURE 1 MDOT PMS development and implementation program.

ing the files to create various and informative output listings for the management level of any organization are some of the requirements of a data base. After a careful study of several data management software packages, the FOXPRO data base was selected for the MDOT PMS. FOXPRO has pull-down menus, pop-up dialogues, list boxes, and scroll bars. It is menu driven, and the table to be viewed appears in a moveable and sizeable window.

Although the entire PMS data base consists of only one file, it can include as many records as the number of homogeneous sections in the network. For purposes of bookkeeping seven subfiles—testdata.dbf, testorgm.dbf, overlay.dbf, traffic.dbf, rating.dbf, memo.ndx, and password.ndx—were developed to store the appropriate data elements. Essential historical information constitutes the first three subfiles. Items contained in the fourth subfile (traffic subfile) are average daily traffic (ADT), annual growth rate, percentage of trucks, and 18-kip equivalent single axle load (ESAL) factor for both flexible and rigid surfaces. A condition rating subfile for each section includes a distress rating (DR) calculated from the raw coded ratings for each of several distress categories, together with a roughness rating (RR) derived from the road roughness. Rut depth collected by direct measurements is another field in the condition rating file. More about the condition rating procedure and how raw data are reduced will be discussed in a later section.

Each piece of information or field in the data base is identified with what is known as a *homogeneous section* or simply a *section*. A homogeneous section is a segment of highway having basically the same roadway characteristics, for example, geometrical and structural compositions, pavement type, age of surfacing, and traffic.

All seven subfiles are linked together by a unique section identification field with the acronym SECIDNUM. The SECIDNUM consists of route number, county number, direction of travel, and beginning log mile of the section. Through the SECIDNUM only the different subfiles can be accessed or merged to generate numerous other blocks of information.

Inventory Data Base

The inventory data base contains data on the physical features of pavements and has 103 attributes for each section. These include geometric data, such as the total number of lanes, the width of the pavement, and the widths of shoulders; material data, such as the type of material used in different layers and type of subgrade; structural data, such as the thicknesses of various layers; and overlay data and the rehabilitation histories of the sections.

Making use of the capabilities of FOXPRO, a variety of summary reports of the network can be presented. A typical plot showing the distribution of pavement types is shown in Figure 2.

The traffic data base includes ADT, percentage of trucks, growth rate, cumulative ESAL, and so on, and has 36 data elements for each section. There is a memo pad data base in which remarks about a section can be stored against the section identification number. For the sake of security of the entire data base, a password data base has been established. That data base requires details about the user, such as name, district, address, password, and so on.

Monitoring Data Base (Condition Data Base)

The condition data collected from the field include distresses in pavements, including rutting and roughness of the road. The data are obtained from the outside lane in both directions for multilane divided roadways and in an easterly or northerly direction for two-lane or multilane undivided roadways. The traffic in the two directions on two-lane roads is considered to be the same, as is the load distribution. All the pertinent distresses are surveyed for 100 percent of the design lane by using high-speed automated equipment. The distress manifestations are photographed by using five high-resolution electronic color video cameras capturing different views. Office evaluation and reductions in field distress (extent and severity) are attempted only for two 500-ft sample segments per mile of the section unless section length is less than 0.5 mi. The video images are analyzed in the office by experienced personnel who document the distress type, severity, and extent. The distress type and severity are coded in accordance with the Mississippi Distress Manual, much like the Strategic Highway Research Program Long-Term Pavement Project manual, with revisions to include the distress extent as well. The distresses are then extrapolated for the whole section. Rutting is calculated from elevation measurements obtained by using five ultrasonic sensors mounted on the vehicle. Faulting is also measured by elevation measurements obtained with ultrasonic sensors. Illustrated in Figure 3 is the distribution of alligator cracking for the District 2 data base.

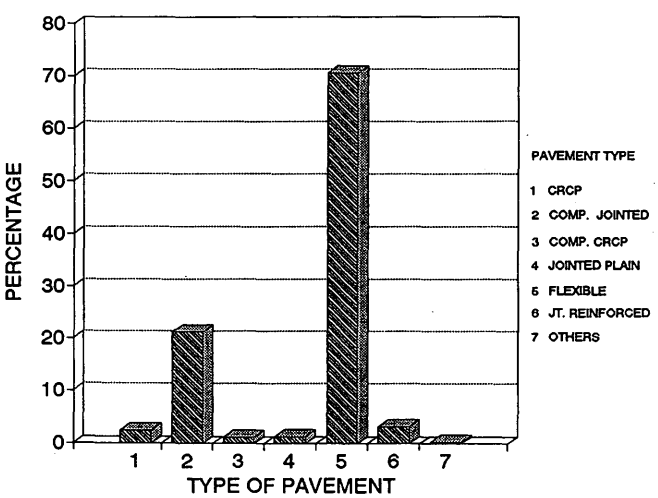


FIGURE 2 Percentage of different types of pavements.

MDOT PMS uses a South Dakota-type profiler for roughness measurements. The roughness data are collected longitudinally in both wheelpaths for 100 percent of the road lane surveyed. The ride quality is expressed in terms of the international roughness index (IRI). The measurements from the South Dakota-type profiler are used in a computational procedure called quarter-car simulation, which represents the response of a single wheel yielding IRI (in m/km).

The condition data base contains 30 attributes indicating the ratings of the sections. It includes roughness rating, distress rating, pavement condition rating (PCR), and survey year. Ride quality, designated as RR, is measured on a scale of from 0 to 100, and is computed by Equation 1:

$$RR = \left(\frac{12-IRI}{12}\right)^a 100 \tag{1}$$

DR is calculated by a deduct point approach. A detailed procedure for DR calculation can be seen in a later section. PCR is an aggregate index scaling the overall index of a section, the development of which is described in the next section.

PCR and Its Development

PCR is an aggregate index employed to assess the overall condition of pavements. PCR is a rater's assessment, on a scale of from 0 to 100, of the serviceability of a pavement with respect to quality of ride and pavement distresses. As used by MDOT, PCR is an objective statistic determined by combining ride quality and distress manifestations.

The Delphi technique was used to gather expert opinions and form the rating scale. The pavement condition was indicated on a PCR scale of from 0 to 100, with 100 representing the best pavement condition and 0 representing the worst pavement condition. Three service levels on a pavement deterioration curve, which are important from practical considerations, were requested from a panel of engineers. The three levels are (*a*) the trigger-level PCR requiring routine maintenance, (*b*) the trigger-level PCR warranting an overlay, and (*c*) the PCR level when major rehabilitation/reconstruction would be required. A panel of 11 department engineers was provided with a hypothetical performance curve on which each member was to write the number that best describes the three levels. Arrived at by consensus opinion are PCR numbers 76, 57, and

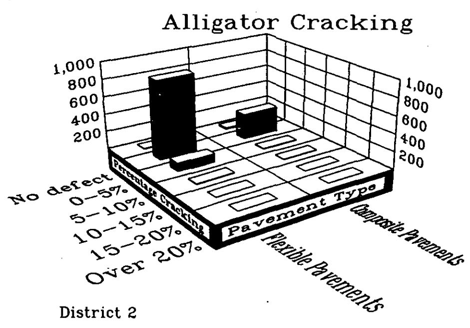


FIGURE 3 Percentage distribution of typical cracking.

38 corresponding to the three trigger levels. With the guidelines of PCR scale developed as already described, a panel was formed to rate the PCRs of 26 selected pavement sections by visual observation and by riding over these sections. Four types of pavements are recognized in the study. After the elimination of outliers by Chevnaut's criterion, the mean of the panel rating for each section was considered to be its PCR (subjective).

The sample section units previously referred to were surveyed manually for distress in accordance with the Mississippi Distress Manual. Deduct points, much like weighting factors, are introduced to signify the magnitude of the effect that each particular distress type, severity, and extent combination has on pavement condition. The underlying principle in the development of deduct point curves is much like that adopted in the PAVER PMS (*2*). A complete set of curves has been provided previously (*3*).

For the calculation of PCR the equation form selected is:

$$PCR = 100\left[\frac{(12 - IRI)}{12}\right]^{a} \cdot \left[\frac{(DP_{max} - DP)}{DP_{max}}\right]^{b} \quad (2)$$

where

IRI = international roughness index (m/Km),
DP_{max} = probable maximum deduct points for the type of pavement: 205, 230, 185, and 145 for flexible, composite, jointed, and continuously reinforced concrete pavements, respectively; and
DP = total deduct points calculated for the distresses present.

For each pavement type the panel-rated PCR is substituted on the left side of Equation 2 and regression analysis is conducted to determine the coefficients *a* and *b* in Equation 2. Only *a* and *b* of flexible pavement group are given here for brevity: $a = 0.96$ and $b = 1.49$. The PCR calculated (employing Equation 2) is a composite index that will be used to assess the overall condition of the network, present and future, once PCR prediction models are developed. A graphical representation of the miles of pavements in different PCR categories is presented in Figure 4. Other graphical representations available are RR, bar chart; DR, bar chart; and rut depth, bar chart.

CONDITION INTERFACE PROGRAM

The interface program developed converts the condition data obtained from video interpretation into a suitable format for ready use in maintenance selection and estimation of contract quantities. The interface program analyzes the raw data and gives the following three reports:

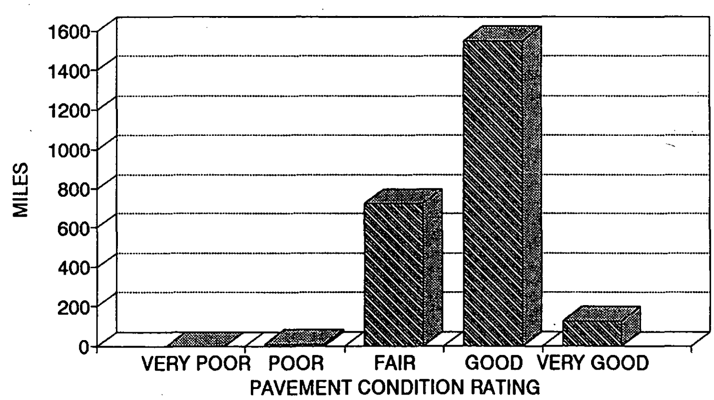
1. Video distress data report by sample unit, which gives the distress in each 500-ft sample unit of a section. This includes the distress type, severity, and quantity on each sample unit.
2. A section video distress summary report yields a distress summary report of the entire section. As shown in Table 1 it includes distress type, severity, and distress density of the whole section.
3. On the basis of the distress quantities an aggregate distress index, designated visual pavement condition evaluation rating (VPCER), is assigned to each section. VPCER is a composite rating derived from the extent and severity of three major distress groups: surface cracking, deformation, and surface defects. On the basis of the extent and severity of these distress groups the values of visual ratings are assigned none, low, medium, and high (or N, L, M, and H, respectively).

A summary of all of the distresses collected in the field and the condition measures calculated for each section are displayed in another table. The various distress elements, which are tabulated and subsequently used in the analysis program, include rut depth, IRI, PCR, and VPCER.

MPB PROGRAM

The MPB program analyzes the roads/highways in each district by each PMS section across all carriageway lanes and shoulders and selects rational maintenance treatments, accumulates maintenance costs, performs economic analysis, and produces a rank listing of all roads.

The required input data for the MPB analysis are collected from the data base. This information includes type of pavement, VPCER,



PCR: VERY POOR 0-29 ; POOR 30-54 ; FAIR 55-74; GOOD 75-89
VERY GOOD 90-100 .

FIGURE 4 District 2 pavement network categorized according to condition.

TABLE 1 Section Video Distress Summary Report

```
Beginning Log Mile: 0.000
SECIDNUM:        55 08N000.000              Survey Date: 08/15/91
County:          8, Road: I55N              Pavement Type: ASP
Road:            I55N                       Section Length: 11516m Surveyed width: 3.66m
Surveyed Lane:   1
```

DISTRESS	CODE	SEVERITY	DISTRESS QUANTITY	UNIT	DISTRESS DENSITY
Alligator	CA	0	528.9 Total: 528.9	sq.m	1.26% 1.26%
Transverse Transverse	CT CT	0 1	Total:142.16 26.33	sq.m sq.m	0.34% 0.06% 0.40%
Patch	PA	0	4.91 Total: 4.91	sq.m	0.01%

```
Distress Groups:  Cracking (CA,CB,CT,CL,CR,CE)    Distress Density: 1.66% Distress Index: 0
                  Deformation (PH, SD)            Distress Density: 0.00% Distress Index: 0
                  Surface Defects (PA,RW,BL)      Distress Density: 0.01% Distress Index: 0
```

IRI, rutting, whether lane widening or shoulder construction is required, analysis period, and so on. With the input information, which in essence portrays the condition of the pavement and the criteria for intervention, a suitable maintenance treatment is chosen. Typical methods for maintenance treatment selection currently in use include (*a*) assignment of maintenance treatments on the basis of past experience, (*b*) a life-cycle cost analysis that selects the most economical one, and (*c*) an optimization of benefit or a reduction in pavement distress over a period of time (*4*).

The MDOT PMS uses the first method, based on a unique approach that considers both distress severity and extent. Depending on the conditions a pavement may require major maintenance (for example, resurfacing) or minor maintenance, such as pothole filling or crack filling and so on. Detailed data on each maintenance treatment are needed, including the unit cost, the expected performance immediately after its application, and long-term performance. A catalog table for a major maintenance treatment is included in Table 2. The expected life, years of maintenance-free service, and unit

TABLE 2 Typical Attributes of M,R&R Strategy

M,R&R CATALOGUE TABLE

Code Number: AC01
Description: AC Structural Overlay (0.10m Default Thickness)

Pavement Type	Expected Life (Years)	Min. Year to Major Maintenance	Cost ($)	Cost Unit	Cost Multiplier	Production Rate/Day*	Factor for Condition		
							IRI**	PCR**	VPCER***
ASP	20	15	0.89	c.y.	1.0	100	1.0	1.0	N
COMP	20	15	0.89	c.y.	1.1	100	1.0	1.0	N
JCP	20	15	0.89	c.y.	1.2	100	1.0	1.0	N
CRC	20	15	0.89	c.y.	1.1	100	1.0	1.0	N
JRCP	20	15	0.89	c.y.	1.1	100	1.0	1.0	N

* Units for cost and production rate are same

** Factor indicating the condition of road after M & R; 1 means no distress condition, and zero implies maximum possible distresses.

*** VPCER: Visual pavement condition evaluation rating based on distress extent and severity.

cost from the table serve as inputs to the MPB program. Information on the production rate per day is useful for determining the time required to apply this treatment to a section. The last three columns in the table indicate factors for assessing the posttreatment conditions of pavements in terms of IRI, PCR, and VPCER, respectively.

The distress manifestations, for example, rut depth and IRI, need to be grouped into four categories, which then become useful in maintenance strategy selection. On the basis of these groupings MPB condition and maintenance intervention (CMI) criteria are arrived at. The CMI table, explaining the significance of CMI attributes, can be seen in Table 3.

Maintenance policies have been developed for maintenance strategy selection. A typical policy table for asphalt pavement is shown in Table 4. Not only can they be updated or modified but new policy tables can also be created by the user. Similar policies are established for each type of pavement. Two major maintenance treatments can be assigned for a given condition scenario.

Also, policy guidelines are established for local minor maintenance, which are based on visual distress type and severity level. Having decided the type of maintenance treatment to be provided, the next step is to calculate the cost of the treatment. Because the dimensions of the section are known from the inventory data base and the amount of distress is assembled by the interface program, the maintenance quantity can be easily estimated. This is multiplied by the unit cost to obtain the cost of the treatment.

The long-term maintenance costs for each section are calculated in conjunction with the vehicle operating cost model described in a later section. Pavement deterioration models, as described previ-

TABLE 3 CMI Criteria Based on Pavement Condition

CMI Attribute	0	1	2	3
IRI,m/km	< 2.00	2.00 to 4.00	4.00 to 6.00	> 6.00
Rut,mm	< 6.00	6.00 to 12.00	12.00 to 25.00	> 25.00
VPCER (Distress)	N (None)	L (Low)	M (Moderate)	H (High)

CMI Criteria Explanation

0: Pavement in excellent condition, no major maintenance required
1: Warning level, no major maintenance required
2: Major maintenance intervention level
3: Maximum applicable pavement deterioration, major maintenance required

TABLE 4 M,R&R Policy Table for Asphalt Pavement

ICMI IRI	RCMI Rut	DISTR VPCER	PAVEMENT TYPE: ASP NO:	Codes	Maintenance Treatment Strategy	Change in Surface Level*
0-1	0-1	N	1.	None	No Major Maintenance	N
		L	1.	MM Codes	Local Minor Maintenance	
			2.	AC15	Surface Seal	N
		M - H	1.	AC08+AC10	1" Milling and inlay	N
			2.	AC05	Modified AC Overlay - 1.5"	Y
	2-3	N - L	1.	AC08+AC10	1" Milling and Inlay	N
			2.	AC13+AC10	Heater Scarification & 1" AC Repaving	N
		M - H	1.	AC09+AC11	1.5" AC Milling & Inlay	N
			2.	AC06	1.5" AC Modified AC Overlay with SAMI	Y
2-3	0-1	N - L	1.	AC10	1" AC Resurfacing	Y
		M - H	1.	AC08+AC10	1" Ac Milling and Inlay	N
			2.	AC13+AC10	AC Repaving	N
	2-3	N - L	1.	AC09+AC11	2" Milling and Inlay	N
			2.	AC05	2" Modified AC Overlay	Y
		M - H	1.	AC16	AC Surface Reconstruction	N
			2.	AC04	2" AC Overlay with SAMI	Y

* Whether existing surface level changes or not owing to M & R activity.

ously (*5*), are used for various types of pavements to determine the present serviceability index (PSI) in future years during the analysis period. If the PSI of any section falls below 2.5, then major maintenance is warranted according to a tentatively selected fixed policy. The present worth of major maintenance, if applicable, is calculated; this constitutes the long-term cost.

Agency Cost

Agency cost comprises major, minor, and routine maintenance and lane widening and shoulder costs. The major maintenance policy table stipulates two alternative strategies for each candidate section. The agency cost can be based on Strategy 1 or Strategy 2. Strategy 1, the base strategy, refers to the maintenance treatment that is most economical. Strategy 2, the alternate strategy, is the most appropriate for the distresses present; it is higher in cost, however. In addition, \$500/mi is earmarked for annual routine/emergency costs.

Vehicle Operating Cost

The vehicle operating cost (VOC) model and life cycle analysis (*5*) are developed to quantify the cost-effectiveness of alternate strategies for investment planning and maintenance management of roads and highways. The methodology evaluates user costs on the basis of vehicle operating cost data from FHWA in conjunction with the deterioration models developed by World Bank (*6*) and NCHRP studies (*7*). Detailed documentation of the VOC model has been provided previously (*5*).

The input data for user cost calculations are generally available from the inventory and condition data base files of the PMS data base. The required data can be grouped into three categories:

1. Section-specific data including ADT, traffic growth rate, and 18-kip ESAL;
2. Decision criteria including maintenance intervention criteria for each pavement classification, adjusted present serviceability rating (PSR) (or IRI) after M,R&R treatment, and the analysis period; and
3. Vehicle operating unit costs (for each vehicle type) for fuel, depreciation, oil, repair, and tires.

The calculation of VOC comprises the following steps:

- Pavement deterioration prediction over the analysis period for each pavement type on the basis of current condition. Cumulative ESAL plays a major role in the deterioration prediction.
- Cumulative traffic prediction by vehicle type over the analysis period.
- Calculation of the VOC stream for each year of the analysis period as a function of pavement condition, vehicle type, associated VOC parameters, and geometric characteristics.

The VOC calculations for a typical pavement section include the year and the total ADT in that year, the cumulative ESAL, the predicted IRI/PSI, and components of VOC owing to fuel, depreciation, oil, repair, and tires. The total VOC for a 30-year analysis period for a pavement with an initial IRI of 1.11 m/km is illustrated in Figure 5.

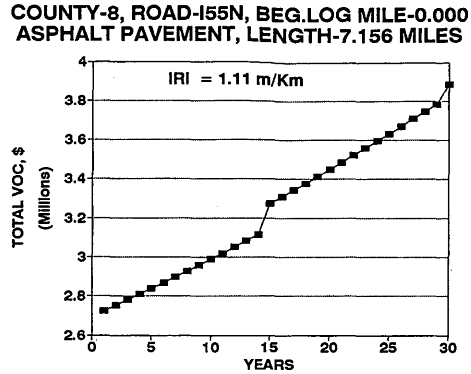


FIGURE 5 Typical VOC for 30-year analysis period.

Total Cost

The sum of major and minor maintenance, lane widening/shoulder, and annual routine/emergency costs yields the total agency cost. Long-term maintenance can be made a part of the total agency cost as well. A typical output from the MPB program with agency cost and user cost applicable to the two suggested treatment strategies is shown in Table 5. There is provision at almost every step to update the inputs on the basis of feedback from the field.

PRIORITY RANKING AND ANNUAL WORK PROGRAM

A formal procedure that ranks the sections in the order in which the maintenance actions are to be undertaken is developed, thereby assembling a work program on a networkwide basis. Two approaches are used for priority ranking: (*a*) on the basis of agency cost excluding long-term cost and (*b*) on the basis of functional classification of the road, traffic, and condition.

The first method ranks the sections on the basis of decreasing agency cost. The agency cost (first cost) can be with respect to either Strategy 1 or Strategy 2, as explained in a previous section.

The second method of priority ranking is based on functional classification, traffic in terms of ADT, condition of the pavement, and committed priority, if applicable. Committed priority is assigned under unusual circumstances such as consideration of safety and an increase in the importance of a road. The functional classification recognizes three categories: Interstate, US, and state routes. The traffic is classified into three groups on the basis of ADT, as follows: low, $<$2,000; medium, 2,000 to 5,000; and high, $>$5,000.

Each of the group of pavements has been assigned an adjustment factor, analogous to a weighting factor, depending on the traffic (three levels) and functional classification (three levels). To recognize pavement condition in the priority ranking the network is partitioned into three categories: Priority Category 1 requires major maintenance, Priority Category 2 warrants local minor maintenance, and others are placed in Priority Category 3. The priority indexes of 100, 58, and 24 are assigned to Categories 1, 2, and 3, respectively. For every section in each of these categories an adjusted priority index is calculated by multiplying the adjustment factor by the priority index. The sections are then ranked in descending order of the adjusted priority index (API). Should there be more than one

TABLE 5 MPB Report for a Section

County Number : 36	Pavement Type : Flexible
Road Name : SR7N	Section Length: 1643 m
Beg. Log Mile : 0.000	Surveyed Lane width: 3.66 m
	Total Section Width: 6.71 m

Lane Widening/Addition: ? : N	New Shoulder Construction Type: 0
Widening Area: 0.00 s.m;	
IRI: 2.93 m/km	New Shoulder Area: 0 s.m
Rutting: 1.52mm	
VCPER: H	

First Year of Analysis: 1993	Analysis Period: 30 year
Planned Year of Maintenance: 1993	Years Before Maintenance (YBM)=0

Section Maintenance Data	Strategy 1	Strategy 2
Major Maintenance Treatment Code	AC08 AC10	AC05
Change in Original Pavmt. Surf. level?	No	Yes
(a) Major Maintenance Cost,	$44804	$52711
(b) Lane Widening/Shoulder cost	$0	$11980
(c) Local Minor Maintenance Cost	$1389	$1389
(d) Annual Routine/Emergency Cost	$0	$0
(e) Long Term Maintenance Cost (PW)	$34405	$34405
Total Agency Cost, (a+b+c+d)	$46193	$66079
Total Agency Cost/s.y.	$3.51	$5.01
Total User Cost for 30 Years (PW)	$2266690	$2266690
Average User Cost (30 years)/1000 vehicle mile(PW)	$212.00	$212.00

section with the same API, the ordering is based on the section with the highest VOC, which receives first priority for maintenance treatment.

The priority ranking list also contains the cost of maintenance for each section and the cumulative cost of maintenance for those sections requiring maintenance. This list becomes useful either for estimating the budget required for the so-called need projects or assembling a list of projects constrained by the available budget.

SUMMARY

A PMS is established for the optimal use of funds for maintaining the Mississippi State road network of more than 12,000 mi. The MDOT PMS has four main modules: a data base, an interface program, a PMS, and analysis and priority ranking. A comprehensive data base, the heart of the PMS, is developed with FOXPRO software. It includes inventory, traffic, overlay, and other data bases. A unique section identification number, identified by the acronym SECIDNUM, is introduced. It will be an index for a given section by which all the data for a section can be accessed.

Truck-mounted high-speed automatic video cameras are used to capture surface distress manifestations. The IRI and rutting/faulting are surveyed by a South Dakota-type profiler and by height measurements obtained by using ultrasonic sensors, respectively. An interface program that will transform the condition data into a suitable format for analysis was developed. With the total quantity of each distress calculated in the program, contract packaging is simplified. The need for a composite measure to indicate the overall condition of a section is addressed by introducing what is known as PCR, which combines distresses and IRI to give a single index. The analysis of the condition data yields the required maintenance strategy and the corresponding agency cost. The agency cost includes major maintenance, minor maintenance, lane widening/shoulder, routine maintenance, and long-term costs. An index based on the condition of the pavement, functional classification, and traffic is used for priority ranking of projects.

Although the MDOT PMS provides needed information for maintenance and rehabilitation of the state's road system, it encompasses some unique features, including

- Rapid network quality condition data collection;
- An interface program that accesses the raw condition data and presents the individual distress quantities in a suitable format for rational maintenance treatment selection;
- A unique maintenance policy table for each pavement type that relies on all of the major distress groups, rutting, and road roughness;
- A VOC model that can be used for life-cycle analysis and long-term maintenance cost prediction; and
- A rational ranking/prioritization system for assembling an annual work program.

Other key items that will be researched in the future are as follows:

- Video interpretation of distress manifestations will need continued improvement;
- Structural evaluations of pavements, both at the network level and at the project level, need to be incorporated into the system, as do the surface texture/skid resistances of pavements;
- Prediction models for both distresses and PCR measures are needed; and
- A rational optimization/prioritization scheme for single-year as well as multiyear work planning is needed.

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Infrastructure Management System: Case Study of the Finnish National Road Administration

VESA MÄNNISTÖ AND RAIMO TAPIO

The Finish National Road Administration has used its network-level pavement management system, the Highway Investment Programming System (HIPS), as a decision support tool since 1989. However, this system is capable of addressing strategic questions concerning paved roads only; it is not capable of addressing questions concerning bridges, for example. A new idea has been to review the existing pavement management system (HIPS) and take the basic features of the Finnish Bridge Management Systems and to modify and couple these two systems into one system, the infrastructure management system (IMS), to optimize simultaneously bridge and pavement maintenance and rehabilitation under the same budget and other constraints. The development of the first version of IMS appeared to be rather successful. The first results show that it is possible to allocate monies between pavements and bridges by using the minimization of social costs as an objective function. Compared with the short-term allocation procedure in HIPS, the influence of traffic volume is stronger in IMS. The system can be modified further. This means that other parts of the infrastructure, like power transmission lines, can be incorporated into the analysis.

The Finnish National Road Administration (FinnRA) has used its network-level pavement management system, the Highway Investment Programming System (HIPS), as a decision support tool since 1989. However, this system is capable of addressing strategic questions concerning paved roads only; it is not capable of addressing questions concerning bridges, for example. A new idea has been to review the existing pavement management system (HIPS) and take the basic features of the Finnish Bridge Management Systems and to modify and couple these two systems into one system, the Infrastructure Management System (IMS), to optimize simultaneously bridge and pavement maintenance policies under the same budget and other constraints (*1*).

As such the system considers comprehensively all of the main expenditure items associated with the networks under consideration, that is, bridges and the pavements, in this case, of road networks. Furthermore, the system incorporates discounted cash flow techniques, so that investment efficiency indicators can be estimated for each investment alternative associated with a prespecified level of budgetary availability.

The contents of the case study were

- Modification of the six submodels in HIPS to be used for both the pavements and the bridges, three for each structure;
- Estimation and calculation of the current condition data, deterioration models, and agency and user cost data;
- Modification of the existing software for simultaneous runs of bridges and pavements; and
- Revision of optimization procedures and incorporation of rate of return and other investment efficiency indicators for comparison of alternatives.

This case study was completed by the end of July 1993. At a later stage it is expected that the case study would be integrated into the training program of EDINU of the World Bank.

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INFRASTRUCTURE MANAGEMENT SYSTEM

This section describes version 1.0 of the IMS, which was developed for FinnRA by Statistical Computing Ltd., InfraMan Ltd., and Viasys Ltd. IMS is a modified version of the Finnish network-level pavement management system, HIPS, which was developed in cooperation with Cambridge Systematics, Inc., of Cambridge, Massachusetts. (Highway Investment Programming System: User's Manual for Version 1.0 Finnish National Roads Administration, Unpublished, 1989.)

The purpose of the system is to optimize pavement and bridge rehabilitation policies and the allocation of funding among pavement and bridges. The model covers general classes of rehabilitation actions from general patching to total reconstruction. Because it is a strict network-level model, the system analyzes road policies at an aggregate level, considering only subnetworks of roads or bridges.

The system is based on Markov dynamic program, which categorizes pavements into 135 condition states and eight actions and bridges into 81 states and five actions and represents deterioration as the probability of making transitions among all possible pairs of condition states over 1 year. An agency cost model estimates the cost of each possible action, and a user cost model evaluates the costs for road users in terms of travel time, fuel consumption, and vehicle depreciation and for bridges in costs of diverted traffic because of weight restrictions.

Separate models are available for six models: three traffic volume classes for pavements and three for bridges. The Markov model and standard economic efficiency indicators optimize budget allocations within each of these six models, and a benefit-cost procedure optimizes the funding among them.

System Description

The structure of the IMS is shown in Figure 1.

Two classifications of analysis are provided to address the resource allocation policy questions of interest to the Ministry of Finance, the Ministry of Transport, and the Highway Administration. These are the

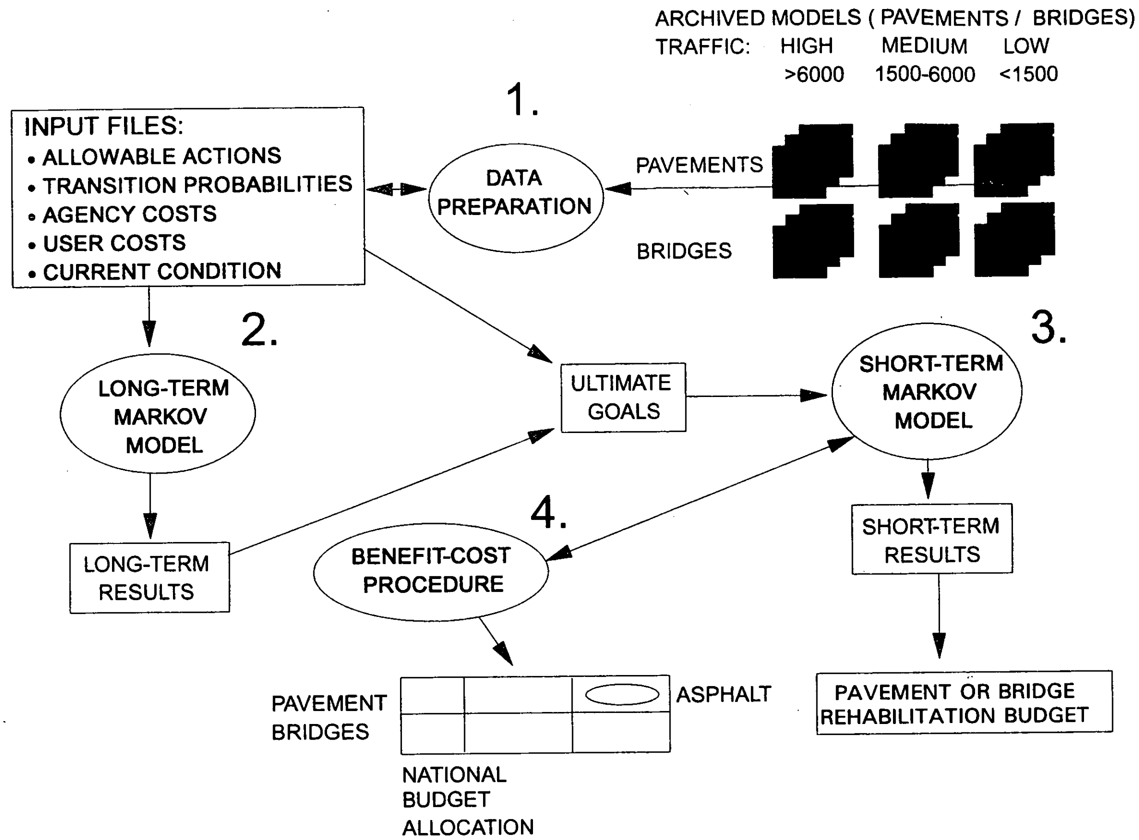


FIGURE 1 Structure of IMS.

- *Structure type level,* pavements or bridges. Each structure type has its own set of condition variables and actions to be modeled.
- *Traffic volume class level,* which affects the rate of deterioration of pavements and bridges as well as the level of user costs associated with pavement condition and diverted traffic costs.
- *Long-term model,* which analyzes possible long-term goals and tries to find a policy that minimizes social costs (the sum of user and agency costs) and that is sustainable indefinitely in the future. The long-term model is not tied to the current condition of the network and imposes no requirements on the specific year in which it should be achieved.
- *Short-term model,* whose first goal is to find the quickest means of achieving the optimal network condition level and whose second goal is to minimize the social costs incurred in the short-term period between the present and the time when the long-term goals are achieved.

As indicated in Figure 1 the flow of activities in using the IMS starts at a very abstract level and ends at a more concrete level. The long-term level defines goals broadly and at some undetermined time in the future; this then proceeds to the short-term level, which is more concrete because it is explicitly tied to the current observed condition of the road or bridge network.

The ellipses numbered 2 to 4 in Figure 1 represent the major analytical features of the IMS in the order in which they are normally used. Central to all of these features is the optimization model in Processes 2 and 3 and the economic analysis and resource allocation within and between models in Process 4. All of these models include the following components:

- *Agency cost model,* giving the average costs for eight (five for bridges) general categories of maintenance and rehabilitation, from do nothing to reconstruction.
- *User cost model,* which quantifies in economic terms the increase in travel time, fuel consumption, and vehicle depreciation associated with deteriorated road condition or additional detours associated with weight restrictions on bridges.
- *Deterioration model,* describing the process by which pavements and bridges deteriorate and thereby cause higher user costs. Similarly it also describes the improvements that can be expected after each of the general rehabilitation actions is applied.
- *Economic model,* describing the economic indicators and the process by which decision makers are able to compare various maintenance and rehabilitation strategies.

The following policy questions are addressed in the framework:

- What is the optimal level of expenditure on rehabilitation on the nationwide road and bridge network and within selected subnetworks?
- At funding levels that do not minimize societal costs, what is the optimal allocation of funding among subnetworks, and what is the most cost-effective means of spending the available money: what actions should be applied to what kind of roads or bridges?
- To what extent do budget constraints increase the level of costs borne by road users, and what does this tell decision makers about the importance to society of user costs relative to agency costs?

Many different modeling methodologies have been applied to these questions around the world. None of these methodologies has been proven to be superior to the others. The methodology selected for HIPS (and IMS) was an adaptation of Markov dynamic programming. The attributes that make it attractive are (*2*) that

- It describes the behaviors of pavements and bridges in a simple manner and fits the decision-making process well at the strategic level; thus, it is suitable for the anticipated training.
- It explicitly recognizes the stochastic nature of pavement and bridge behaviors, and therefore expresses its conclusions in a suitable form.
- The same approach is most obviously useful to other countries and other parts of the infrastructure.

Definition of Pavement and Bridge Condition

Altogether in the pavement (bridge) models there are 135 (108) condition states and 8 (5) action types, for a total of 1,080 (405) states describing each stage. Each state has associated with it an agency cost, a user cost, and a current condition distribution.

For defining an asphalt pavement's condition state the following four major condition variables are used:

- Bearing capacity (five classes, representing ranges of MN/m^2),
- Defects (cracking and patching, three classes, in m^2),
- Rut depth (three classes, in mm), and
- Roughness (three classes, representing ranges of the International Roughness Index).

For bridges there are 81 condition states:

- Superstructure (three classes; good, fair, and poor),
- Substructure (three classes; good, fair, and poor),
- Bearing capacity (three classes; good, fair, and poor), and
- Deck (three classes; good, fair, and poor).

The bearing capacity of a bridge is considered to be the major factor affecting the road user costs of bridges. Other variables also have an influence on the deterioration of other factors.

The maintenance districts have a larger number of standard rehabilitation procedures, but for the purposes of the Markov model they are condensed into several categories, which are for pavements:

- Do nothing (routine maintenance),
- Rut patching,
- General patching,
- Planing,
- Thin overlays,
- Thick overlays,
- Light reconstruction, and
- Heavy reconstruction.

For bridges the categories are

- Do nothing,
- Minor improvements,
- Strengthening,
- Superstructure rehabilitation, and
- Reconstruction.

Optimization

Optimization in IMS is executed in four steps:

1. Long-term optimization,
2. Resource allocation among subnetworks,
3. Short-term optimization, and
4. Optimization by economic indicators.

The first three steps are executed inside the original software, and their definitions can be found in a previous report (*2*). The fourth step, optimization and money allocation by economic indicators calculated from short-term results, is executed inside separate EXCEL procedures.

To compare policy alternatives some measures that describe the benefits from the policy or the investment are needed. The following standard measures are used (*3*):

- Net present value (NPV),
- Internal rate of return (IRR),
- First-year benefit (FYB),
- Time to break even (TBE), and
- Marginal revenue of the investment (MRI).

The economic analysis package (Step 4) consists of three EXCEL 4.0 worksheets:

- ECON,
- GAIN, and
- MODEL.

The most profitable policy for each model is calculated in ECON. The module GAIN compares policies with a chosen discount rate. The last program, MODEL, is used in the allocation of monies between the models.

The first program, ECON, does the calculations needed in the choice of the most profitable policy (Figure 2). The first row contains the name of the model (Pavement High, . . . , Bridge Low). The second row contains the names of the policies (max 1 + 5, do nothing, and five other policies, usually different budget constraints) to be considered. The first column contains the results from the reference policy in terms of social costs. The next five columns contain the results from the other policies. The reference policy should usually be the do-nothing policy, but other reference policies can be used as well.

The result represented by the first value gives the social costs from each policy except the reference policy. If the curve is decreasing the condition of the item in that row is improving in 8 years, and vice versa. IRR is the largest discount rate for which each policy is profitable. The second value shows the gain of each policy when it is subtracted from the reference policy. Normally, the gain is negative in one or two of the first years when the investments are started but becomes positive later. The total gain from the policies can be compared by using NPV, which is the discounted value of the investment during the 8-year investment period.

The second program, GAIN, compares policies with a chosen discount rate in terms of profit per dollar. The GAIN worksheet is shown in Figure 3.

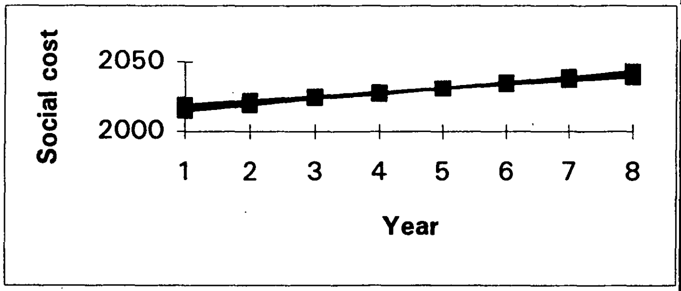
The results in the first table give user cost reduction, agency cost reduction, and social cost reduction during the total 8-year period.

Pavement medium			social cost		
pm0	pm15	pm14	pm13	pm12	pm11
2005	2019	2018	2017	2016	2015
2013	2022	2020	2020	2019	2019
2021	2025	2025	2024	2024	2024
2030	2028	2028	2027	2027	2028
2040	2031	2031	2031	2031	2031
2050	2034	2034	2034	2035	2035
2060	2037	2037	2037	2038	2039
2070	2039	2040	2040	2041	2043

Data aquisition

1. Select database file from Window menu.
2. In the database, select policy/policies .
3. Click right button. Select copy.
4. Select ECON.XLS from the Window menu.
5. Select cell A2-F2. (A2 for the reference policy)
6. Click right button. Select paste.
7. Name data set in cell A1.

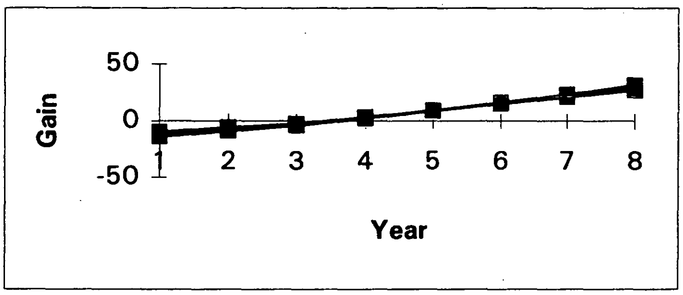
Order of symbols in curves is as follows:
b. square, square, b. diamond, diamond, triangle



The first figure gives social costs from each policy excepting the reference policy.

IRR	24%	26%	29%	31%	31%

IRR gives the largest discount rate for which the investment is profitable.



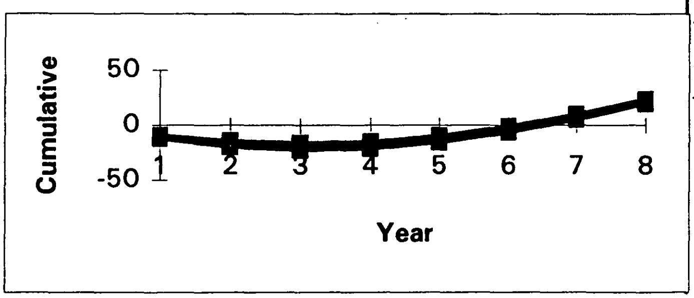
The second figure shows the gain when substracted from the reference policy.

Calculation of the Net Present Value

Write the discount rate in the cell A41. It can be changed when necessary.

10					
NPV	19	21	24	24	22

NPV gives the net value of each policy with given discount rate.



The third figure gives the year, when each policy becomes profitable when compared to the reference policy.

FIGURE 2 ECON module.

In the case in which social cost reduction is negative, agency costs are larger than the gain of the user, and the investment is therefore unprofitable. The first figure gives social cost reduction as a function of agency cost in 8 years for all alternatives used in the analysis.

The second table gives the first-year agency cost (per kilometer) for each policy and the marginal social cost reduction for each policy. The marginal cost is the gain from the last dollar invested in the policy. The second figure gives marginal gain as a function of the first-year agency cost.

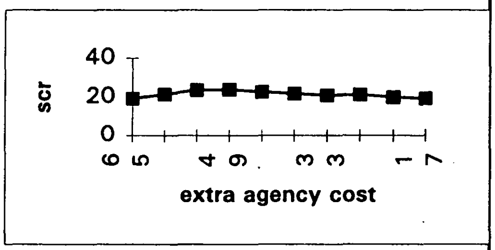
The third program, MODEL, is used in the allocation of monies between the models. The worksheet is shown in Figure 4.

The worksheet consists of the results of the GAIN program for three pavement models and three bridge models. For each model,

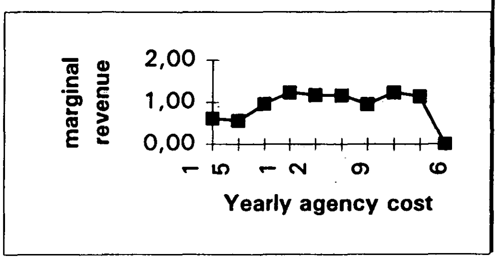
Pavement medium			*social cost*		
pm0	pm15	pm14	pm13	pm12	pm11
2005	2019	2018	2017	2016	2015
2013	2022	2020	2020	2019	2019
2021	2025	2025	2024	2024	2024
2030	2028	2028	2027	2027	2028
2040	2031	2031	2031	2031	2031
2050	2034	2034	2034	2035	2035
2060	2037	2037	2037	2038	2039
2070	2039	2040	2040	2041	2043

Pavement medium			*agency cost*		
pm0	pm15	pm14	pm13	pm12	pm11
1	15	14	13	12	11
2	15	14	13	12	11
2	15	14	13	12	11
3	15	14	13	12	11
3	15	14	13	12	11
4	15	14	13	12	11
4	15	14	13	12	11
5	15	14	13	12	11

10					
agency	65	60	55	49	44
scr	19	21	24	24	22



agency	15	14	13	12	11
marginal	0,61	0,56	0,96	1,23	1,17



Data aquisition

1. Select database file from Window menu.
2. In the database select policy/policies .
3. Click right button. Select copy.
4. Select GAIN.XLS from the Window menu.
5. Select cell A2-F2. (A2 for the reference policy)
6. Click right button. Select paste.
7. Name data set in cell A1.

Order of symbols in curves is as follows:
b. square, square, b. diamond, diamond, triangle

Ten different policies may be compared though only five are shown at a time.

Social and agency costs can also be read simultaneously.

Setting the discount rate

Can be changed when necessary.

Table gives user cost reduction, agency cost and social cost reduction for each policy.

The first figure gives social cost reduction as a function of agency cost in eight years.

Table gives first year agency cost for each policy and marginal social cost reduction (gain from last dollar invested) for each policy.

The second figure gives marginal gain as a function of first year's agency cost.

FIGURE 3 GAIN module.

data consist of total unit agency costs (per kilometer or unit bridge) and marginal cost reduction (user gain from the last dollar invested) with three policies. The central column is the one with which calculations are carried out. On the left and right sides are alternative policies. The cells show the total agency cost (i.e., the total budget of the model). It is the unit agency cost multiplied by the volume (kilometers or unit of bridges). The cell labeled "1st year" at the bottom gives the total budget of all models, its marginal cost reduction, and its average cost reduction, that is, the average cost reduction for each dollar invested.

CASE EXAMPLE

The following example shows the practical results of the IMS for the main road network in Finland.

Introduction

FinnRA is responsible for its main road network, comprising 12 000 km of roads and 4,000 bridges. For this IMS analysis the

Pavement	23	*high*
39	38	37
1,24	1,07	0,80
Pavement	45	*medium*
13	12	11
0,96	1,23	1,17
Pavement	14	*low*
7	6	
0,58	0,00	0,00
Bridges	37	*high*
45	40	39
0,74	1,22	0,61
Bridges	10	*medium*
12	11	10
0,83	1,05	0,94
Bridges	2	*low*
6	5	
0,62	0,00	

1st year	132	*agency*
total	1,25	*marginal*
total	1,68	*average*

Resource allocation between models

The yellow column contains the current allocation of resources between the six models. The yellow cells contain the first year total agency cost for each model together with their sum. Below each yellow cell we have the unit agency cost and below that the marginal user cost reduction in eight years , i.e. the user gain from the last dollar invested for each model and for the total investment. The total average denotes the average user cost reduction for all dollars invested in all of the models.

Changing resource allocation.

1. Choose one of the models by moving the cursor on one of the green or yellow cells of the model. Click left button.
2. Use either button in the bottom of the sheet to decrease or increase resources in the model.
3. Wait until calculations are updated.

FIGURE 4 MODEL module.

infrastructure is divided into three subnetworks according to the average daily traffic (ADT). There are thus six models, three for roads and three for bridges. The size of this infrastructure is as follows:

	>6,000 (high)	*1,500–6,000 (medium)*	*<1,500 (low)*
Roads (km)	2,179	5,852	3,887
Unit bridges (no.)	932	938	392

The main problem that FinnRA has in maintaining these structures is the allocation of money for these models under certain multicriteria goals, such as minimal allowable condition and budgetary constraints. The investment period is arbitrarily taken as 8 years.

The budget for the total network is expected to be about 210.000 units of money [1 unit = 1,000 (FIM) Finnish marks (US$1 = about 4.7 FIM)]. The minimum constraint for each model equals 5 units of money per 1 km or 1 unit bridge. This constraint ensures the lowest feasible traffic conditions. This differs from the do-nothing policy, in which only routine maintenance is carried out.

The current condition of the network to be analyzed is given in Table 1 (in marginal distributions, class 0 is the best class).

Other data used in this analysis are retrieved from HIPS (*4*) and the Finnish Bridge Management System (*5,6*).

As an example, the Pavement Medium model is used.

For the economic analysis of the Pavement Medium model, the results from several IMS runs are collected in the data base beforehand. First policies with five different budget constraints ranging from 11 to 15 are compared. The first sheet (Figure 2) shows the basic results from the ECON program. The first figure shows that there is only a slight difference in the costs of these policies. This is because in each policy only minor reparations are made. The IRRs of the policies are also near each other ranging from 24 to 31 percent. In almost every case they will be profitable. According to IRR, Policies pm12 and pm11 have the highest IRRs (31 percent), showing that they are the least sensitive to the discount rate. The second figure shows the gain from each policy when compared with the do-nothing policy (pm0). Again the results for all policies are similar. With a discount rate 10 percent the net present value is largest (24) for Policies pm12 and pm13, indicating that in such a case these are the most profitable ones. The last figure shows that each policy pays the investment back in about 6 years.

Shown in Figure 3 are the basic results from the GAIN program for each of the policies. The results show that the NPV of 24 is achieved for Policy pm13 with an agency cost of 55 and for Policy pm12 with an agency cost of 49. When these two policies are further considered, the marginal revenue of Policy pm13 equals 0.96, showing that the gain from the last dollar invested after Policy pm12 is only 96 cents and the extra investment is, therefore, unprofitable. Thus, the Policy pm12 is chosen as the optimal one; policies with larger agency costs have smaller payoffs in terms of social costs, and they are in both respects less profitable. When the same analysis is performed for all models, the optimal policies are as listed in Table 2.

The total of these is about 227 million FIM, which is greater than the expected budget. Hence it is not possible to keep all models in the economic optimum, and the budgets of some of them must be decreased. This kind of tuning is carried out by using the MODEL program. As a rule the budget of the model with the smallest marginal cost reduction is reduced. It often happens that several models have marginal cost reductions of the same size. In such a case the decision maker may use his or her expertise and reduce the budget of some other model, too. In this way it is possible to take into account details that are important, even though they are not included in the models (Table 3). As can be seen in Table 3 the investment level depends heavily on the traffic volume. Low-volume roads and bridges get only a minimal fraction of the budget.

TABLE 1 Current Condition of Network To Be Analyzed

		pavement high	pavement medium	pavement low
Roughness	T0	45.0	41.8	31.7
	T1	52.0	54.1	60.2
	T2	3.0	4.1	8.1
Bearing Capacity	K0	71.8	80.6	68.2
	K1	6.8	5.5	13.2
	K2	7.3	5.6	6.7
	K3	4.5	2.9	4.9
	K4	9.6	5.4	7.0
Defects	V0	95.9	86.1	77.5
	V1	3.5	11.3	17.1
	V2	0.6	2.6	5.4
Rutting	U0	94.1	97.9	98.5
	U1	3.4	1.7	1.0
	U2	2.5	0.4	0.5

		bridges high	bridges medium	bridges low
Substructure	A0	23.4	21.9	38.2
	A1	70.0	75.4	57.6
	A2	6.6	10.7	4.2
Super-structure	S0	24.0	20.9	30.6
	S1	75.1	75.8	64.8
	S2	0.9	3.3	4.6
Deck	D0	34.7	29.7	41.2
	D1	65.3	69.5	54.7
	D2	0.0	0.8	4.0
Bearing capacity	B1	98.6	99.1	99.5
	B2	1.4	0.9	0.5
	B3	0.0	0.0	0.0

CONCLUSIONS

Described in this paper is the IMS developed for FinnRA and the World Bank. This system is a network-level management system that optimizes pavement and bridge maintenance policies under the same budgetary constraints.

The development of the first version of IMS appeared to be rather successful. The first results show that it is possible to allocate monies between pavement and bridges by using the minimization of social costs as an objective function. Compared with the short-

TABLE 2 Costs of Various Models

model	kFIM/km	total (1000 FIM)
pavement high	39	84.981
pavement medium	12	70.224
pavement low	6	23.322
	kFIM/ub	
bridges high	36	33.552
bridges medium	14	13.132
bridges low	5	1.960
TOTAL		227.171

TABLE 3 Adjusted Costs of Various Models

model	kFIM/km	total (1000 FIM)
pavement high	38	82.802
pavement medium	10	58.520
pavement low	6	23.322
	kFIM/ub	
bridges high	35	32.620
bridges medium	11	10.318
bridges low	5	1.960
TOTAL		209.542

term allocation procedure in HIPS, the influence of traffic volume is stronger in IMS.

The system can be modified further. This means that other parts of the infrastructure, like power transmission lines, can be incorporated into the analysis.

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Selection of Preferred Pavement Design Alternative Using Multiattribute Utility Analysis

THOMAS J. VAN DAM AND DEBORAH L. THURSTON

Pavement design requires the specification of design criteria, the generation of feasible alternatives, the consideration of trade-offs, and the selection of the best overall choice. Pavement design alternatives typically present the engineer with unavoidable trade-offs between initial cost, maintenance cost, and design life. These trade-off decisions entail a great deal of complexity and controversy, making it difficult to select the alternative that might be the best. A rigorous decision-analytic method that can be used to compare alternatives, multiattribute utility analysis (MAUA), is described, and its advantages over two traditional approaches, life-cycle costing and weighting methods, are described. An example that illustrates a pavement design selection problem in which alternatives present the designer with trade-offs between initial cost, maintenance cost, construction flexibility, and pavement life is presented, thus demonstrating the applicability of the MAUA approach.

When designing either a new or a rehabilitated pavement section, it is recommended that a number of feasible design alternatives be generated, with each meeting the design criteria yet possessing unique characteristics that influence overall project costs, constructability, and performance. The skeleton of the preferred alternative can be extremely complex, because various design attributes must be weighed one against another, generally requiring that trade-offs be made between such fundamental characteristics as initial cost, maintenance costs, and long-term performance. The preferred alternative is that which maximizes the overall perceived benefit at the lowest total cost or, in other words, that provides the greatest value. For example, a project with a low initial cost may have a significantly shorter expected life and higher maintenance costs than an alternative with higher initial costs. Thus a decision that maximizes the perceived value of low maintenance costs and longer life against higher initial cost must be made. Because of the complexity and controversy surrounding such a selection process, a systematic and defensible approach must be adopted (*1*).

Life-cycle costing and weighting methods are the two most common approaches to this problem. Other decision-aiding methods commonly used in other industries, but not often applied in pavement engineering, include multiattribute utility analysis (MAUA), fuzzy set theory, and analytic hierarchy process. Described in this paper are the life-cycle and weighting approaches, with emphasis placed on methodology, advantages, and limitations. MAUA is then introduced and illustrated through the use of an example problem. It is argued that because of its ability to systematically accommodate nonmonetary factors and nonlinear preferences and to address interactions between the design attributes, MAUA offers a powerful alternative to the two traditional approaches.

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METHODS

A number of methods are currently used to select from among many feasible alternatives. Simple nonsystematic approaches, such as always selecting the alternative with the lowest initial cost or the alternative that has always been constructed in the past, is poor engineering practice and can potentially lead to much higher total costs and poorer overall performance (*1*). Therefore a systematic approach such as life-cycle costing, weighting, or another formal decision-aiding procedure is often applied.

Life-Cycle Costing Methods

Life-cycle costing is based strictly in terms of monetary value, and is thus purely an economic analysis of the various cost components involved in the construction, maintenance, rehabilitation, and salvage of a pavement structure over the analysis period. According to Winfrey and Zeller (*2*) the basic purpose in applying an economic analysis is to achieve the desired goals with the maximum satisfaction obtainable at a given cost or to achieve a defined goal at a minimum cost. More specifically, Peterson (*3*) states that "Engineering economics generally provides a type of formal analysis where the time value of money is considered and where the analysis adheres to well-organized procedures. The basis for the comparison is monetary, and all inputs and outputs must be assigned a monetary value. The effects of time are addressed through the application of a discount rate, which is used to normalize all costs and benefits to a given time period.

Life-cycle cost analysis should include all costs anticipated over the life of the facility. Entering common usage in the early 1970s, the method is a key element in any contemporary discussion of economic analysis as applied to pavements (*3–6*). The concept has been accepted by FHWA, FAA, and most state highway agencies. According to a 1984 FHWA policy statement (*7*), "The economic analysis of design alternatives should be made on the basis of life-cycle costs, which encompass all the costs associated with constructing, maintaining, and rehabilitating the pavement over the analysis period being used."

Cost Components

In a recently completed study, 41 North American transportation agencies reported using an economic analysis in their selection of the preferred pavement design alternative (*3*). The cost components included in the analysis varied from agency to agency, with 9 including design costs, 40 including initial construction costs, 26 including maintenance costs, 31 including rehabilitation costs, 12 including salvage value, and 3 including user costs. The results of that study indicate that the application of life-cycle costing techniques is not universally applied and that in the majority of the current approaches all associated costs are not included.

This is in contrast to recommended practice. According to the *AASHTO Guide for Design of Pavement Structures,* (*4*), the major and recurring costs that should be considered in the economic evaluation of alternative pavement strategies include the following:

1. Agency costs
   –Initial construction cost
   –Future construction or rehabilitation cost
   –Maintenance costs
   –Salvage value
   –Engineering and administration costs
   –Traffic control costs
2. Cost to the highway user
   –Travel time
   –Vehicle operation
   –Accidents
   –Discomfort
   –Time delays and extra vehicle operating costs incurred during rehabilitation

This basic breakdown of relevant costs is echoed by other sources (*3,8,9*).

When they are estimating costs most agencies can closely estimate the initial construction cost, engineering and administration costs, and traffic control costs, but they are less certain when they are estimating the timing and costs of future maintenance and rehabilitation. Additionally, the salvage value of the pavement at the end of the analysis period is also difficult to assess. Even greater uncertainty arises when estimating user costs. Epps and Wootan (*9*) state that it is most common only to include initial construction costs, rehabilitation costs, maintenance costs, and salvage value when conducting a life-cycle cost analysis, including user costs only on certain facilities where the impact on the user warrants consideration. Although this simplifies the procedure, it can lead to the selection of alternatives that have higher user costs throughout the analysis period.

By not including all relevant costs and through poor estimation of timing and the costs of the various components that are included, the results of the life-cycle costing procedure can be controversial. If it is perceived that the life-cycle costing procedure unfairly favors one design alternative over another, a transportation agency may find itself in a damaging fight to verify the veracity of its assumptions. Because the difficulties involved in this process are generally recognized, it has forced many agencies to adopt simplistic approaches that contain only cost components that can be easily estimated, neglecting such important components as maintenance, rehabilitation, and user costs.

Discount Rate

Another area of controversy surrounds the discount rate chosen for use in the analysis. According to AASHTO (*4*):

> The discount rate is used to adjust future expected costs or benefits to present day value. It provides the means to compare alternative uses of funds, but it should not be confused with the interest rate, which is associated with the cost of actually borrowing money.

There is a general agreement that the discount rate should be the difference between the market interest rate and inflation using constant dollars (*4*). Epps and Wootan (*9*) suggest the use of a "discount rate of return of 4 percent . . . when constant dollar are used to estimate future rehabilitation and maintenance costs and salvage value," because the "real long term rate of return on capital has been between 3.7 and 4.4 percent since 1966. The value of 4 percent is also endorsed by Peterson (*3*). Oglesby and Hicks (*10*) mention that 4 percent has been estimated by some individuals for government investment, but that higher minimums would be appropriate when the risk is higher. It is further stated that the decision regarding discount rates "has a tremendous influence on the results of economy studies," and therefore the discount rate should be chosen carefully (*9*).

For example, if a low discount rate is selected, design alternatives with lower anticipated future costs will become more appealing. This is because all future costs, such as the costs of maintenance and rehabilitation, are discounted according to the discount rate, with a lower rate resulting in less discounting. If the discount rate was 0 percent, all cost in the analysis would simply be summed, with no adjustment made for the time value of money. The issue is further clouded by the fact that transportation agencies are not in a position to either spend the money now or invest it for future use. Instead, they spend what is appropriated in the present year and hope that in the next year things will not get worse but instead will remain constant or improve slightly. Thus some argue that the discount rate used for transportation projects should more accurately reflect the type of funding and not the discount rates commonly applied to commercial industry.

Advantages

Some advantages of life-cycle costing are that (*a*) it is a systematic, theoretically sound method for examining all of the costs incurred during the life of a pavement structure; and (*b*) through the use of the discount rate the time value of money is accounted for.

Limitations

Some of the limitations of the life-cycle costing procedure are as follows:

- The procedure cannot accommodate nonmonetary factors such as the availability of materials, contractor expertise, agency policies, worker safety during construction, and incorporation of experimental features.
- The accuracy of the estimation of each cost component varies from good (i.e., design and initial construction costs and timing) to poor (i.e., maintenance, rehabilitation, and user costs and timing). Because each cost component has a potentially large impact on the selection of the preferred alternative, that lack of confidence in any estimation is of concern. The use of pavement management will improve the estimations, yet because of inherent variability in pavement structures it is not likely that accurate predictions of pavement

maintenance and rehabilitation needs will be easily forecast in the foreseeable future.

- The procedure treats all costs as if they are considered to be equally important. Thus, it does not address issues of cost distribution, equally weighting user costs with agency costs. Because many agencies are far more concerned with their own costs than those incurred by the user, this may not be desirable. Additionally, initial and rehabilitation costs are generally financed with federal participation, whereas maintenance activities are not. Thus, a transportation agency may desire to more heavily weight maintenance costs, a situation that is not possible under standard life-cycle costing procedures.
- The selection of the discount rate is very important because it can result in the selection of different alternatives if one discount rate is chosen over another. Some advocate that a sensitivity analysis be used to determine the effect of the discount rate, yet if the selection process is found to be highly sensitive, the agency must still make a decision based on a single rate.

Weighting Methods

Weighting methods attempt to address some of the limitations of life-cycle costing through the introduction of nonmonetary factors and a weighting/rating scheme. A weighting method can be used to either supplement the life-cycle costing approach or replace it.

Procedure

The following is a procedure used by one state agency (*1*):

1. Generate alternative designs over a given analysis period.
2. Develop critical design attributes for the selection of the preferred alternative. An example is presented in Figure 1, which includes the design attributes of initial cost, duration of construction, service life, repairability and maintenance effort, rideability and traffic orientation, and proven design life. Note that most of these factors are non-monetary and therefore could not have been included in a life-cycle costing analysis.
3. Each design attribute is assigned a weighting factor on the basis of its perceived relative importance in the design selection process. The weighting factors given in Figure 1 are presented as percentages, with the sum of the weighting factors totaling 100 percent. It is recommended that the decision makers be directly involved in the selection of these weighting factors, because they have a large impact on the resulting selection process.
4. Conduct any analyses required to calculate the attributes.
5. Each alternative is rated independently against the decision attributes by using a selected scale (such as 0 to 100 used in the example in Figure 1). The rating should be conducted for each alternative within a design attribute before moving on to the next attribute. This rating is then placed in the upper left-hand triangle as shown in Figure 1.
6. The assigned ratings are then multiplied by the weighting factors assigned to each attribute, and the values are placed in the lower right-hand triangle in each cell. The total rating for each alternative is then determined by summing the values in the lower triangles across all the attributes. The alternative with the highest total rating is selected as the preferred alternative. In the example in Figure 1, Alternatives 1 and 1A ranked the highest, with ratings of 80.5.

Advantages

The advantages of the weighting method over life-cycle costing include the following:

- Nonmonetary design attributes can be included in the analysis. This allows the use of more generic classification of difficult-to-quantify costs, replacing monetary values with a rating scale. For

	Criteria								
	Initial Cost	**Duration of Construction**	**Service Life**	**Repairability & Maintenance Effort**	**Rideability & Traffic Orientation**	**Proven Design in State Climate**		**Total Score**	**Rank**
Relative Importance	**20%**	**20%**	**25%**	**15%**	**5%**	**15%**		**100.0**	
Alternative 1	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15		80.5	1
Alternative 2	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15		80.5	1
Alternative 3	60 / 12	60 / 12	70 / 18	50 / 7.5	60 / 3	40 / 6		58.0	5
Alternative 4	60 / 12	60 / 12	70 / 18	50 / 7.5	60 / 3	40 / 6		58.0	5
Alternative 5	60 / 12	40 / 8	100 / 25	80 / 12	100 / 5	90 / 14		75.5	2
Alternative 6	60 / 12	80 / 6	40 / 10	20 / 3	40 / 2	20 / 3		44.0	8
Alternative 7	40 / 8	60 / 12	40 / 10	50 / 7.5	50 / 2.5	30 / 4.5		44.5	7
Alternative 8	70 / 14	80 / 16	60 / 13	50 / 7.5	80 / 4	40 / 6		60.0	4
Alternative 9	100 / 20	100 / 20	20 / 5	20 / 3	40 / 2	40 / 6		56.0	6
Alternative 10	30 / 6	60 / 12	100 / 25	100 / 15	100 / 5	30 / 4.5		67.5	3

FIGURE 1 Example of weighting method (*1*).

example, instead of trying to estimate specific maintenance costs, it would be possible to assign each alternative a ranking such as low, medium, or high maintenance costs.

- Monetary design attributes can be weighted to reflect their relative importance to the agency. This can be used to address some of the concerns related to sources of funding (i.e., local versus federal) as well as agency costs versus user costs.
- This procedure helps an agency to define which attributes are really important and which are not. If the procedure is conducted in groups, as recommended, all of the decision makers can work together, allowing a balanced decision criterion to be established. Additionally, this procedure is conducive to a sensitivity analysis, allowing weighting factors as well as individual ratings to be varied to assess their impacts on the selection process. This not only helps in developing a systematic approach but also helps in defending the approach if required.

Limitations

Although a weighting system addresses many of the limitations that exist within the life-cycle costing approach, a number of limitations still exist, including the following:

- The establishment of weighting factors in this procedure is somewhat arbitrary and may not accurately reflect the true preferences of the decision makers. Thurston (*11*) has shown that assignment of weighting factors to reflect relative importance can lead the designer to develop and select inferior alternatives.
- When establishing the rating for each alternative, it is important that a scale that represents the entire feasible range of the attribute is established. A systematic procedure should be used to establish consistency throughout the rating and to eliminate biases. This is difficult, as biases are easily introduced into the rating process because each design alternative is being rated one after another in an open format.
- The weighting method might not accurately establish the preference function because preference is assumed to be linear over the entire range of the attribute. Considerable research has shown that this assumption is not correct, because preference is most commonly nonlinear (*12*). Additionally, without the use of a systematic methodology, preference may actually be discontinuous or inconsistent, resulting in ambiguous results.
- Interactions that may occur between attributes are not readily identifiable, possibly leading to confusing results.

These limitations can only be addressed through application of more sophisticated decision analysis tools.

MAUA

A number of decision analysis tools are currently in use to assist in the decision-making process. These methods include fuzzy set theory, analytic hierarchy process, and MAUA. Only MAUA will be considered in this paper.

General Description

MAUA is a systematic, theoretically based decision-aiding procedure. It rests on a number of axioms that state that preferences exist, that they are transitive, that preference is monotonic over the domain of interest, that probabilities of outcomes exist and can be quantified, that preferences are linear with probability, that ranking of preferences over any pair of attributes is independent of the other attributes, and that the utility function is independent (*13,14*). A detailed description of these axioms and their implications has been provided previously (*11*). It is noted that if the basic axioms can be met a MAUA can be conducted.

A key element of this type of analysis is the concept of utility. Utility is simply a way of establishing value through ranking the order of relative preference between sets of consequences, of benefits and costs (*12*). What makes it extremely useful is that its units have meaning relative to each other, sharing a common metric. Thus it is possible to evaluate choices analytically, even if preference is nonlinear, as illustrated in Figure 2. Additionally, the use of a common metric allows for various utility functions from separate attributes to be combined and analyzed through a MAUA. In general, the utility scale is set between 0 and 1, with 0 having the least acceptable preference offered by any given attribute.

A MAUA requires that a set of design criteria or attributes be established and that single-attribute utility functions be determined, and the various single-attribute utility functions are then combined to calculate the multiattribute utility for any given design alternative. This procedure is easily computerized and extremely flexible, allowing a wide range of monetary and nonmonetary attributes to be considered.

Procedure

The following is the recommended procedure for conducting a MAUA (*13*):

1. Specify a set of design attributes that accurately represents the entire design problem. Once the design attributes are selected, the upper and lower ranges of interest are established, as are the most and least preferred tolerable extremes.
2. Verify preferential independence for the selected attributes. It is recommended that a few pairs of two attributes be ranked, and then determine if changes in the ranking would occur through vari-

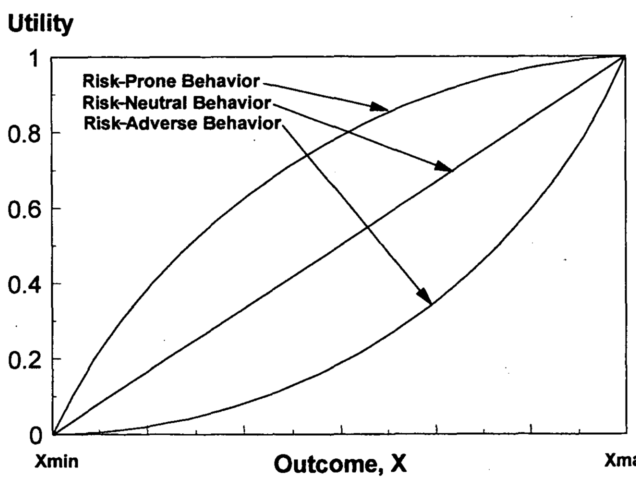

FIGURE 2 Nonlinear preference functions (*12*).

ations in the levels of other attributes. This process should be repeated for each two pairs of attributes. If no change in ranking is reported, preferential independence can be assumed.

3. Single-attribute utility functions are established through a series of questions that force the decision maker to choose between certainty and a lottery. A lottery is simply a condition in which chance is introduced through the use of probabilities. For example, a simple binary lottery is illustrated by the condition in which there is a 50 percent chance of receiving $1,000 or a 50 percent chance of receiving nothing. To assess a decision maker's utility, such a lottery is generally compared with a certainty, such as $500. The decision maker is then asked to choose either the certainty or the lottery or to state indifference. A lottery can also be compared with a lottery to obtain the same result. If preference exists, the conditions of the problem are reformulated by changing either the amount of the certainty or the probabilities of the outcome in the lottery until indifference is obtained. Through careful construction and execution of the problem, the utility function of the decision maker can be assessed over the entire feasible range of each attribute. This procedure, although it sounds complex, is straightforward and quickly accomplished when conducted by someone familiar with MAUA.

At the conclusion of this process a single-attribute utility function is determined for each attribute. These functions model the decision maker's preferences, indicating where riskadverse, riskneutral, and riskprone behaviors exist.

4. After obtaining the single-attribute utility functions, the scaling factors (k_i) used to relate the attributes are determined through the use of a certainty-lottery approach. The method used sets the certainty equivalent to the most preferred value for the attribute in question, whereas all other attributes are set at their least preferred value. A binary lottery is then constructed. The binary lottery has as one outcome all attributes set at their most preferred value and as the other outcome all attributes at their least preferred value. The probability of obtaining the preferred outcome is varied until indifference is reached between the certainty and the lottery. This probability is the scaling factor.

5. The next step is calculation of the normalizing parameter K (*14*):

$$K + 1 = \Pi\,(Kk_i + 1) \tag{1}$$

The normalizing factor ensures consistency between the numerous single-attribute utility functions and the multiattribute utility function so that the multiattribute utility will always lie between 0 and 1.

6. Determination of the multiattribute utility is done by using the following formula (*14*):

$$KU(X) + 1 = \Pi\,[Kk_i U(X_i) + 1] \tag{2}$$

The multiattribute utility $[U(X)]$ is determined from the normalizing factor (K), the individual scaling factors (k_i), and the individual single attribute utilities $[U(X_i)]$. In an analysis of multiple alternatives the actual design values are used to determine the single-attribute utility for each attribute. These are then used to compute the multiattribute utility for each design. The design alternative with the highest utility is then selected as the most preferred.

Advantages

MAUA shares many of the advantages that the weighting system has, with the following additional strengths:

- MAUA allows for all attributes to be treated independently, minimizing bias in the analysis. It is much less susceptible to bias than the weighting method.
- The entire procedure is systematic and theoretically sound, and thus as long as the axioms are met, consistent results will be obtained.
- MAUA encompasses the entire tolerable range of each design attribute and is thus not sensitive to preexisting alternatives. Thus it is easy to evaluate additional designs after the initial analysis has been completed.
- The procedure is easy to computerize. This not only allows for ease of use but also permits a sensitivity analysis to examine how sensitive the recommendation is to changes in the scaling factors and the values of individual attributes.
- Nonlinear preferences are systematically integrated into the design selection process. This is one of the major advantages of MAUA.
- Because of the rigorous procedure interactions between attributes are quantified.
- Uncertainty in expected attribute performance can be accommodated by calculating expected utility.

Limitations

The following are limitations of MAUA:

- The rigorous procedure requires that an individual trained in MAUA be involved in the process. This is most critical initially, but as an agency develops in-house expertise, the need for outside assistance would diminish.
- The time required to set up the problem, assess utility, and complete the analysis makes this technique difficult to use in situations in which an answer is required immediately. This is not believed to be a constraint in the selection of the preferred pavement design alternative.

EXAMPLE OF MAUA IN PAVEMENT DESIGN SELECTION

MAUA can be used in the design selection process both for new construction and for rehabilitation. The example presented below is based on an ongoing project examining how MAUA can be used in conjunction with historical performance data to select the preferred new pavement design alternative.

Problem Statement

This illustration is based on historical distress data collected on low-volume pavements. The pavement sections evaluated in the study are distributed over a wide geographical area, providing an interesting mix of different pavement types subjected to varying climatic conditions.

All pavement sections were designed by a standardized design procedure. Depending on the layer thicknesses and the materials used, three distinct pavement types were identified. For the purpose of this illustration they are simply labeled ALT1, ALT2, and ALT3. As expected, each alternative has unique characteristics that result in different initial costs, maintenance requirements, and anticipated useful life.

The orientation of the region under study results in significant climatic differences, with the northern region receiving more severe winter weather and the southern region having hotter summer conditions. Pavement performance is dramatically affected by climate, and to reflect these differences the study area was divided into three separate climatic regions.

Pavement condition data have been collected since 1980. Inspections were conducted on a 2-year cycle, meaning that each airport was inspected once every 2 years. The data collected were in accordance with the pavement condition index (PCI) procedure (*15*).

The PCI is a numerical rating between 0 and 100, with a rating of 100 corresponding to a pavement in perfect condition. A rating of 0 would be given to a pavement that is impassable. Typically, the range in PCI values lies between 50 and 100, with 50 being considerably below acceptable condition levels. In the present study the administrative agency considers a pavement having a PCI below 70 to be near the end of its useful life and in need of rehabilitation.

Because these procedures are well documented and repeatable, the rate of pavement deterioration can be monitored year after year. These data can then be used to program rehabilitation and monitor the effectiveness of various construction and maintenance techniques related to improving pavement life.

The PCI data were used to estimate pavement life for the three pavement types in each of the climatic regions previously identified. Only pavement sections that have not been previously rehabilitated were considered. The results of this analysis were used to develop regression models of PCI versus age for the various pavement type-climatic region combinations. Both linear and nonlinear regression techniques were applied. For the purpose of demonstrating MAUA in this illustration, the linear models were considered to be adequate. The predicted pavement lives for each pavement type in each climatic region are provided in Table 1.

MAUA

After identifying the problem a MAUA was conducted to determine which pavement type provided the greatest utility for each climatic region. This analysis consisted of identifying attributes, determining single-attribute utility functions, conducting the MAUA, and running a sensitivity analysis.

Design Attributes

A great number of design attributes were initially considered for this illustration, including soil strength, aircraft weight, number of aircraft repetitions, and pavement functional use, to name but a few. For matters of simplification the list of relevant attributes was reduced to four: pavement life, initial cost, annual maintenance costs, and construction flexibility. It is noted that in a real application additional attributes would likely be required. This would not add significantly to the complexity of the MAUA analysis

Pavement Life As discussed previously pavement life is considered to be one of the most important design attributes. The longer the pavement serves traffic at a high condition level the more value is obtained. The administrative agency desires to design and construct pavement sections that continue to perform adequately for the design period.

TABLE 1 Tabulation of Design Attributes

		Attribute Levels Set for Each Alternative			
Climatic Region	Pavement Type	Pavement Life (Yrs)	Initial Cost ($/m2)	Maintenance Costs($/m2/yr)	Construction Flexibility
Northern	ALT1	14	29.30	0.16	2
	ALT2	12	32.85	0.12	5
	ALT3	16	34.00	0.07	0
Central	ALT1	8	27.00	0.21	2
	ALT2	20	29.30	0.08	5
	ALT3	40	30.50	0.02	0
Southern	ALT1	10	24.65	0.19	2
	ALT2	16	27.00	0.09	5
	ALT3	40	29.30	0.03	0

Through examination of the pavement condition data this attribute range was set from 5 to 40 years. A 5-year pavement life was considered to be the minimum expected reasonable age, and a 40-year life was about the maximum life one could hope to obtain. It is noted that in some cases pavement lives have been recorded outside this range, particularly for some of the long-lived sections recorded in the central and southern climatic regions. Although this is true the established range is representative of the minimum and maximum expected pavement lives and thus was used. The estimated life for each pavement alternative in each climatic region is listed in Table 1.

Initial Cost Another important attribute was initial cost. Measured in dollars per square meter of pavement surface, this attribute is the cost of initial pavement construction, not including markings, electrical work, or excessive groundwork. Through review of construction bid tabulations for 1991 and 1992 and discussions with agency personnel, the range of this attribute was set at \$23.50 to \$35.20/m^2. This range covers the wide spectrum of construction types as well as regional and local variations that occur owing to labor rates, competition, and material availability. Listed in Table 1 are the initial costs used in this illustration.

Maintenance Costs The agency considered this to be a very important attribute, because pavement maintenance is the responsibility of the local managing authority, which is typically financially strapped. Through examination of agency records, \$0.00 to \$0.24/m^2/year was estimated as a feasible range of annual maintenance expenditures for these types of pavements. The maintenance costs established for use in this illustration are listed in Table 1.

Construction Flexibility Construction flexibility was established as an attribute in an attempt to account for the advantage of some pavement types that allow staged construction. An arbitrary range was established with a minimum value of 0, indicating little flexibility, to 5 for pavement types offering maximum flexibility. The values chosen for this illustration are listed in Table 2.

Single-Attribute Utility Functions

After establishing the attributes and their ranges a certainty equivalent-lottery procedure was prepared and administered to two

TABLE 2 Results of Single-Attribute Utility Analysis

Attribute	Single Attribute Utility Functions	Scaling Factors
Pavement Life	$U(X_{pl}) = -0.413 + 0.110X - 0.00397X^2 + 0.0000523X^3$	0.7
Initial Cost	$U(X_{ic}) = 1.831 - 0.00159X - 0.00198X^2$	0.2
Maintenance Costs	$U(X_{mc}) = 1.002 - 2.344X - 13.102X^2$	0.5
Construction Flexibility	$U(X_{cf}) = 0.00172 + 0.0248X + 0.112X^2 - 0.00153X^3$	0.05

senior agency personnel. At the onset of the interview utility independence was established between the attributes. The results of the certainty equivalent-lottery procedure were used to establish the single-attribute utility functions summarized in Table 2. Note that in all cases the preference functions were nonlinear.

Scaling Factors

The scaling factors presented in Table 2 were determined by using a certainty equivalent-lottery approach. Because they do not sum to 1 the multiplicative form of the MAUA was used.

MAUA

The normalizing parameter K was calculated to be -0.7684 by Equation 1. The multiplicative form, shown in Equation 2, was then used to conduct the MAUA. The results of the analysis are presented in Table 3. These results suggest that the ALT3 pavement type is providing the best overall utility for all climatic regions. The difference is most evident in the central and southern climatic regions, where its estimated long life and low maintenance costs dominate the utility analysis. In the northern climatic region, the difference is not as dramatic, yet the ALT3 option enjoys a sizable utility advantage over the other two pavement types.

Sensitivity Analysis

A sensitivity analysis was conducted to examine the effect of changing the values of the various attributes and the scaling factors on the outcome of the MAUA.

Pavement Life Pavement life was varied for each pavement type in each climatic region to determine at which point the ranking of the alternatives changed. In the northern climatic region, the ALT1 pavement would need to have an increase in life from 14 to 27 years before its total utility became greater than that of the ALT3 pavement. The ALT2 option would need an increase from 12 to 19 years to overtake the ALT3 pavement type in overall utility. For the ALT3 pavement to have less utility than the next closest alternative, life must decrease from 16 to 10 years. These results indicate that significant changes in anticipated pavement life are required before either the ALT1 or the ALT2 pavement options are more highly ranked than the ALT3 option.

In the central and southern climatic regions the analysis was even more robust. Even if the ALT1 and ALT2 lives were maximized to 40 years no change in ranking would occur. If the ALT3 pavement life were reduced from 40 to 14 years the ALT2 option would become the preferred choice.

The impact of the scaling factor k_{pl} was evaluated by raising and lowering it from 0.5 to 0.9, a range selected as representative of possible extremes. It was found that no change in rank occurred in any of the climatic regions.

TABLE 3 Results of MAUA

Climatic Region	Pavement Type	Single Attribute Utilities: Pavement Life	Initial Cost	Maintenance Costs	Construction Flexibility	Multi-Attribute Utility
Northern	ALT1	0.499	0.554	0.417	0.376	0.580
	ALT2	0.431	0.235	0.637	1.0	0.601
	ALT3	0.552	0.120	0.814	0.0	0.684
Central	ALT1	0.243	0.748	0.116	0.376	0.374
	ALT2	0.626	0.554	0.774	1.0	0.768
	ALT3	1.0	0.452	0.950	0.0	0.946
Southern	ALT1	0.346	0.925	0.292	0.376	0.502
	ALT2	0.552	0.748	0.731	1.0	0.741
	ALT3	1.0	0.554	0.920	0.0	0.946

Initial Cost The initial cost sensitivity analysis indicated that within the range of the attribute, even if the initial cost for either the ALT1 or the ALT2 option were minimized or if the ALT3 initial cost were maximized, no change in the ranking would occur. Only in a situation in which the ALT3 initial cost was maximized and the ALT2 initial cost was minimized would a change in rank occur.

A sensitivity analysis for the scaling factor for initial cost, k_{ic}, was also conducted. This analysis varied k_{ic} from 0.1 to 0.5. This did not result in any change in rank in any of the climatic regions.

Maintenance Costs The sensitivity analysis indicated that maintenance costs could have an impact on the results of the MAUA in extreme instances. For example, in the northern climatic region a reduction in annual maintenance costs from \$0.16 to \$0.08/m^2/year for ALT1 pavements or from \$0.12 to \$0.05/m^2/year for ALT2 pavements would change the ranking. A change would also occur if the ALT3 maintenance costs were raised from \$0.07 to \$0.14/m^2/year. Once again, in the central and southern climatic regions the analysis is more robust, with no change in ranking observed even if maintenance costs were minimized for either the ALT1 or the ALT2 option. Only if ALT3 maintenance costs were increased to \$0.22/m^2/year would the ALT2 option have greater utility in the central climatic region.

The value for the scaling factor k_{mc} was varied from 0.3 to 0.7 in a sensitivity analysis. No change in order was observed in any climatic region as a result.

There is a great amount of uncertainty in estimating maintenance costs. Thus, further research should be instituted to better quantify this attribute. Changing it to a nonmonetary attribute, simply ranking expected maintenance costs on a 5-point scale, would address some of this uncertainty.

Construction Flexibility The MAUA was relatively insensitive to this attribute. Because this attribute already had the minimum value for the ALT3 option and the maximum for the ALT2 option, no changes in rank were incurred in any case.

A sensitivity analysis was also conducted by varying the scaling factor k_{cf} from 0.0 to 0.1. This effected no change in the order in which the pavement types were ranked.

CONCLUSIONS

Reviewed in this paper are two methods currently being used for comparison of pavement design alternatives: the life-cycle costing method and the weighting method. Examined are the procedures, advantages, and disadvantages of each, proposing that a MAUA approach can address the limitations observed while providing additional flexibility and power.

MAUA is a more sophisticated decision-making tool than life-cycle costing or weighting methods. Its advantages are that it can deal with both monetary and nonmonetary attributes, reflect nonlinear preferences over an attribute range, accurately measure a willingness to make trade-offs between attributes, and incorporate the decision maker's attitude toward risk. These benefits come at the cost of an increased level of analytic effort during the alternative comparison stage of pavement design. The motivation for developing MAUA was that simpler methods did not yield satisfactory results. As with any engineering analytic tool, it is up to the designer to determine whether the complexity of the decision problem warrants the extra effort that MAUA entails. The theoretical underpinnings of the MAUA were developed years ago, but it is only with the recent availability of microcomputers that widespread implementation has been feasible.

The approach facilitates organized, logical, depoliticized discussion, either in technical group decision making or in a public forum. It does this by disaggregating the problem into separate components on which diverse interests can reach consensus; performance is first separated from any particular alternative, and minimum performance criteria are established and separated from negotiable performance attributes, which in turn are dealt with separately from trade-off issues.

An illustration was presented to demonstrate the feasibility of the MAUA approach. It was shown that some of the limitations encountered in the selection of the preferred design alternative by using the more common methods can be addressed while adding additional flexibility. The initial experience with this methodology is favorable, and it is hoped that this effort can be expanded to incorporate a wider selection of design attributes.

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National Economic Development and Prosperity Related to Paved Road Infrastructure

CESAR QUEIROZ, RALPH HAAS, AND YINYIN CAI

The relationships between the extent and quality of paved road infrastructure and national per capita income are investigated. A number of selected variables are incorporated in the relationship involving a study of 98 countries. The analyses include cross-sectional and time series methods, with data going back to 1950. Consistent and significant associations between economic development, in terms of per capita gross national product, and paved road infrastructure, on a per capita basis, are demonstrated. The data show that the per capita stock of paved road infrastructure in high-income national economies is dramatically greater than those in middle- and low-income economies. For example, the average density of paved roads (km/1 million people) varies from 170 in low-income economies to 1,660 in middle-income economies to 10,110 in high-income economies. In other words the average density of paved roads in high-income economies is nearly 60 times that in low-income economies. Although it is less definitive, road condition also appears to be associated with economic development. The average density of paved roads in good condition ranges from 40 km/1 million people in low-income economies to 470 in middle-income economies to 8,550 in high-income economies. The information and relationships presented can be used as indicators of weakness or strength in national road infrastructure stock or asset. An initial analysis of Canada's relative ranking is provided. Causality (that is, does an increase in road stock lead to growth, or vice versa?) is also briefly explored.

Contemporary news often hints at the way a country's economy relates to its road and transport infrastructure. During the Gulf War in 1991, the allies, in a quest to neutralize Iraq, concentrated their fire on the destruction of the country's road and bridge network with an intensity second only to that against Iraq's military targets. More recently, countries such as the United States, Canada, and Australia, in their bid to revitalize their economies, have announced stimulus packages with significant components devoted to reconstruction and repair of their road networks.

Restriction of accessibility limits efficient mobility and defers the transfer of human and material resources to places where they can be employed most productively. Conversely, transportation development helps to attain an efficient distribution of population, industry, and income.

In developing countries road transport plays an essential role in marketing agricultural products and providing access to health, education, and agricultural inputs and extension services by providing about 80 to 90 percent of the total inland and border crossing transport of people and goods. A World Bank long-term perspective study (*1*) emphasizes that, although better market incentives (especially related to prices and inputs) to farmers remain important factors in agriculture, the effects of these would be blunted if the physical barriers and economic costs of transporting goods to and from markets remain high.

The impact of road transportation in developed regions is also significant. As an example, in the United States it accounts for 15 percent of the gross national product (GNP) and 84 percent of all spending on transportation (*2*). An efficient road system gives a country a competitive edge in moving goods economically. Conversely, lack of accessibility or poor road conditions are barriers to agriculture, industry, and trade and may hinder the entire development effort. Nevertheless, the contributions of transport to national development may be difficult to quantify in economic terms.

This paper presents information that can be used as indicators of areas of weaknesses or strengths in a country's road infrastructure stock. It is based on correlations or direct comparisons of selected variables on existing road networks and on national output. As pointed out by Owen (*3*), comparisons of income and road infrastructure are not meant to imply that a road by itself is capable of developing a country or region but that it is a necessary element in the development process. An initial analysis of Canada's relative ranking is provided. Causality (that is, does an increase in road stock lead to growth, or vice versa?) is also briefly explored.

CROSS-COUNTRY COMPARISON OF ROADS AND DEVELOPMENT

Recent analyses have been carried out to explore the empirical linkage between road infrastructure and economic development; those analyses have used per capita GNP as the dependent variable and selected indicators of magnitude and condition of road networks as independent variables (*4*). Some key variables are defined in the following paragraphs.

GNP is the measurement of the total market value of the final goods and services produced in a nation's economy during a given time period, normally 1 year. GNP per capita is a country's GNP divided by its population, henceforth denoted PGNP. Spatial road density is a country's road length per land area, and road density is per capita length of the road network.

Road conditions are defined in the World Bank policy paper on road deterioration (*5*) as (a) good, meaning that paved roads are substantially free of defects and require only routine maintenance or unpaved roads need only routine grading and spot repairs; (b) fair, meaning that paved roads have significant defects and require resurfacing or strengthening or that unpaved roads need reshaping or resurfacing (regraveling) and spot repair of drainage; and (c) poor, meaning that paved roads have extensive defects and require im-

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mediate rehabilitation or reconstruction or that unpaved roads need reconstruction and major drainage works.

Using existing data from 98 developing and developed countries, Queiroz and Gautam (*4*) developed the following significant relationship between PGNP and density of paved roads:

$$\text{PGNP} = 1.39 \times \text{LPR}$$

where PGNP is per capita GNP ($/inhabitant) and LPR is the per capita length (or density) of paved roads (km/1 million inhabitants). The coefficient of determination (R^2) is 0.76, the number of degrees of freedom is 97, and the t-statistic of the coefficient is 20.7. The y-intercept term was not found to be significant at the 0.01 level of significance; consequently, it was not included in the equation and the R^2 value was appropriately adjusted. The scatter diagram and the derived relationship are shown in Figure 1.

For comparison purposes an analysis was also carried out by using the spatial density of roads as an independent variable. A slightly less significant regression equation (with an R^2 value of 0.50) was obtained between PGNP and the spatial density of paved roads:

$$\text{LGNP} = 2.25 + 0.49 \times \text{LD}$$

where LGNP is the decimal logarithm of PGNP ($/inhabitant), and LD is the logarithm of density of the paved roads (km/1,000 km^2 of land area).

COMPARISON OF TIME SERIES DATA IN UNITED STATES AND CANADA

A vast amount of historic data is available on the road network and economy of the United States (*6,7*). By carrying out a time series analysis of U.S. data from 1950 to 1988, Queiroz and Gautam (*4*) found a significant positive relationship between PGNP (in

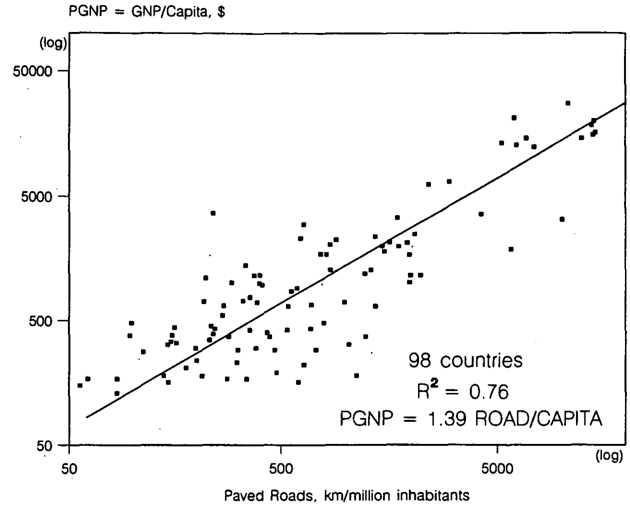


FIGURE 1 Relationship between GNP and paved road density per capita.

$1,000/inhabitant, using 1982 constant dollars) and LPR (in km/1,000 inhabitants):

$$\text{PGNP} = -3.39 + 1.24 \times \text{LPR}$$

with an R^2 value of 0.93; the number of degrees of freedom is 37, and the t-statistic of the coefficient is 21.4 (Figure 2). The intercept (i.e., −3.4) in the previous equation is difficult to interpret. However, a null GNP is well beyond the inference space. Moreover, if we force the equation through the origin, the resulting regression equation is still significant: PGNP = 0.97LPR, with an R^2 of 0.88. However, the 0.97 coefficient in the latter equation would be biased (because the y-intercept is significant). Therefore, the equation with the −3.39 intercept is preferred.

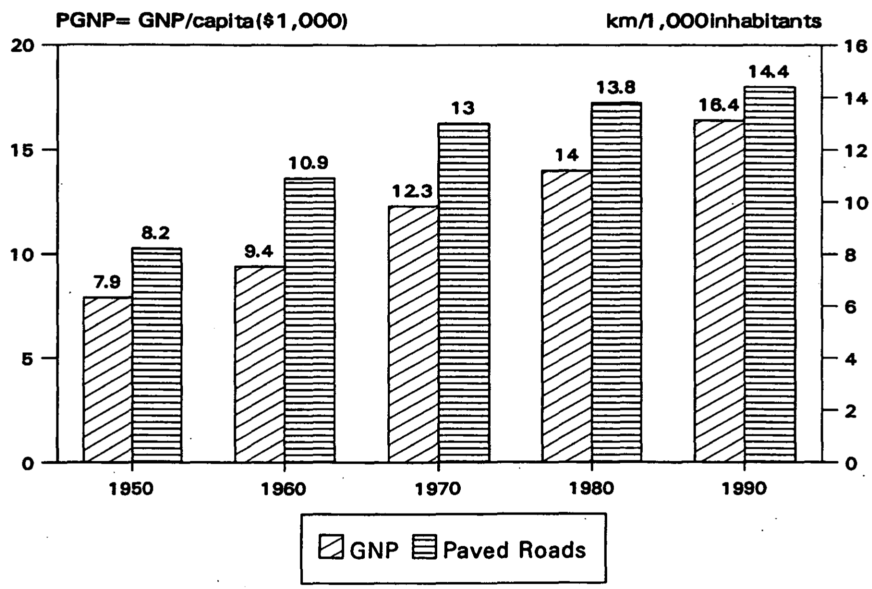


FIGURE 2 Evolution of GNP per capita and paved road density per 1,000 capita in the United States.

An interesting situation exists in running regressions between PGNP and LPR by using different time lags: the highest correlation existed when PGNP for a given year was associated with LPR 4 years earlier (*4*). This appears to indicate that paved roads had an effect on GNP, but there was a time lag of about 4 years between construction and ultimate impact. This 4-year time lag is in broad agreement with the "half a decade" lag period observed by Aschauer (*8*). Aschauer has shown that productivity (i.e., output per unit of private capital and labor) is positively related to government spending on infrastructure, including roads. Analyzing data from the United States for the period 1949 to 1985, he observed that underinvestment in infrastructure started in about 1968, and the effects of deterioration became evident half a decade later, when a productivity slump began in the United States.

That result was obtained for the United States. Now similar data available for Canada are examined. Reasonable data are available in terms of GNP and LPR (*9*). Data for GNP are not so readily available on a consistent basis for the number of years covered in Figure 2. However, the use of several sources plus a number of assumptions made it possible to develop Canadian GNP data for the period 1950 to 1988 (details are available from the authors). The following significant positive relationship between PGNP (in $1,000/inhabitant, using 1988 constant U.S. dollars) and LPR (in km/1,000 inhabitants) was found:

$$PGNP = 0.85 + 1.33 \times LPR$$

with an R^2 value of 0.88; the number of degrees of freedom is 37, and the t-statistic of the coefficient is 16.42.

Like the United States, Canada has the same evolutionary trend between GNP and paved road infrastructure (Figure 3). The 4-year time lag between road paving and economic advance also appears to exist in Canada.

More detailed time series analyses are suggested for future considerations. This could include, inter alia, two-stage least squares tests for causality (e.g., the Granger test) and analyses of data from other countries.

COMPARISON OF CROSS-SECTION AND TIME SERIES ANALYSES

It is interesting to compare the equations resulting from the cross-section analysis of data from 98 countries (circa 1988) and from the time series analysis of the U.S. data (1950 to 1988). The time series equation $PGNP = -3.4 + 1.24LPR$ was derived with constant 1982 dollars. To make it comparable with the cross-sectional equation it should be expressed in 1988 constant dollars taking into account the change in the GNP implicit price deflator between 1982 and 1988, that is, a factor of 1.213 (*7*). The resulting equation is

$$PGNP_{88} = -4.1 + 1.50 \times LPR$$

where $PGNP_{88}$ is real PGNP (1988 $1,000/inhabitant) and LPR is the per capita length (or density) of paved roads (km/1,000 population).

The inference spaces for both equations can be approximately defined by (a) cross-sectional analysis, with an LPR between 60 and 20,000 km/1 million population, and (b) time series analysis, with an LPR between 8,000 and 20,000 km/1 million population. Figure 4 shows the two equations according to their inference spaces. As can be seen in Figure 4, there is relatively good consistency between both equations.

The equation resulting from the time series analysis for Canada (1950 to 1988) is

$$PGNP_{88} = 0.86 + 1.33 \times LPR$$

where $PGNP_{88}$ is real PGNP (1988 $1,000/inhabitant) and LPR is the per capita length of paved roads (km/1,000 population).

The time series line for Canada is merged in Figure 4. It indicates a very good agreement with the relationship for the 98 countries but some offsets with the U.S. relationship. In other words for any given PGNP value the United States has a greater paved road density, currently about 13 percent greater. One can speculate on the reasons, which may include, for example, greater efficiency of economies of

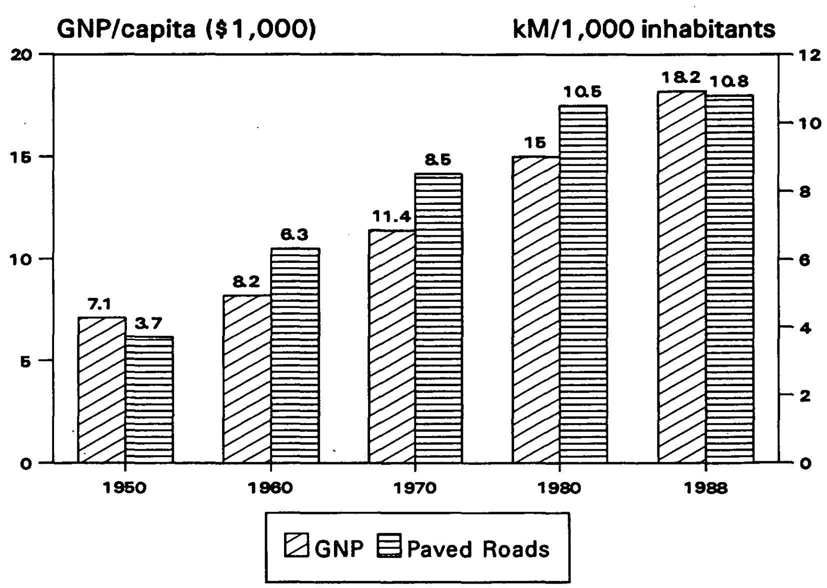


FIGURE 3 Evolution of GNP and paved road infrastructure in Canada.

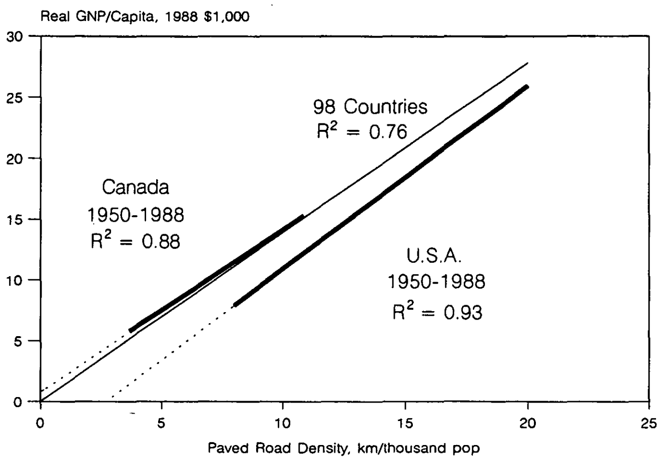


FIGURE 4 Comparison between cross-sectional and time series analyses.

scale. This could be supported by the fact that the United States annually has twice the hot mix paving tonnage of all of Europe combined.

COMPARISON OF ROAD SUPPLY IN WORLD ECONOMIES

A comparison between the supplies and conditions of paved road networks in 98 developing and developed countries is shown in Figure 5. The country groups in Figure 5 are defined as follows (*10*):

1. Low-income economies are those with a PGNP of $545 or less in 1988,
2. Middle-income economies are those with a PGNP of more than $545 but less than $6,000 in 1988, and
3. High-income economies are those with a PGNP of $6,000 or more in 1988.

The 98 countries summarized in Figure 5 comprised (a) 42 low-income economies (average PGNP of $320), (b) 43 middle-income economies (average PGNP of $1,720, and (c) 13 high-income economies (average PGNP of $17,420).

As shown in Figure 5 the supply of road infrastructure in high-income economies is dramatically higher than those in middle- and low-income economies. For instance, the average density of paved roads (km/1 million inhabitants) varies from 170 in low-income economies to 1,660 (plus 876 percent) in middle-income economies and 10,110 in high-income economies, the latter 5,800 percent higher than that in low-income economies. Road condition is also associated with economic development: the average density of paved roads in good condition (km/1 million inhabitants) varies from 40 in low-income economies to 470 in middle-income economies and 8,550 in high-income economies (an increase of 21,000 percent over that in low-income economies).

These results appear to indicate that economic development has a link with paved road density and also to the maintenance standards of those roads. A similar trend probably exists for unpaved roads, because there is high correlation between the extent of a country's paved and unpaved road networks.

The limited resources devoted to the upkeep of road networks in developing countries in the past decade, together with the growth of heavy freight traffic, have created a large backlog of road maintenance and rehabilitation needs. In several countries many kilometers of roads have deteriorated from good to fair and from fair to poor condition. It is not exceptional for sections of main trunk roads to have lost most or all of their blacktop, effectively resulting in a decrease in a country's paved road network. Although many other factors are involved, several countries in which PGNP has de-

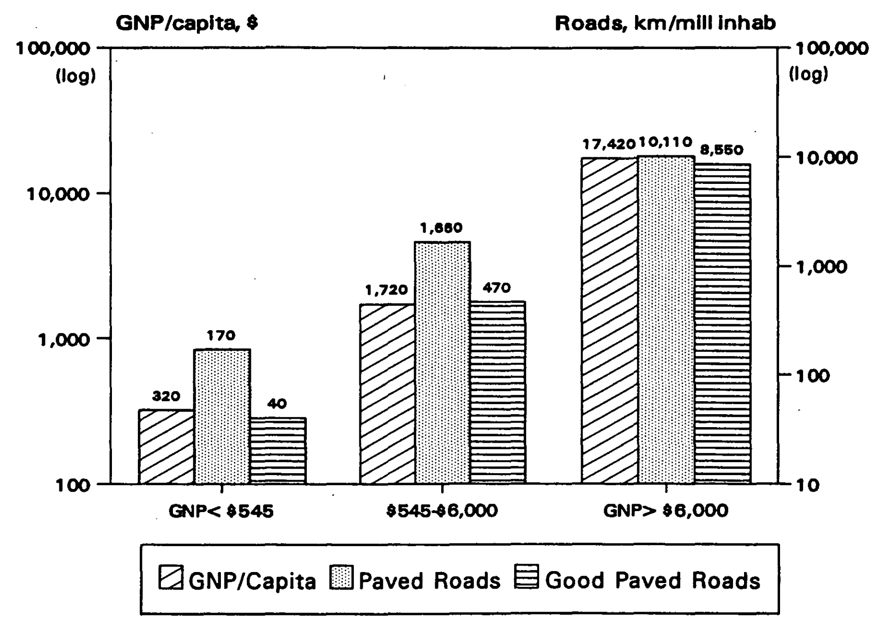


FIGURE 5 Road infrastructure in low-, middle-, and high-income countries.

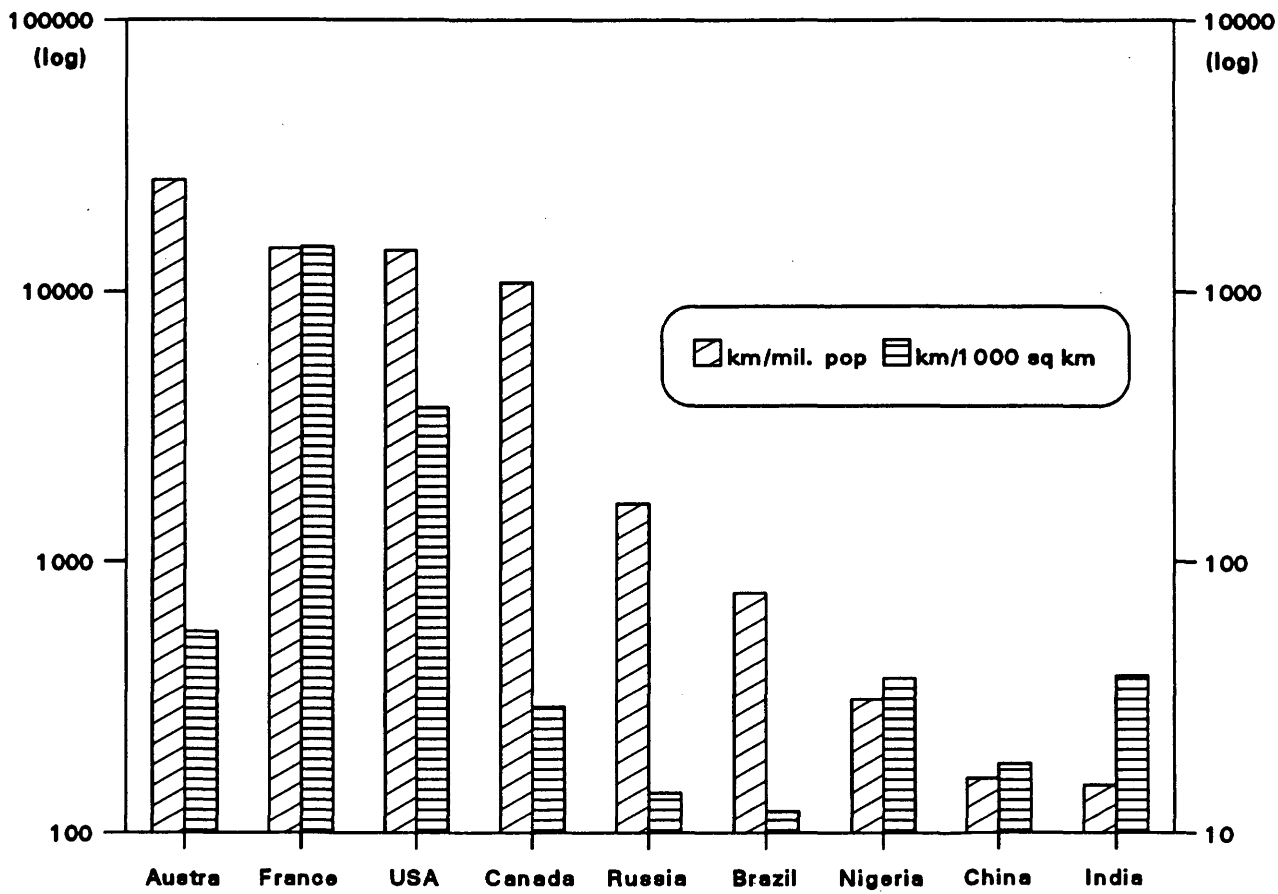
creased in recent years have also faced significant deterioration in their road networks.

For illustrative purposes a comparison of paved road density (km/1 million population) and spatial density (km/1,000 km^2 of land area) in nine large countries is given in Figure 6. As shown in Figure 6 there is a wide range in density, from 25,745 km/1 million population in Australia (labeled Austra in Figure 6) to 1,630 km/1 million population in Russia to 150 km/1 million population in India. Canada is one of the nine countries included in Figure 6 for comparative purposes (*11*). The two types of densities for Canada are similar to those for Australia. This is logical because the two countries are also reasonably similar in terms of size, population, and economy.

DISCUSSION OF CAUSALITY

Assessing the impact of road infrastructure on economic performance is not straightforward because many other factors are involved, and the direction of causation between changes in income and changes in road infrastructure is not clear-cut. One could argue that causation in the equations previously shown could run in either direction. Causality is an issue highlighted for future research. Notwithstanding the controversy over cross-country studies of growth described by Levine and Renelt (*12*), there appears to be a consensus in that comparisons of income and road infrastructure are not meant to imply that a road by itself is capable of developing a country or region but that it is a necessary element in the development process (*3*). The following examples further illustrate the linkage between road infrastructure and development:

1. Chhibber (*13*) and Binswanger (*14*) found that the lack of roads is a significant constraint on the supply response of agriculture.
2. Shah (*15*) used a restricted equilibrium framework to estimate the contribution of public investment in infrastructure to private sector profitability in Mexico. He concluded that a policy emphasis should be to upgrade the public infrastructure (including roads) so that scale economies could be exploited in the future.
3. Aschauer (*16*) has shown that productivity (i.e., output per unit of private capital and labor) is positively related to government spending on infrastructure, including roads. Analyzing U.S. data for the period 1949 to 1985, he observed that underinvestment in infrastructure started in about 1968 and the effects of deterioration became evident half a decade later, when a productivity slump began in the United States.



km/mil. pop	25745	14402	14172	10755	1630	763	310	159	150
km/1 000 sq km	55	1459	372	29	14	12	37	18	38

FIGURE 6 Paved road density in selected countries.

4. Easterly and Rebelo (*17*) found that investment in transport and communication is consistently correlated with growth with a coefficient that implies a high return to public investment.

Therefore, the notion that road infrastructure is a necessary element in the development process is supported by several pieces of research. However, many factors can influence the impact of roads on income. In particular, an exploration of the linkages between policy distortions and the actual outcomes of infrastructure investments carried out by Kauffman (*18*) concluded that a distorted policy environment reduced significantly the ex-post return of the investments. A good example of policies that would probably increase the impact of road investments on productivity was given by Small et al. (*19*). Their policy recommendations include a set of pavement-wear taxes for heavy trucks, a set of congestion taxes for all vehicles, and a program of optimal investments in road durability. Such policies are based on two economic principles: efficient pricing to regulate demand for highway services and efficient investment to minimize the total public and private cost of providing them (*19*).

CONCLUSIONS

The data discussed in this paper show that there is a statistically significant relationship between road infrastructure and economic development on a worldwide basis: cross-section analysis of data from 98 countries (circa 1988) and time series analysis of U.S. and Canadian data between 1950 and 1988 showed significant relationships between PGNP, or per capita gross domestic product in the case of Canada, and density (i.e., LPR) of paved road network. Moreover, there is relatively good consistency between the regression equations from cross-section and time series analyses when they are compared according to their respective inference spaces. Because of the high degree of correlation between the densities of paved and unpaved roads, LPR should be interpreted as a proxy for a country's road stock, paved and unpaved.

The per capita stock of road infrastructure in high-income economies is dramatically greater than those in middle- and low-income economies. For instance, the average density of paved roads (km/1 million inhabitants) varies from 170 in low-income economies to 1,660 (plus 876 percent) in middle-income economies and 10,110 in high-income economies, the latter 5,800 percent higher than that in the low-income economies. Road condition also appears to be associated with economic development: the average density of paved roads in good condition (km/1 million inhabitants) varies from 40 in low-income economies to 470 in middle-income economies and 8,550 in high-income economies.

Causality is an issue highlighted for future research: Does an increase in road stock cause growth or is it the other way around? Assessing the impact of the supply and quality of road infrastructure on economic performance is a complex area of research with potentially important implications on the international infrastructure lending strategy to developing countries.

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Belief-Function Framework for Handling Uncertainties in Pavement Management System Decision Making

B. N. O. ATTOH-OKINE AND DAVID MARTINELLI

Belief functions, otherwise known as the Dempster-Shafer theory of evidence, were applied to pavement management system (PMS) decision making. The theory has been advocated by many as a method of representing incomplete evidence of a system's knowledge base. Dempster-Shafer theory has attracted much attention in the artificial intelligence community in recent years because it suggests a coherent approach to aggregate evidence bearing groups of mutually exclusive hypotheses. Two related issues in PMS decision making are examined: (a) the handling of overall uncertainty in project-level and network-level decisions and (b) the handling of incomplete and imprecise data and information. A prototype evidential decision network for pavement management is constructed to illustrate the applicability of the theory. The resulting formulation demonstrates that many of the shortcomings of alternative methods of handling uncertainty may be overcome.

Recently, there has been considerable interest in addressing uncertainties in pavement management systems (PMS) decision making. Attoh-Okine (*1,2*) proposed the use of Bayesian influence diagrams, a type of directed acyclic graphs (DAGs). DAGs express outcomes in terms of combinations of primitive events. In addition, the graphical structure of these models captures the dependency structure among events, enabling the decision maker to exploit conditional independence to reduce specification and computation. Attoh-Okine (*1,2*), using influence diagrams and value-of-information analysis, addressed the question of perfect and imperfect information in PMS decision making under uncertainty. Bayesian influence diagrams provide users with a clear view of the variables in a PMS framework and the relationship between them. Madanat (*3*) used the latent Markov process, which explicitly recognizes the presence of measurement errors in facility condition assessment. Madanat (*3*) uses a methodology "value of more precise information," which allows the decision maker to evaluate various measurement technologies with different precisions and costs and shows how the methodology fits into a PMS framework. Kulkarni (*4*) discussed the application of Markovian decision processes in PMS decision making. Using the fact that the behavior of pavement is not deterministic but probabilistic, Kulkarni developed probability-based decision making in PMS.

Although there are currently several alternatives for addressing uncertainty in pavement management, they have several shortcomings.

1. They have difficulty handling incomplete or conflicting evidence. It is well understood that many data bases for pavement management are quite incomplete. Because data collection for condition assessment can be performed in several ways, conflicting evidence is quite common in pavement management.
2. They have difficulty incorporating updates or corrections in evidence. As new techniques for measurement and data collection emerge, updates in hypotheses relevant to pavement decision making will increase in frequency.
3. They have difficulty addressing the nested or hierarchical nature of hypotheses for pavement management. Hypotheses for pavement management can typically be broken down into subhypotheses. For example, the validity of pavement performance can be split into the validity of data collection and the validity of the prior condition assessments.
4. They have difficulty generating alternative solutions.
5. They have difficulty taking advantage of all available information.

One characteristic feature of the previous model is that the management of uncertainty in the decision-making process is based on conventional probabilities, an assumption of repetitive situations in PMS data collection and measurements that are possible and can be used readily. Unfortunately, at the present stage of PMS data collection and measurements and with the changing nature of pavement condition and the interaction between various pavement condition variables, it will be appropriate to use the belief-function framework in decision making and in addressing uncertainty. This is because the aforementioned factors (pavement condition, data collection and measurements, etc.) are constantly changing. Furthermore, the uncertainties of subjective judgments are also present when a decision must specify an optimal alternative, like in PMS decision making. Therefore, instead of using a fixed sample frame, one must be able to constantly recognize new relationships between frequency and experience. The aim of this paper is to discuss the belief function, otherwise known as Dempster-Shafer theory, in handling uncertainties in PMS decision making.

Fundamental to the belief function is the representation of uncertain knowledge in the form of basic probability assignment in which the probabilities can be assigned directly to subsets of the states of nature and to individual states of nature. The direct consequence of this kind of assignment is that, although the actual probability of any individual subset of the nature may not be specified, its minimum and maximum values will be specified (*5*).

Given pieces of independent evidence, general inferences may be made about what each piece implies. Dempster-Shafer theory of evidence reasoning allows one to combine evidence in a consistent and probabilistic manner. The theory can be applied to obtain a more complete assessment of what the entire body of evidence

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taken as a whole implies. For example, in obtaining condition assessments for pavement sections several pieces of evidence may be compiled, including equivalent single axle loadings, roughness, rutting, and cracking. Each piece of evidence alone may be used to lend credibility or "belief" to a particular hypothesis; however, it is helpful to know what each piece of evidence implies relative to the set of all possible outcomes.

For example, given roughness measurements, a pavement engineer may be 70 percent confident that a section may be in poor condition within a 5-year period. The engineer is therefore 30 percent confident that the roughness measurements tell nothing. In this case it would be wrong to assume that the remaining 30 percent probability contradicts the notion that the section will be poor in 5 years. This remaining probability should be assigned to the complete solution space. In this way evidence in support of a particular hypothesis does not diminish the strength of future evidence rejecting it. If a Bayesian approach had been followed, then the 30 percent probability would have been assigned to the notion of rejecting the hypothesis. A problem then arises if there is future evidence indicating that the section will not be poor within 5 years. Then no matter how strong this evidence might be it cannot carry a weight greater than 30 percent.

BACKGROUND

Formulation of problems by using the theory involves defining the set Θ to contain all possible outcomes or hypotheses about the problem. Θ is commonly referred to as the *frame of discernment.* An example of Θ in the context of pavement condition 5 years from now might be (excellent, good, fair, poor), where each element represents a particular hypothesis. The basic assignment (BPA) has a range of (0,1) and reflects the quantity of belief in a hypothesis. Given the one piece of evidence stated, we can assign values to the sets H_1 and Θ.

H_1 = (poor),
BPA =.70,
Θ (excellent, good, fair, poor), and
BPA = 1 −.70 =.30.

Note that Θ is not the complement of H_1 but encompasses all possible outcomes, including H_1. Therefore, if later tests give strong evidence (e.g., 60 percent) that the section condition will be "good," then there is sufficient probability to reclaim the belief reflected by this evidence.

Next, the theory uses the quantity known as the *plausibility of a given hypothesis* PL(H). The plausibility is the maximum amount of belief possible, given the amount of evidence negating the hypothesis. Specifically, it is obtained by subtracting the BPA associated with all subsets of the complement of the hypothesis (H).

The next basic element of Dempster-Shafer theory is the belief function, BEL(H). The belief function measures the amount of belief in the hypothesis only on the basis of the observed evidence. Specifically, it is obtained by combining the BPA of H with that of all of its subsets.

The BEL(H) and PL(H) represent lower and upper limits of belief in the hypothesis, respectively, and form the belief interval. These intervals effectively measure the degree in which further evidence might increase the belief in H. Larger intervals reflect a greater uncertainty in the value of BEL(H). In other words, there is a greater opportunity for additional evidence to further substantiate H.

The belief-function approach is linked to conventional probability by considering a multivalued mapping from one space to another (*5*). Figure 1 shows the concept of multivalued mapping. Let Θ represent the parameter space of interest, let θ represent the region of values where the value might lie, and let $\theta \subset \Theta$ represent each individual possible value; T will represent a probability space with the probability density μ_T on it; $\Gamma(t) \subset \Theta$ will represent a multivalued mapping from T to Θ, which means that an observation t in T is equivalent to the observation that the true value of θ is $\Gamma(t) \subset \Theta$. The conventional probability distribution μ_T in T is called *imprecise probability distribution on* Θ.

The belief-function approach (*6*) involves three related representations for belief concerning a topic: the belief function (BEL), the plausibility function (PL), and the basic probability assignment, a generalization of a probability mass distribution. Let the Θ frame of discernment be a set of mutually exclusive and exhaustive hypotheses about some problem domain. A basic probability assignment (bpa) is a function m from 2^Θ, the power set of Θ to (0,1), such that

$$m(\phi) = 0 \tag{1}$$

$$\sum_{A \subseteq \Theta} m(A) = 1 \tag{2}$$

The quantity $m(A)$, called A's basic probability number, corresponds to the measure of belief that is committed exactly to hypothesis A in general and not to the total belief committed to A. Hence, a belief function is defined as BEL induced by a bpa m by

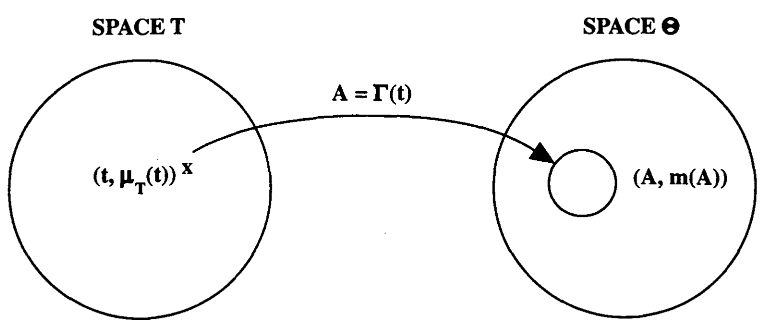

FIGURE 1 Multivalued mapping from T to Θ, which generates a bpa on Θ.

$$\mathrm{BEL}(A) = \sum_{B \subseteq A} m(B)\ (A, B \subseteq \Theta)$$

$$m(A) = \sum_{B \subseteq A} (-1)^{|A-B|}\,\mathrm{BEL}(B) \tag{3}$$

$A - B$ denotes $A \cap \tilde{B}$, and $|A - B|$ denotes the cardinality of this set. m-values either can be assigned directly by the decision maker basis of subjective judgment or they can be derived from compatibility relationships between a frame with known probabilities and the frame of interest. From Equation 3

$$\mathrm{BEL}(A) + \mathrm{BEL}(\bar{A}) \leq 1 \tag{4}$$

a nonadditive formalism. This is different from probability (Pr) theory in which

$$\Pr(A) + \Pr(\bar{A}) = 1 \tag{5}$$

From Equation 4

$$\mathrm{BEL}(A) \leq 1 - \mathrm{BEL}\ (\bar{A}) \tag{6}$$

The quantity $1 - \mathrm{BEL}(\bar{A})$ is called the plausibility of A and is denoted by PL(A). Intuitively, the plausibility of A is the degree to which A is plausible in light of the evidence. A zero plausibility for a hypothesis means that we are sure that it is false, but a zero degree for a preposition means only that we see no reason to believe the preposition.

Notice that each function from {m, BEL, PL] uniquely determines the other two. The equation

$$\mathrm{BEL}(A) + \mathrm{BEL}(\bar{A}) = 1 \tag{7}$$

which is equivalent to

$$\mathrm{BEL}(A) = \mathrm{PL}(A) \tag{8}$$

holds for all subsets of A if and only if BEL's focal elements are all singletons. A subset of A of Θ is called a focal element of BEL if $m(A)$ is greater than 0. By setting $m(\Theta)$ equal to 1 and $m(A)$ equal to 0, for every subset of A of Θ, BEL also satisfies BEL(A) equal to 0 for every subset A; this is called *vacuous belief function.* The BEL indicates no positive beliefs at all to where the truth of Θ lies. This belief-function is appropriate when evidence being considered does not itself tell us anything about which element of Θ is the truth (*6*).

In the belief-function theory, the information about the degree of certainty of an element is represented by the belief interval: [BEL(A) PL(A)]. The belief and plausibility functions denote a lower and an upper bound for unknown probability function. The lower bound represents the degree to which the evidence supports the preposition; the upper bound represents the degree to which the evidence fails to refute the preposition to the degree to which it remains plausible.

If two bpa's on Θ are obtained as a result of two pieces of independent information, they can be combined by using Dempster's rule of combination to yield new bpa's m. The combination can be performed as follows:

$$m(C) = m_1(A) \oplus m_2(B) = \mathrm{K}^{-1} \sum_{A \cap B = C} m_1(A)\, m_2(B) \tag{9}$$

where

$$K = 1 - \sum_{A \cap B = \Phi} m_1(A)\, m_2(B)$$

The second term in K represents the conflict between two items of evidence. If the conflict term is unity, that is, if the two terms contradict each other, K is equal to 0; in such a situation, the two items of evidence are not combinable.

APPLICATION TO PMS DECISION MAKING

The primary advantage of using belief functions in PMS decision making is that each data collection procedure, reliability of various pieces of equipment used for the data collection, pavement performance, and cost analysis can be expressed at a level of detail of its own environment. The ability to represent ignorance concerning the reliability of data collection and the equipment used reduces the likelihood of erroneous interpretation of the overall decision making. Another advantage is that in PMS beliefs are assigned not to a single preposition but to sets of prepositions. Finally, in PMS decision making decision makers have no a priori probabilities of all the variables that form the decision framework.

Figure 2 is a prototype evidential network that represents various objectives in PMS decision making. In this example it is assumed that the overall payoff of the final decision in regard to the maintenance and rehabilitation decision depends on the budget level and the pavement performance. The pavement performance objective depends on data collection procedures, measurements, and how well the previous survey data were interpreted. The objectives are represented with rounded rectangles, and the circular nodes represent relations between the objectives that are of interest to the decision maker. In the present example it was assumed that all the values of the objectives are binary and only "AND" tree relationships may exist among the objectives. Furthermore, it is assumed that there is only one item of evidence for each objective. Figure 3 is an example of an "AND" tree and three nodes. Thus, we will have only one m-value at different objectives. It was assumed that the decision has judgment (although subjective) about the level of support. To determine if the level of payoff is adequate or if there is overall support for payoff on the basis of mutually exclusive evidence, one must aggregate all the evidence to the payoff objective node. This is obtained by propagating the m-values. Because all the objectives are binary we will represent each m-function by triplet [$m(p)$, $m(-p)$, $m(p, -p)$]. For example if $m(p)$ is equal to 0.8, $m(-p)$ is equal 0, and $m(p, -p)$ is equal 0.2, we write mp (0.8, 0, 0.2).

In the present example (Figure 2) there are seven nodes and seven items of evidence. The items of evidence in this present example are considered from the methods and procedures pavement engineers and decision makers relied on to make certain assumptions and decisions on the nodes shown. Table 1 shows this procedure and method. It is assumed that the decision maker has made judgments about the level of support obtained from these procedures and methods for the respective nodes. These values are represented as m-values.

To determine the overall support for each node as a result of aggregating all evidence, we propagate m-values at each node and combine the m-values received by each node from its neighbor with the m-value defined at the node. The combination is done by using Dempster's rules of combination.

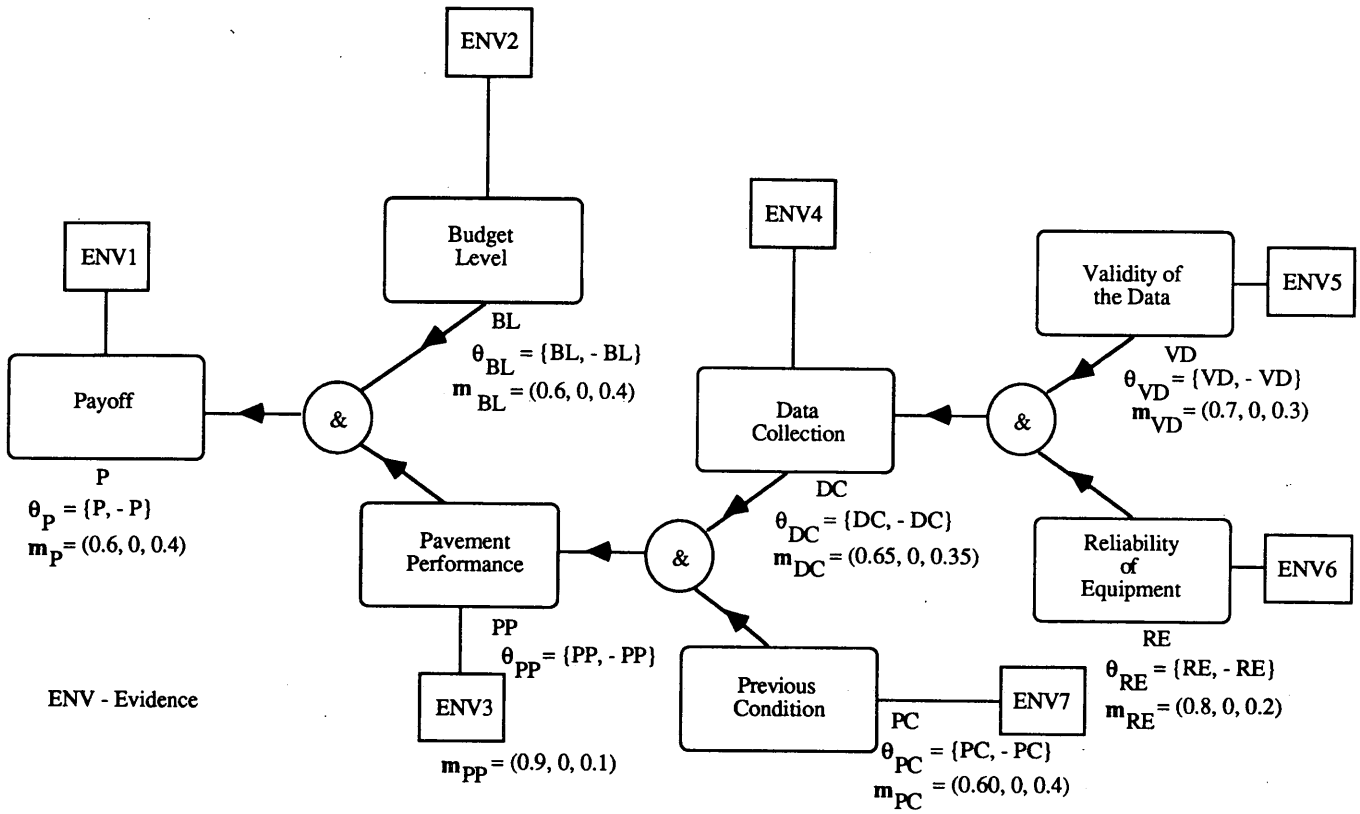

FIGURE 2 Prototype evidential tree for PMS.

In the example we first propagate validity of data (*VD*) and reliability of equipment (*RE*) to data collection (*DC*). This yields $m'_{DC \leftarrow VD+RE}$. The second step is to combine m'_{DC} with m_{DC} to obtain m''_{DC}. The next step is to combine *DC* and *PC* to *PP*, and this yields $m'_{PP \leftarrow DC+PC}$; $m'_{PP \leftarrow DC+PC}$ is then combined with m_{PP} to obtain m''_{PP}. The same steps are used to combine *PP* and *BL* ($m'_{P \leftarrow PP+BL}$), and finally m'_P is combined with m'_P to obtain m''_P. Tables 2 to Table 7 illustrate the propagation and combination of various nodes on the basis of the "intersection tableau" proposed by Gordon and Shortliffe (*7*). In using the intersection tableau, $m_1 \oplus m_2(\phi)$ for any subset is set to be equal to zero. By definition $\Sigma m_1 \oplus m_2\ (X)$ is equal to 1. m''_P is the resulting total *m*-values obtained by all of the mutually

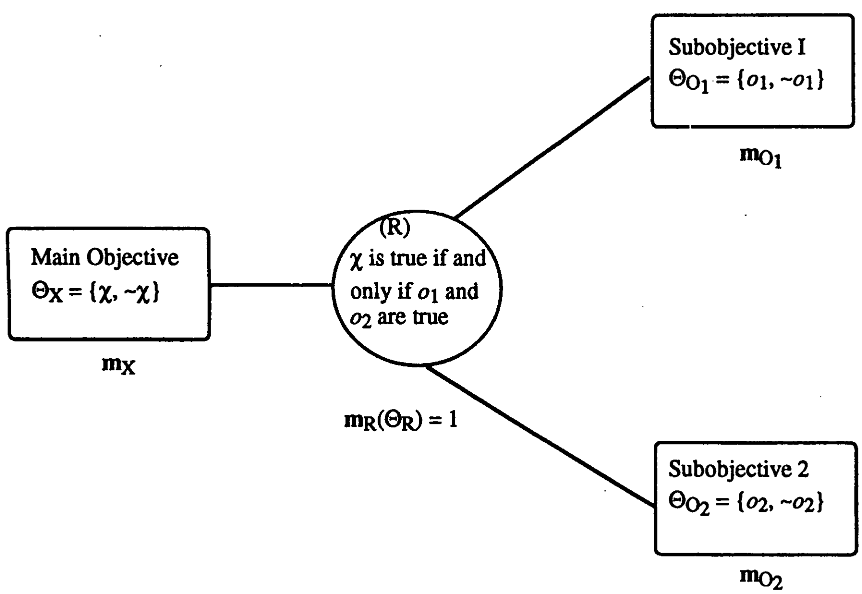

FIGURE 3 "AND" tree with three nodes.

TABLE 1 Potential Sources of Evidence

Evidence Number	Recommended Procedure and Method
1	Prior Years Experience in PMS Decision Making
2	The percentage difference between proposed budget and approved budget
3	Use R-squared obtained from pavement performance equation
4	Frequency of Data Collection
5	Outliers and the distribution pattern should be the major focus
6	They should be based on both operator competence and the reliability of previous collected data.
7	Previous years condition of the pavement based on subjective judgment.

exclusive gathered evidence on the payoff node P. m''_P is (0.072, 0, 0.928).

By definition, the corresponding beliefs are

$$\mathrm{BEL}''_P(P) = 0.072,\ \mathrm{BEL}''_P(-P) = 0, \text{ and } \mathrm{BEL}''_P[(P - P)] = 1$$

and the corresponding plausibilities are

$$\mathrm{PL}''_P(P) = 1 - \mathrm{BEL}''_P(-P) = 1$$

$$\mathrm{PL}''_P(-P) = 1 - \mathrm{BEL}''_P(P) = 0.928$$

The results indicate that there is an overall assurance of 0.072 on the payoff node given the evidence or level of support in the present example that there will be a payoff. $\mathrm{PL}''_P(-P)$ can be expressed as the risk involved in the main objective node (payoff) on the basis of the evidence given.

SUMMARY

This paper illustrates that belief functions can be used in PMS decision making. In designing decision-analytic framework models in PMS, decision makers must formulate relationships between various objectives, incorporate subjective judgment, and pool evidence from various independent sources. The existing analytical tools presently used in PMS decision making do not adequately address such issues in a comprehensive manner.

The belief-function approach provides a more rigorous but straightforward approach to dealing with decision making in PMS with imprecise probabilities and incomplete information from independent sources.

TABLE 2 Combination of m'_{VD} and m_{RE}

m_{VD} / m_{RE}	$(m_{VD} \oplus m_{RE}) \rightarrow m'_{DC}$ {VD} (0.7)	θ(0.3)
{RE} (0.8)	ϕ (0.56)	{RE} (0.24)
θ(0.20)	{VD} (0.14)	θ(0.06)

There is one null entry in the table

∴ K = 0.56

1 - K = 1 - 0.56 = 0.44 , thus

$m_{VD} \oplus m_{RE}$ {RE} = 0.24/0.44 = 0.546

$m_{VD} \oplus m_{RE}$ {VD} = 0.14/0.44 = 0.318

$m_{VD} \oplus m_{RE}$ {θ} = 0.06/0.44 = 0.136

$m_{VD} \oplus m_{RE}$ is zero for all other subsets θ

TABLE 3 Combination of m'_{DC} and m_{DC}

m'_{DC} / m_{DC}	$(m'_{DC} \oplus m_{DC}) \rightarrow m''_{DC}$ {RE} (0.546)	{VD} (0.318)	θ(0.136)
{DC} (0.65)	ϕ(0.355)	ϕ (0.207)	{DC} (0.088)
θ(0.35)	{RE} (0.191)	{VD} (0.111)	θ(0.048)

There are two null hypotheses

K = 0.355 + 0.207 = 0.562 1 - K = 1 - 0.562 = 0.438

$$m_{DC} \oplus m'_{DC}\ \{DC\} = \frac{0.088}{0.438} = 0.201$$

$$m_{DC} \oplus m'_{DC}\ \{RE\} = \frac{0.191}{0.438} = 0.436$$

$$m_{DC} \oplus m'_{DC}\ \{VD\} = \frac{0.111}{0.438} = 0.253$$

$$m_{DC} \oplus m'_{DC}\ \{\theta\} = \frac{0.048}{0.438} = 0.110$$

$m_{DC} \oplus m'_{DC}$ is zero for all other subsets θ

TABLE 4 Combination of m''_{DC} and m_{PC}

m''_{DC} / m_{PC}	$(m''_{DC} \oplus m_{PC}) \rightarrow m'_{PP}$ {DC} (0.201)	{RE} (0.436)	{VD} (0.253)	θ(0.110)
{PC} (0.60)	ϕ(0.121)	ϕ(0.262)	ϕ(0.152)	PC (0.066)
θ(0.40)	{DC} (0.080)	{RE} (0.174)	{VD} (0.101)	θ(0.044)

K = 0.121 + 0.262 + 0.152 = 0.535 1 - K = 0.465

$$m_{PC} \oplus m''_{DC}\ \{PC\} = \frac{0.066}{0.465} = 0.142$$

$$m_{PC} \oplus m''_{DC}\ \{DC\} = \frac{0.080}{0.465} = 0.172$$

$$m_{PC} \oplus m''_{DC}\ \{RE\} = \frac{0.174}{0.465} = 0.374$$

$$m_{PC} \oplus m''_{DC}\ \{VD\} = \frac{0.101}{0.465} = 0.217$$

$$m_{PC} \oplus m''_{DC}\ \{\theta\} = \frac{0.044}{0.465} = 0.095$$

$m_{PC} \oplus m''_{DC}$ is zero for all other subsets θ

TABLE 5 Combination of m'_{PP} and m_{PP}

m'_{PP} / m_{PP}	$(m'_{PP} \oplus m_{PP}) \rightarrow m''_{PP}$ {PC}(0.142)	{DC}(0.172)	{RE}(0.374)	{VD}(0.217)	θ(0.095)
{PP}(0.9)	ϕ(0.128)	ϕ(0.155)	ϕ(0.337)	ϕ(0.195)	{PP}(0.085)
θ(0.10)	{PC}(0.014)	{DC}(0.017)	{RE}(0.037)	{VD}(0.022)	θ(0.010)

K = 0.128 + 0.155 + 0.337 + 0.195 = 0.815 = 1 - K = 0.185

$$m_{PP} \oplus m'_{PP}\ \{PP\} = \frac{0.085}{0.185} = 0.459$$

$$m_{PP} \oplus m'_{PP}\ \{PC\} = \frac{0.014}{0.185} = 0.076$$

$$m_{PP} \oplus m'_{PP}\ \{DC\} = \frac{0.017}{0.185} = 0.092$$

$$m_{PP} \oplus m'_{PP}\ \{RE\} = \frac{0.037}{0.185} = 0.200$$

$$m_{PP} \oplus m'_{PP}\ \{VD\} = \frac{0.022}{0.185} = 0.119$$

$$m_{PP} \oplus m'_{PP}\ \{\theta\} = \frac{0.010}{0.185} = 0.054$$

$m_{PP} \oplus m'_{PP}$ is zero for all subsets θ

TABLE 6 Combination of m''_{PP} and m_{BL}

m''_{PP} / m_{BL}	$(m''_{PP} \oplus m_{BL}) \rightarrow m'_P$					
	{PP}(0.459)	{PC}(0.076)	{DC}(0.092)	{RE}(0.200)	{VD}(0.119)	θ(0.054)
{BL}(0.6)	φ(0.275)	φ(0.046)	φ(0.055)	φ(0.120)	φ(0.071)	{BL}(0.032)
θ(0.4)	{PP}(0.184)	{PC}(0.030)	{DC}(0.037)	{RE}(0.080)	{VD}(0.048)	θ(0.022)

K = 0.275 + 0.046 + 0.055 + 0.120 + 0.071 = 0.567

1 - K = 0.433

$m''_{PP} \oplus m_{BL}$ {BL} = $\frac{0.032}{0.433}$ = 0.074

$m''_{PP} \oplus m_{BL}$ {PP} = $\frac{0.184}{0.433}$ = 0.425

$m''_{PP} \oplus m_{BL}$ {PC} = $\frac{0.030}{0.433}$ = 0.069

$m''_{PP} \oplus m_{BL}$ {DC} = $\frac{0.037}{0.433}$ = 0.085

$m''_{PP} \oplus m_{BL}$ {RE} = $\frac{0.080}{0.433}$ = 0.185

$m''_{PP} \oplus m_{BL}$ {VD} = $\frac{0.048}{0.433}$ = 0.111

$m''_{PP} \oplus m_{BL}$ {θ} = $\frac{0.022}{0.433}$ = 0.051

TABLE 7 Combination of m'_P and m_P

m'_P / m_P	$(m'_P + m_P) \rightarrow m''_{PP}$					
	{BL}(0.074)	{PP}(0.425)	{PC}(0.069)	{DC}0.085)	{RE}(0.185)	{VD}(0.111)	θ(0.051)
{P}(0.6)	φ(0.044)	φ(0.255)	φ(0.041)	φ(0.051)	φ(0.111)	φ(0.067)	{P}(0.031)
θ(0.4)	{BL}(0.030)	{PP}(0.170)	{PC}(0.028)	{DC}(0.034)	{RE}(0.074)	{VD}(0.044)	θ(0.020)

K = 0.044 + 0.255 + 0.041 + 0.051 + 0.111 + 0.067 = 0.569

1 = K = 0.431

$m_P \oplus m'_P$ {P} = $\frac{0.031}{0.431}$ = 0.072;

$m_P \oplus m'_P$ {BL} = $\frac{0.030}{0.431}$ = 0.070;

$m_P \oplus m'_P$ {PP} = $\frac{0.170}{0.431}$ = 0.394;

$m_P \oplus m'_P$ {PC} = $\frac{0.028}{0.431}$ = 0.065;

$m_P \oplus m'_P$ {DC} = $\frac{0.034}{0.431}$ = 0.079;

$m_P \oplus m'_P$ {RE} = $\frac{0.074}{0.431}$ = 0.172;

$m_P \oplus m'_P$ {VD} = $\frac{0.044}{0.431}$ = 0.102;

$m_P \oplus m'_P$ {θ} = $\frac{0.020}{0.431}$ = 0.046

$m_P \oplus m'_P$ is zero for other subsets of θ

Finally, the framework can be used to quantify the level of uncertainty associated with the payoff node. This is equivalent to the width of the belief interval [BEL$''_P(P)$ PL$''_P(P)$], which is the amount of uncertainty in the main objective payoff node with respect to the items of evidence given. The uncertainty interval associated with the present case study is [0.072, 0.928].

ACKNOWLEDGMENTS

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Distress as Function of Age in Continuously Reinforced Concrete Pavements: Models Developed for Texas Pavement Management Information System

TERRY DOSSEY AND W. RONALD HUDSON

In 1989 FHWA required all states to implement a formal pavement management system by February 1993. To comply with this mandate the Texas Department of Transportation (TxDOT) is developing the Pavement Management Information System (PMIS). PMIS will assist Texas planners in providing cost-effective maintenance of the state pavement inventory. To correctly priority rank pavement rehabilitation and predict future needs, PMIS must accurately predict the development of pavement distress with time under Texas conditions. By using a data base containing 20 years of historical condition survey data taken on continuously reinforced concrete pavements across the state, models were developed for the following distress types: punchouts (minor and severe), patches (asphalt and portland cement concrete), crack spacing, loss of ride quality, and spalling. The significance of additional extrinsic factors relating to traffic, pavement structure, and environment was also investigated. These factors may be incorporated into the distress models at a later date, when the data base currently supporting the PMIS is expanded.

In 1989 FHWA mandated that all states have a formal pavement management system in place by February 1993. To comply with this mandate the Texas Department of Transportation (TxDOT) will implement the Pavement Management Information System (PMIS). PMIS will allow TxDOT administrators to monitor statewide trends in pavement condition. It will also assist in the monitoring, selecting, and priority ranking of paving projects and in estimating future needs (*1*).

A rehabilitation strategy for a given pavement section may consist of doing nothing, applying preventive maintenance, or performing light, medium, or heavy rehabilitation. A decision to do nothing in the current year may result in the need for more costly rehabilitation later. For PMIS to perform a multiple-year optimization it must be able to predict the development of pavement distress as a function of age and other possibly significant factors such as traffic, structural design, and the environment.

A literature survey was made to determine the important distress manifestations for continuously reinforced concrete pavements (CRCPs) In Texas. At the same time existing pavement performance data bases were examined to determine what type of distress data had been collected. The principal data bases considered were the Pavement Evaluation System data base maintained by TxDOT, the Rigid Pavement Data Base developed by the Center for Transportation Research (CTR) at the University of Texas, and the COPES data base (FHWA) (*2*).

The CTR Rigid Pavement Data Base was selected for this phase of the analysis because it was the only source of CRCP performance data available that directly addressed Texas conditions and provided sufficient historical depth for the analysis. The CTR data base (*3*) contains condition survey data taken on a regular basis since 1974 as well as associated traffic, environmental, and structural data for the pavement sections.

The seven distress indicators selected for CRCP are minor punchout, severe punchout, asphalt patching, Portland Cement Concrete (PCC) patching, transverse crack spacing, loss of ride score, and crack spalling.

INFERENCE SPACE FOR MODELS

The CTR rigid pavement data base was used for the analysis. The first step was to examine the inference space in the data base used for developing the models. This inference space, of course, determines the applicability of any model derived from the data.

Pavement Age

Because the desired models all predict distress as a function of age, several frequency distributions relating to pavement age were examined. Figure 1 shows the basic age distribution of the condition survey data. Every observation in the data base from 1974 to 1987 (the last year a survey was performed) is considered separately. Thus, a section built in 1964 and surveyed in 1974 and 1984 would produce two observations and be counted in the 10-year and 20-year bars on the graph. As can be seen from Figure 1 many observations are available over a wide range of pavement ages.

Using the date of first overlay field in the data base, a rough indication of CRCP performance can be plotted. Figure 2 shows the distribution of pavement life as indicated by years to first overlay. The mean time to first overlay was 16.7 years; most of the pavements were overlaid after 20 years.

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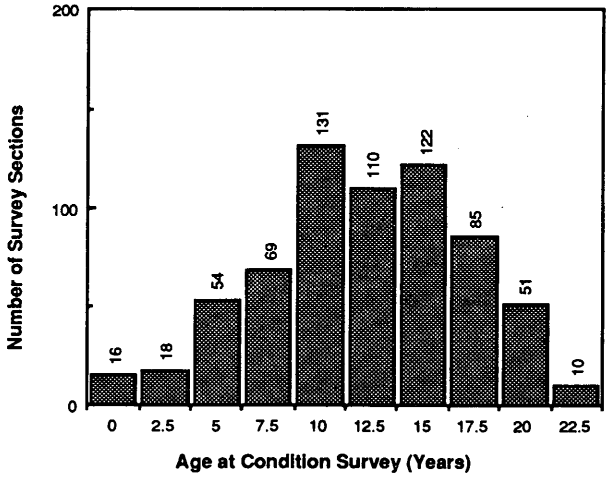

FIGURE 1 Age distribution of model inference space.

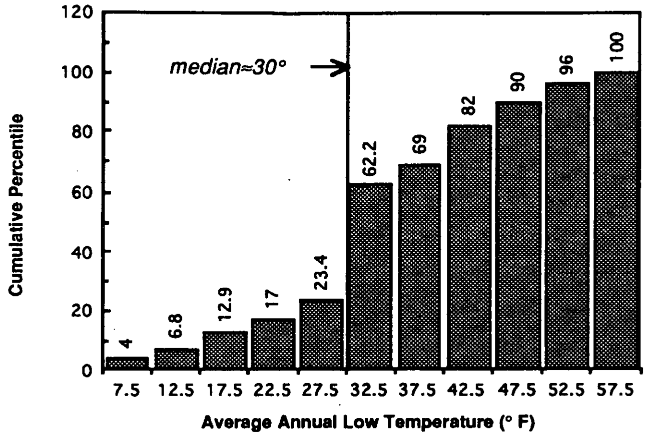

FIGURE 3 Average minimum temperature distribution (30°F = −1°C)

Temperature

Because environmental factors are expected to have an impact on the distress curves, a distribution of average annual minimum temperature (AAMT) was plotted. The AAMT (*4*) is the yearly minimum temperature recorded at the weather station nearest the pavement segment, averaged over the years 1951 to 1980. This is a potentially important variable, because the interaction of temperature with rainfall (freeze-thaw cycling) and the interaction of temperature with coarse aggregate type (thermal expansion in the aggregate) may play a role in cracking and punchouts. As shown by Figure 3, low temperatures in Texas vary greatly, from a minimum of 7.5°F to about 60°F (−9 to 15°C). A median low temperature of 30°F (−1°C) was selected as a separator level to differentiate "low" temperature conditions from "high" temperature conditions.

Rainfall

In a similar manner the distribution of rainfall was examined (Figure 4). A separator level of 30 in./year (75 cm) was chosen (*5*) to distinguish between high and low rainfall conditions in Texas; the median rainfall amount of 33 in./year (83 cm) found in this analysis agreed with that finding. Rainfall amount may also interact with soil type (swelling or nonswelling), which is already available in the data base.

Pavement Thickness

It is expected that thicker pavements will exhibit distress later (in terms of time and loading) than thinner pavements. Unfortunately, most survey sections in the data base are 8-in. (20-cm). Some thicker sections have been added recently and are being monitored. At this time, however, there are too few thick sections to contribute significantly to the analysis.

Traffic

It is expected that traffic history will have a significant effect on pavement distress. For the purposes of this analysis 18-kip equiva-

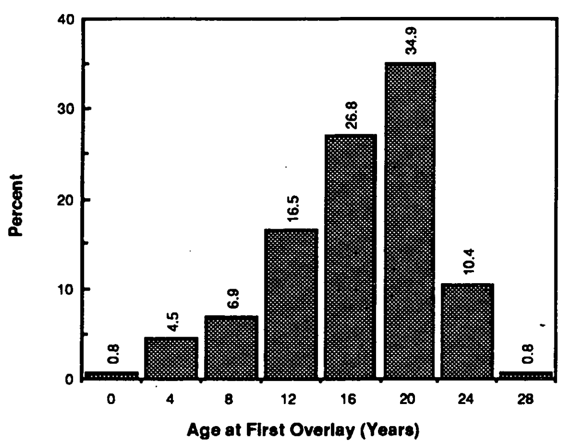

FIGURE 2 Pavement age at first overlay.

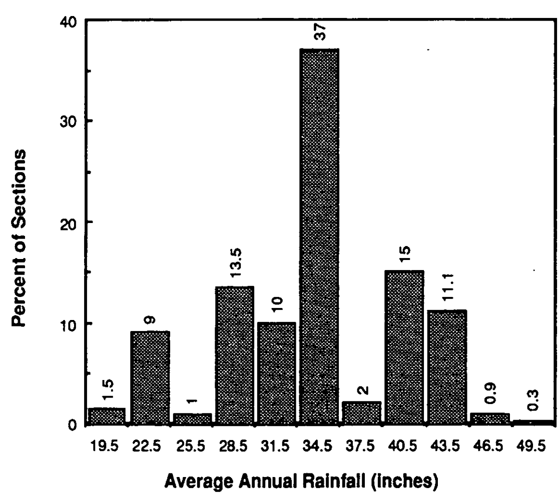

FIGURE 4 Distribution of average annual rainfall (1 in. = 2.56 cm).

lent single axle loads (ESALs) will be used. To fit into the predetermined PMIS scheme, traffic will not be treated as a continuous variable but as an adjustment to the time-distress curve in terms of high or low traffic.

Because loading has a cumulative effect on pavement performance, cumulative ESALs were calculated for each section from the time of construction to the date the section was surveyed. These detailed data were available only for a limited number of sections. At this point the data were analyzed only to determine a breakpoint for ESALs per year, which could be used to differentiate high traffic from low traffic. Figure 5 shows the average cumulative ESALs versus age for all the sections in the data base with detailed traffic data. From this analysis 1.4 million ESALs per year was chosen as the dividing line between high and low traffic.

ESAL figures given thus far are two-way ESALs across all lanes. Because only outside lanes were surveyed, a traffic distribution factor must be assumed. Approximately 75 percent of the data is for pavement with two lanes in each direction.

ANALYSIS OF VARIANCE

An analysis of variance (ANOVA) was performed to determine which factors were significant predictors for each distress type (Table 1). Age, cumulative ESALs since construction (CTRAF), average annual minimum temperature (TEMP), average yearly rainfall in inches (RAIN), coarse aggregate type (CAT), subbase treatment (SBT), swelling content of soil (SOIL), and their two-way interactions were examined. HT is highway type [Interstate highway (IH) or US highway] and is significant in terms of maintenance. On the basis of that analysis the following factors (in addition to age) were determined to be highly significant.

DISTRESS CURVES

Because pavement age was highly significant for every distress type and because few predictors may be available in the early implementation of the state PMIS, a preliminary analysis was performed by using only age as a predictor. Pavement sections older than 15 years were not used in the analysis, because after 15 years more than half of the sections had been overlaid and the remaining sections

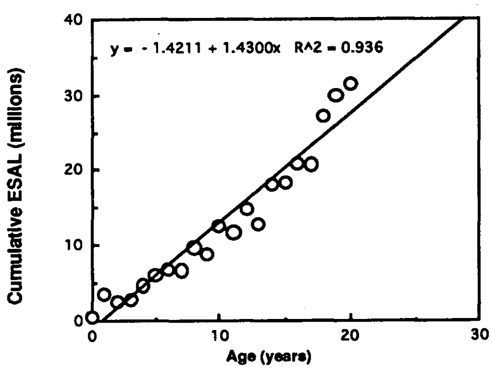

FIGURE 5 Average cumulative ESALs for data base sections.

TABLE 1 Significant Factors from ANOVA

	SIGNIFICANT FACTORS
Minor PUNCHOUT	SOIL, SOIL*CAT, TEMP*CAT, RAIN*SOIL
Severe PUNCHOUT	CAT*AGE, AGE*TEMP, SOIL, TEMP*RAIN
Port. Cmnt. PATCHES	AGE*CAT, AGE*SBT AGE*RAIN, AGE*HT
Asphalt PATCHES	AGE*TEMP, AGE*RAIN AGE*HT
CRACK SPACING	TEMP*CAT, CTRAF*RAIN, RAIN, RAIN*AGE
Loss of RIDE	* ANOVA not performed
SPALLED Cracks	AGE*CAT, AGE*RAIN, AGE*SBT

began to exhibit a "survivor effect." That is, any remaining data in the data base are nonrepresentative, because those 8-in. pavements that were weaker than average have already been overlaid. The NLIN procedure of SAS, a nonlinear least-squares analysis (*6*), was used to find the best-fit coefficients for the generalized sigmoidal function specified by TxDOT.

$$D = \alpha\, e^{-\left(\frac{\chi \epsilon \sigma \rho}{N}\right)^{\beta}}$$

where?

D = predicted level of distress,
N = age of the pavement,
α, β, and ρ = shape parameters estimated by regression,
χ = a factor to adjust for traffic,
ϵ = a factor to adjust for environment, and
σ = a factor to adjust for pavement structure.

To more clearly show the trend with time, weighted average values were used for the analysis. For all the following analyses χ, ϵ, and σ were fixed at 1.0, because it was expected that data would not be available in the initial implementation of PMIS to determine their values. Table 2 gives the best-fit coefficient values calculated for each distress type.

DISCUSSION OF RESULTS

Minor Punchouts

Figure 6 shows the fit for minor punchouts. A punchout is considered minor when several cracks have intersected but have not yet totally isolated a block of pavement. Data for punchouts (minor and severe) do not include repaired punchouts (patches). Considerable scatter is still evident (presumably because of extrinsic environmental, structural, and loading factors), but a clear trend with age is visible. This model will give a reasonable estimate when age is the only available predictor.

TABLE 2 Best-Fit Coefficients for Sigmoidal Function

	α	β	ρ
MPO/mi	82.9	1.33	18.6
SPO/mi	35	.577	144
ACP/mi	9.72	0.86	36.2
PCP/mi	146	1.23	40.3
CRACK SP SRG	34.9	1.00	0.06
CRACK SP LS	19.79	1.06	0.05
Loss of RIDE	0.269	1.00	1.00
Spalling (SRG)	2.02	6.06	10.0
Spalling (LS)	0.325	1.00	20.0

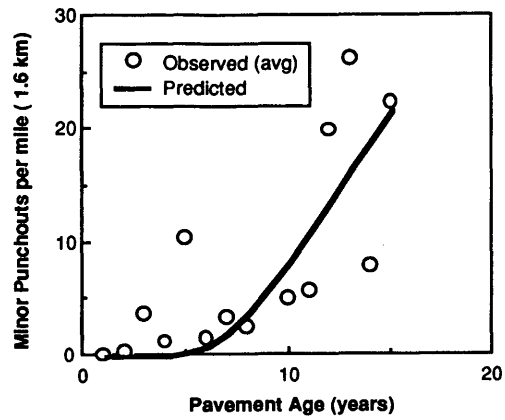

FIGURE 6 Prediction curve for minor punchouts.

Severe Punchouts

Figure 7 shows the fit for the severe punchout model. A punchout is considered severe only if the affected block is completely detached from surrounding pavement. In contrast to minor punchouts, the data show that severe punchouts take longer to begin development, but once they are started their development accelerates rapidly.

Asphalt Patches

Figure 8 shows the fit for the asphalt patch model. As for severe punchouts, the onset of patching is slow to begin, but once it has started it increases rapidly after 5 to 6 years.

PCC Patches

Figure 9 shows the prediction model for PCC patches. A very clear trend with age is evident; no pavements in the sample were patched within the first 5 years, and an inflection point is present around 10 years, after which the rate of patching increases steeply. Punchout and patch models give the number of occurrences per mile; multiply by 0.625 to find the number per kilometer.

Crack Spacing

An increase in the number of cracks per 100 ft (decrease in crack spacing) indicates poor pavement condition. Unlike the other distresses crack spacing does not vary drastically with age. Typically, most early-age cracking occurs within days of slab placement, and nearly all cracking has taken place by the end of the first winter after placement (7). Consequently, several other factors have as much or more influence than age, particularly coarse aggregate type. Be-

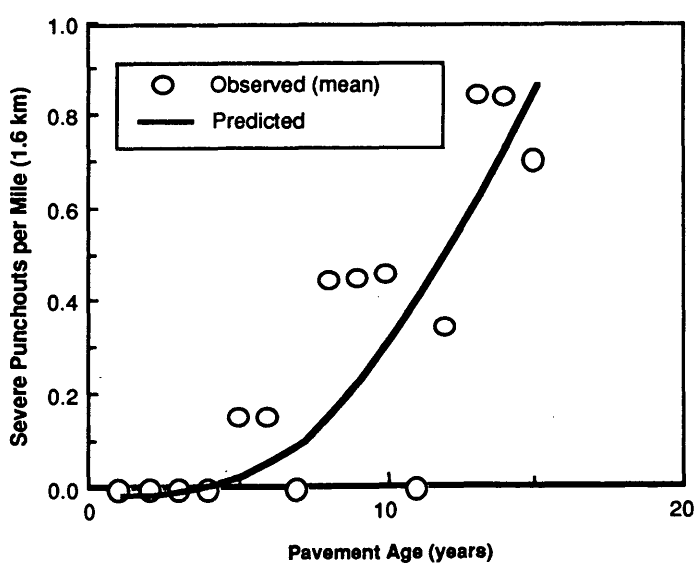

FIGURE 7 Prediction curve for severe punchouts.

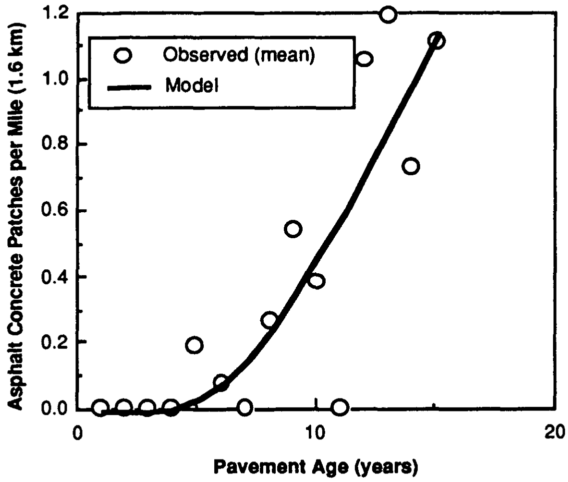

FIGURE 8 Prediction model for asphalt patching.

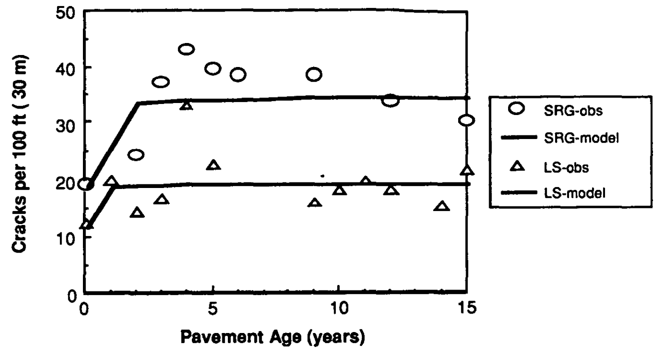

FIGURE 10 Crack spacing performance curves for LS and SRG aggregates.

cause of this two separate curves were fitted, one for limestone (LS) aggregate and another for siliceous river gravel (SRG) aggregate. The results are shown in Figure 10.

Figure 10 shows that crack spacing in LS aggregate pavements tends to decrease from around 8 ft to 5 ft (2.4 to 1.5 m) (20 cracks/100 ft) in a fairly short period of time and then stay basically constant thereafter. SRG pavement crack spacing often decreases with time to under 3 ft (0.9 m; 33 cracks/100 ft). Because 3 ft is the critical level for this distress, the slight rise in SRG crack spacing observed from 9 to 15 years is probably an early-age survivor effect because many of these pavements are overlaid at an early age. Additional scatter in the plot may be explained by other extrinsic factors, such as temperature and season of placement (especially if the peak temperature coincided with peak heat of hydration), which are only approximately known. Subbase friction, percent steel, and slab thickness may also play a role. Minimum temperature is included in the data base, and its interaction with aggregate type was found to be significant. This is probably because of the large difference in thermal coefficient between the two aggregate types (*5*). The crack spacing model predicts the number of cracks per 100 ft; this would be the same as the number per 30 m.

Ride Score

Additional data for ride score (and spalling) were available from archives of historical condition surveys conducted periodically by CTR in 1974, 1978, 1980, 1982, and 1984 (*8*). Approximately 300 projects were selected across the state. A project consists of a continuous length of pavement with homogeneous properties such as pavement thickness, coarse aggregate type, and traffic.

Figure 11 shows the distribution of the data relative to pavement age. Of the total of 8,878 sections surveyed, most were between 6 and 9 years old when surveyed. Although the distribution was skewed toward middle-age pavements, a sufficient number of younger and older pavements were available to proceed with the analysis.

Ride score was modeled as serviceability index (SI) *loss* versus age, normalized to a hypothetical initial SI of 4.5. The normalized SI loss (NSL) was calculated as follows:

$$NSL = (4.5 - PSI)/4.5$$

where PSI is present serviceability index. NSL ranges from 0 (PSI $\geq$ 4.5) to 1 (PSI = 0). For example, if the PSI of a section is 3.5, then the section is assumed to have lost 1 SI unit of ride quality, giving an NSL of 0.22. This means that the section has lost 22 percent of its initial smoothness. Figure 12 shows the fit to the data.

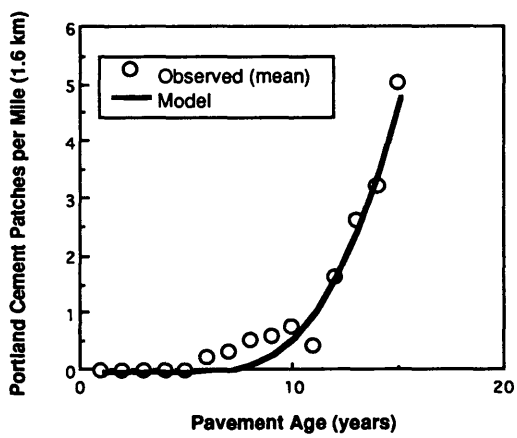

FIGURE 9 Prediction model for PCC Patches.

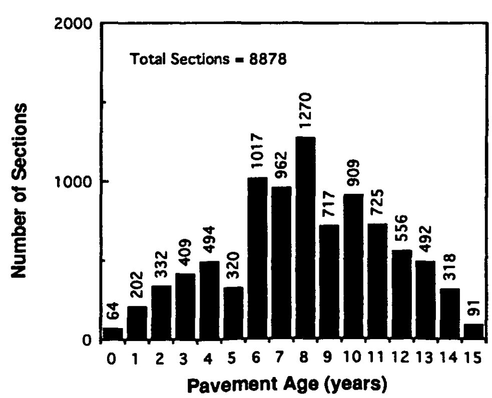

FIGURE 11 Age distribution of ride scores at time of survey.

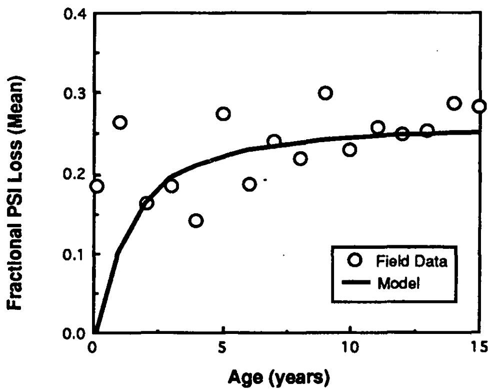

FIGURE 12 Best-fit model for loss of ride score.

When examining Figure 12 it should be remembered that the model was fit to the *weighted* average values of PSI; many more datum points were available for medium-age pavements than for 14- and 15-year old pavements (see Figure 11). Thus, the curve shown in Figure 12 passes through the 12- and 13-year points, which are heavily weighted, but is pulled down from the 14- and 15-year points, for which there were very few observations, and thus they had less of an effect on the regression.

Spalled Cracks

Data for spalling were obtained from the same source as the ride data. Spalling data were divided into two categories, minor and severe. Minor spalling was defined as "edge cracking where the loss of material has formed a spall of one half inch wide or less" (*9*). Because the PMIS definition of spalling (*10*) specifies spalling of "at least 1 inch (25 mm) wide," a decision was made to consider only the CTR severe spalling in the analysis.

As shown in Table 1 the interaction of age with coarse aggregate type was the best predictor for crack spalling; this was followed by the cumulative rainfall on the section (AGE · RAIN), the age of the section, and the interaction of age with the type of subbase treatment.

Figure 13 shows the development of severe spalling with age for several commonly used coarse aggregates. It is clear from Figure 13

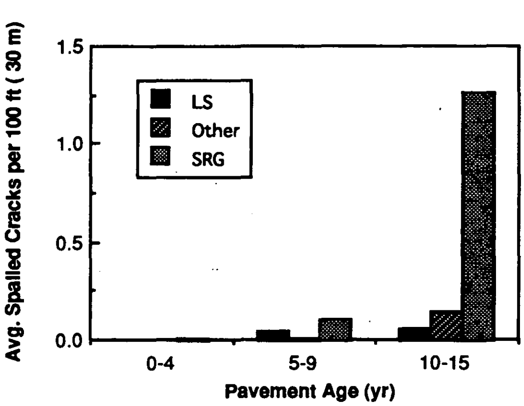

FIGURE 13 Spalled cracks by age and coarse aggregate type.

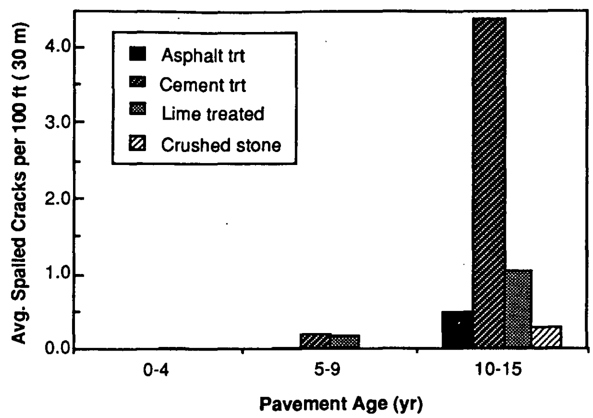

FIGURE 14 Effect of subbase treatment on spalling in SRG Pavements.

that spalling develops with age, apparently at an exponential rate. Pavements constructed with SRG coarse aggregate exhibited an average rate of spalling more than 10 times the rate of LS pavements. The aggregate type "other" in Figure 13 consists of blended LS and river gravel or sometimes slag blended with LS or gravel. These are grouped together because there are relatively few of them compared with the number of LS and SRG pavements. Because they often include some SRG material, it is reasonable that their rate of spalling would lie somewhere in between "pure" LS and SRG aggregates.

Figure 14 illustrates the relative effectiveness of the various subbase treatments, which were identified by the ANOVA as significant in predicting spalling rate. From the limited data available crushed stone gave the best performance; this was followed by asphalt-treated subbase. The worst choice by far was cement-treated subbase. However, the design of pavements in the field is not a controlled experiment; consequently, the choice of subbase treatment is not evenly distributed. If more study in this area is desired, a closer examination of the inference space in terms of subbase treatment is needed.

These extrinsic factors, aggregate type, rainfall, age, and subbase treatment, explain in part why many pavements exhibit no crack spalling at all whereas others are severely spalled. Because of the extremely different performances of LS- and SRG-based pavements, at least two curves are needed to adequately model spalling.

Spalling for CRCPs was expressed as the number of spalled cracks per 100 ft (30 m) of pavement. Weighted average values were used for the analysis. A composite curve modeling both aggregates would do justice to neither, so separate curves are suggested. Figure 15 shows the fit to the SRG pavement data.

Pavements made with LS coarse aggregate were much less prone to spalling. Figure 16 shows the fit to the LS pavement data.

CONCLUSIONS AND RECOMMENDATIONS

For the seven distress types modeled, all but two can be adequately described as functions of pavement age. In the cases of crack spacing and spalling the choice of coarse aggregate used is so important that it overwhelms age as a consideration and must be included in the model. Results from the ANOVA show that all the models could be improved by considering additional environmental, structural, and loading variables; the sigmoidal equation suggested by TxDOT

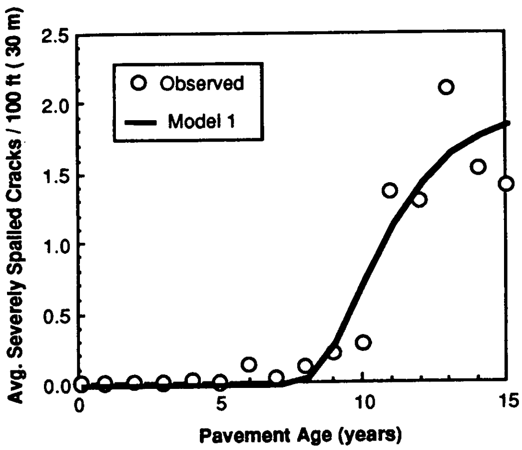

FIGURE 15 Spalling model (SRG).

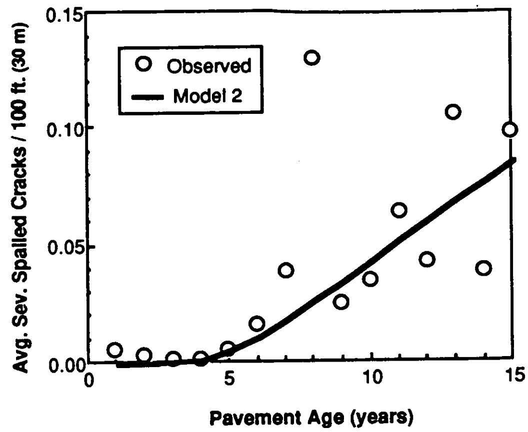

FIGURE 16 Spalling model (LS).

provides these factors. At the present time supporting data for the Texas PMIS are limited, and simple age-dependent models are all that is required. More work is needed to quantify the influences of these additional factors so that the models can be refined and improved as the data base expands.

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Analyzing Consequences of Pavement Maintenance and Rehabilitation Budget Scenarios

M. Y. (Mo) Shahin

A well-tested procedure for analyzing the consequences of various budget scenarios on pavement condition and backlog of maintenance and rehabilitation is presented. The procedure is part of the Micro PAVER system developed by the U.S. Army Corps of Engineers and distributed by the American Public Works Association. The procedure is based on the critical pavement condition index (PCI) concept. The critical PCI concept is explained, and the development of the work plan is demonstrated.

Local agencies including municipalities, military installations, and airports have long struggled to justify their pavement maintenance and rehabilitation (M&R) budgets. Pavements receive low priority as a budget item, especially when plans for preventive maintenance are made. Tools are needed for pavement managers to be able to demonstrate to local government officials the consequences of various budget scenarios, including the do-nothing option. When nothing is done or not enough money is allocated in the budget, the backlog of M&R increases and the cost of restoring the pavement infrastructure in the future becomes prohibitive.

This paper presents a well-tested procedure for analyzing the consequence of various budget scenarios on pavement condition and backlog of M&R. The procedure is part of the Micro PAVER system developed by the U.S. Army Corps of Engineers and distributed by the American Public Works Association. The procedure is based on the critical pavement condition index (PCI) concept and consists of the following steps:

- Perform a PCI survey on pavement sections in the network.
- Group pavement sections into families and develop a PCI deterioration curve for each family.
- Identify the critical PCI (the PCI below which the condition deteriorates rapidly) for each pavement family.
- Assign an appropriate M&R type to each pavement section for each year in the analysis period.
- Priority rank M&R requirements on the basis of budget limitations.
- Calculate M&R cost, future PCI, and backlog of M&R for each budget scenario.

The following is a description of each of the steps, and an example analysis of different budget scenarios for a pavement network is provided.

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PERFORM PCI SURVEY

The PCI is a score of 0 to 100, with 100 being excellent. It is determined on the basis of measured distress. A detailed description of the PCI is beyond the scope of this paper. The PCIs for roads (*1*) and airfields (*2*) have been described elsewhere. The PCI for airfields has been recently published as ASTM standard D5340-93, Standard Test Method for Airport Pavement Condition Index Surveys.

The pavement network is divided into uniform sections on the basis of use, pavement structure, construction history, traffic, and other factors. For the purpose of PCI inspection, pavement sections are divided into sample units. For asphalt pavements a sample unit is approximately 230 m^2 (2,500 ft^2) for roads and 460 m^2 (5,000 ft^2) for airfields. For concrete pavements a sample unit is approximately 20 slabs.

When performing PCI survey at the network level (versus the project level) very few sample units from each section need to be inspected. Analysis of budget scenarios is a network-level activity. The number of sample units to be inspected is a function of section size. The following table is an example inspection schedule used by one agency.

Section Size	*Survey*
1 to 5 sample units	1 sample unit
6 to 10 sample units	2 sample units
11 to 15 sample units	3 sample units
16 to 40 sample units	4 sample units
more than 40 sample units	10 percent

The sample units surveyed are selected to be representative of the average condition of the section.

DEVELOP PAVEMENT FAMILY DETERIORATION CURVES

The first step in developing the deterioration curves is to group the pavement into sections. The grouping is selected by the user on the basis of factors such as pavement type and use (Figure 1). For example, one family may include all secondary (collector) asphalt roads. When the data for a pavement family have been identified datum points are filtered to remove points in error.

The filtered data are then examined to identify datum points that are statistically outliers. The outlier procedure was developed on the basis of research findings (*3*) that the errors between the predicted and observed PCIs are normally distributed. A confidence interval is set by the user; data beyond this interval are identified as outliers.

Category	Example
• Branch Use	Roadway, Runway
• Pavement Rank	Primary, Secondary
• Surface Type	AC, PCC
• Zone	1200,1300
• Section Category	A, B
• Last Construction Date	> DEC / 30 / 1945

FIGURE 1 Pavement family definition.

Figure 2 is an example of outlier processed data with 95 percent confidence limits. The remaining datum points are then fitted with a fourth-degree polynomial by using the constrained least-squares procedure (Figure 3).

The PCI prediction at the section level is performed by assuming that the deterioration of all pavements in a family is similar and is a function of only their present condition, regardless of age. A section prediction curve is developed through the section's latest PCI/age point, parallel to its family curve (Figure 4).

IDENTIFY CRITICAL PCI FOR EACH FAMILY

A critical PCI is defined as the PCI value at which the rate of PCI loss increases with time or at the point when the cost of applying localized preventive maintenance increases significantly.

The critical PCI for a family is identified as follows:

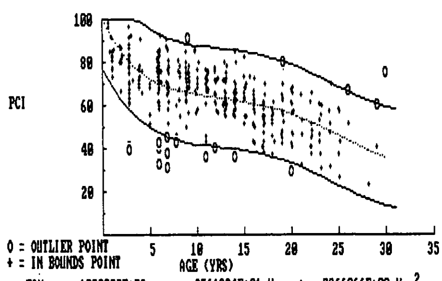


FIGURE 2 Outlier processed data file FNRSAC constrained to 4th-degree curve.

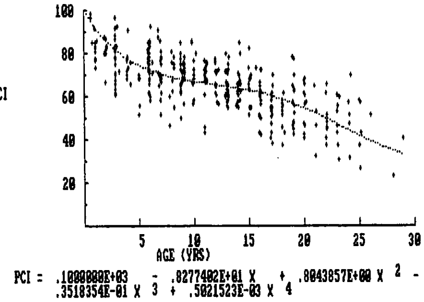


FIGURE 3 Constrained 4th-degree curve for data file.

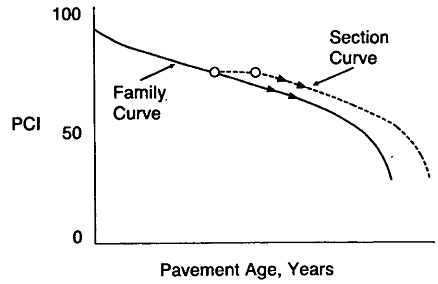


FIGURE 4 Section condition prediction.

1. Visually select the critical PCI range on the basis of the shape of the family deterioration curve (Figure 5).
2. Select a localized preventive distress maintenance policy to be used in the analysis of budget scenarios. Figure 6 is an example of such a policy for asphalt roads.
3. Apply the selected preventive distress maintenance policy to pavement sections in the family. This can be done by using the Micro PAVER network maintenance report.
4. Plot the cost of localized preventive maintenance per unit area versus PCI for each section. Figure 7 is an example of such a plot.
5. Select the critical PCI on the basis of the results from Steps 1 and 4.

ASSIGN M&R TO EACH PAVEMENT SECTION

There are seven M&R types in the current version of Micro PAVER: localized stopgap, localized preventive type, three global preventive types, and two major types. M&R types are assigned to each pavement section on the basis of the section's PCI with respect to the critical PCI (Figure 8). The following is a description of each of the M&R types.

Localized Stopgap (Safety)

Stopgap M&R is defined as the localized distress M&R activities needed to keep the pavement in a safe and operational condition. A stopgap policy is different from a preventive policy in that it will usually include only high-severity-level distresses that could be a safety hazard. Stopgap maintenance should only be applied to pavements with PCIs below the critical PCI.

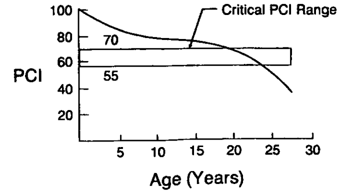


FIGURE 5 Critical PCI range on typical deterioration curve.

Policy Number: 2	Policy Description: PREVENTIVE , ROADS

Distress	Sev	Work Type & Description		Cost	Unit
1 ALLIGATOR CR	M	PA-AD	Patching - AC Deep	5.00	sq. ft.
1 ALLIGATOR CR	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
3 BLOCK CR	M	CS-AC	Crack Sealing - AC	.60	ft.
3 BLOCK CR	H	CS-AC	Crack Sealing - AC	.60	ft.
4 BUMPS/SAGS	M	PA-AS	Patching - AC Shallow	2.00	sq. ft.
4 BUMPS/SAGS	H	PA-AS	Patching - AC Shallow	2.00	sq. ft.
5 CORRUGATION	M	PA-AL	Patching - AC Leveling	1.00	sq. ft.
5 CORRUGATION	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
6 DEPRESSION	M	PA-AD	Patching - AC Deep	5.00	sq. ft.
6 DEPRESSION	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
7 EDGE CR	M	CS-AC	Crack Sealing - AC	.60	ft.
7 EDGE CR	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
8 JT REF. CR	M	CS-AC	Crack Sealing - AC	.60	ft.
8 JT REF. CR	H	CS-AC	Crack Sealing - AC	.60	ft.
9 LANE SH DROP	M	PA-AL	Patching - AC Leveling	1.00	sq. ft.
9 LANE SH DROP	H	PA-AL	Patching - AC Leveling	1.00	sq. ft.
10 L & T CR	M	CS-AC	Crack Sealing - AC	.60	ft.
10 L & T CR	H	CS-AC	Crack Sealing - AC	.60	ft.
11 PATCH/UT CUT	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
13 POTHOLE	L	PA-AD	Patching - AC Deep	5.00	sq. ft.
13 POTHOLE	M	PA-AD	Patching - AC Deep	5.00	sq. ft.
13 POTHOLE	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
15 RUTTING	M	PA-AD	Patching - AC Deep	5.00	sq. ft.
15 RUTTING	H	PA-AD	Patching - AC Deep	5.00	sq. ft.
16 SHOVING	M	PA-AS	Patching - AC Shallow	2.00	sq. ft.
16 SHOVING	H	PA-AS	Patching - AC Shallow	2.00	sq. ft.
17 SLIPPAGE CR	L	PA-AS	Patching - AC Shallow	2.00	sq. ft.
17 SLIPPAGE CR	M	PA-AD	Patching - AC Deep	5.00	sq. ft.
17 SLIPPAGE CR	H	PA-AD	Patching - AC Deep	5.00	sq. ft.

FIGURE 6 Preventive distress maintenance policy for asphalt roads.

Localized Preventive

Localized preventive M&R is defined as localized distress maintenance activities that are performed with the primary objective of slowing the rate of condition deterioration. These activities include crack sealing and various patching techniques. An example of a localized preventive distress maintenance policy is shown in Figure 6. This policy is applied to pavements above the critical PCI. It should be noted that application of a localized preventive maintenance policy to pavement sections with PCIs below the critical PCI is not cost-effective.

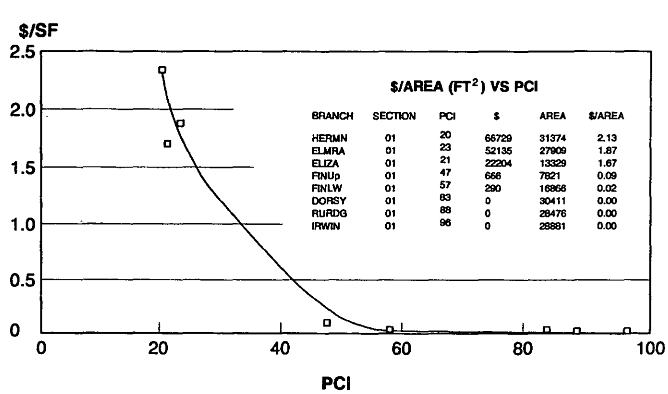


FIGURE 7 Localized preventive maintenance per unit area versus PCI for each section.

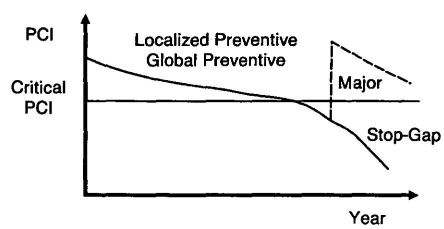


FIGURE 8 M&R types.

Global Preventive

Global preventive M&R is defined as those activities that are applied to the entire pavement section with the primary objective of slowing the rate of condition deterioration. These activities include surface treatments for asphalt-surfaced pavements and joint sealing for concrete pavements. Global preventive M&R is applied to pavements above the critical PCI. Applying global preventive M&R to pavements below the critical PCI is often not cost-effective.

The current version of Micro PAVER accommodates three types of global preventive M&R for asphalt-surfaced pavements. These three types are assigned to the pavement sections on the basis of existing distress types, as shown in Figure 9. This is done to optimize the selection of the surface treatment type on the basis of the existing distresses. Type 3 is recommended for pavements with skid causing distresses such as bleeding. Type 2 is recommended for pavements with climate-related distresses such as block cracking.

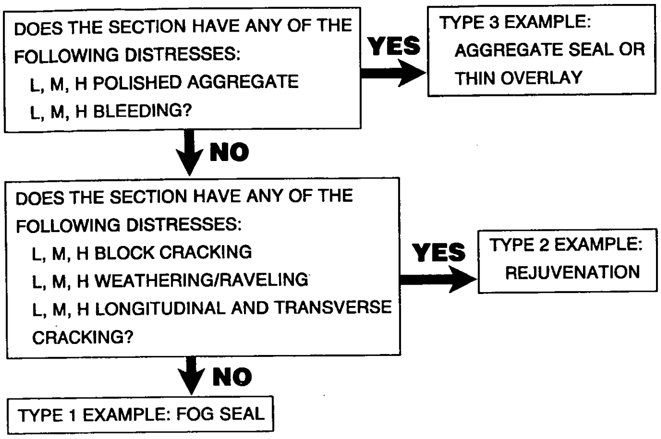


FIGURE 9 Process of assignment of one of three types to pavement sections.

The selection of the M&R type is also a function of the use of the pavement. For example, aggregate seals may not be appropriate for runways because of the possibility of foreign object damage to aircraft engines. Instead, a thin overlay should be used.

Major M&R

Major M&R is applied to the entire pavement section to correct or improve existing structural or functional requirements. Major M&R is divided into two types: major M&R applied to pavement sections above the critical PCI and major M&R applied to pavements below the critical PCI.

PRIORITY RANK M&R

Factors used to priority rank M&R include M&R type, pavement use, pavement rank, and PCI value. Priorities are first established on the basis of the M&R type as follows, with Priority 1 being the highest priority:

1. Localized stopgap (safety);
2. Localized preventive;
3. Global preventive, type 1;
4. Global preventive, type 2;
5. Global preventive, type 3;
6. Major, equal, or above the critical PCI; and
7. Major, below the critical PCI.

For M&R Types 1 through 5 priorities are assigned within each type on the basis of the section PCI, with the lower PCI receiving higher priority. For example, within M&R type 1 a pavement section with a PCI of 20 would receive a higher priority than a pavement section with a PCI of 50.

For M&R Types 6 and 7 priorities are assigned within each type on the basis of the user-defined criterion as shown in Figure 10. Pavement sections located within each major M&R type are ranked on basis of their PCIs, with the pavement with the lower PCI receiving a higher priority.

	Roadway		
PCI \ Rank	P	S	T
70 - 100	2	4	10
Critical - 70	1	3	9
40 - Critical	1	3	9
0 - 40	2	4	10

FIGURE 10 Major M&R priority table.

CALCULATE M&R COST

The calculation procedure is slightly different for pavement sections above and below the critical PCI.

Pavement Sections Above Critical PCI

The first step is to investigate whether the pavement section has a structural distress (Figure 11). Structural distresses include alligator cracking and rutting in asphalt pavements and corner break and divided slabs in concrete pavements.

Pavement Sections with No Structural Distress

1. Apply localized preventive M&R by using the preventive distress maintenance policy and the extrapolated distress data from the last condition survey. For subsequent years in which the PCI is predicted but the distress information is not available, the cost of localized preventive maintenance is estimated on the basis of the user-specified PCI versus unit cost relationship (Figure 12).
2. Apply global preventive M&R on the basis of the user-specified interval between applications. Also the maximum number of applications per pavement section should not be exceeded. The selection of the type of global M&R for asphalt pavements was presented in Figure 9.
3. If a global M&R is selected then the section's PCI will be increased as per specified value (Figure 13). A preferred method for

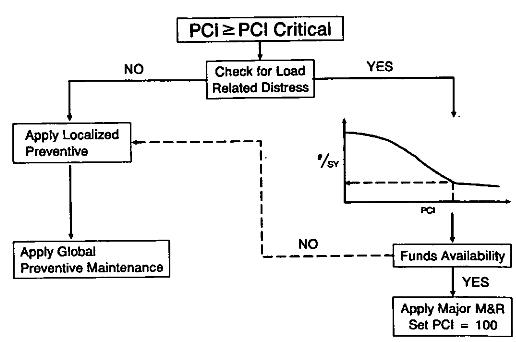


FIGURE 11 Investigation of pavement sections with PCIs above critical PCI.

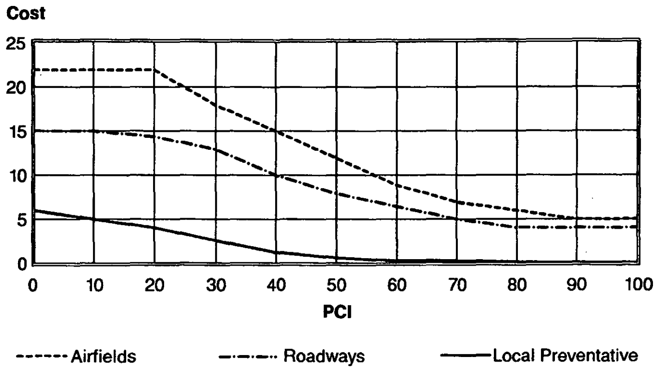


FIGURE 12 M&R cost/yd^2.

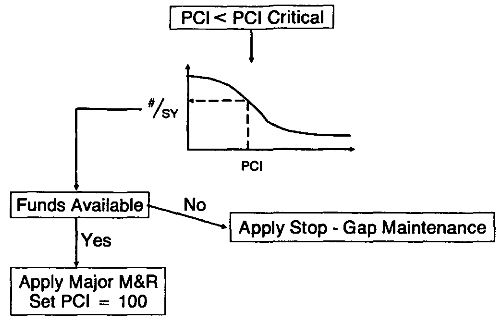


FIGURE 14 Investigation of pavement sections with PCIs less than critical PCI.

accounting for the effect of global preventive maintenance on pavement performance is to let the user specify the ultimate increase in pavement life (ΔT) and calculate the effective increase in PCI (ΔPCI) (Figure 13).

Pavement Sections with Structural Distress

1. Determine the cost for major M&R on the basis of the PCI versus unit cost relationship (Figure 12).
2. Check the availability of funds on the basis of the available budget and major M&R priorities.
3. If funds are available apply major M&R and set the PCI value to 100. If funds are not available apply the same process as described for pavement sections with PCIs above the critical PCI. Check on the availability of funding for major M&R in the following years.

Pavement Sections Below Critical PCI

1. Determine the cost for major M&R on the basis of the user-specified PCI versus unit cost relationship (Figures 12 and 14).
2. Check on the availability of funds on the basis of the budget and priorities.
3. If funds are available apply major M&R and set the PCI value to 100. If funds are not available apply localized stopgap (safety) M&R. Check on the availability of funding for major M&R in the following years.

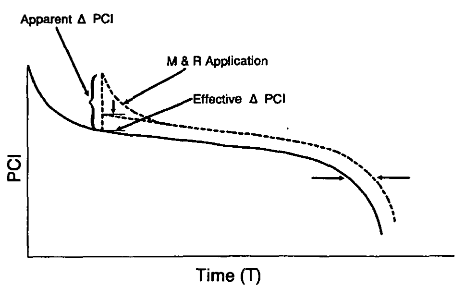


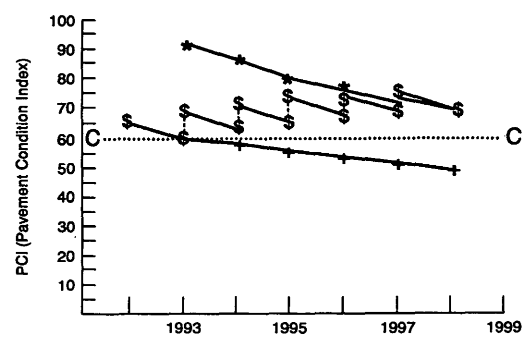
FIGURE 13 Determination of increase in PCI (ΔPCI) because of increase in life (ΔT).

EXAMPLE BUDGET SCENARIO ANALYSIS

The budget scenario analysis procedure presented previously has been automated as part of the Micro PAVER system. The following example is for a road network that is approximately 1.3 million m^2 (1.5 million yd^2). The network is divided into more than 200 pavement sections. The average PCI for the network was 66 in 1992.

Three budget scenarios were analyzed for the planning period 1993 to 1999 on the basis of an unlimited annual budget, an annual budget of \$100,000/year (stopgap budget), and an annual budget of \$2.0 million/year (affordable budget).

Figure 15 shows a comparison of the average network PCI for each of the three budget scenarios. The analysis showed that the \$100,000/year (stopgap budget) will result in a cumulative backlog of M&R (unfunded requirements) of \$14.6 million in 1999. The unlimited budget resulted in the lowest total cost of \$10.6 million, as compared with \$15.3 million for the budget of \$100,000/year. The unlimited budget, however, required \$6 million to be spent in the first year (1993), which was not obtainable. A \$2 million annual budget was approved, which resulted in a total cost of \$12.5 million.



	Budget Limit	M&R cost	Unfunded @ 1999	Total Cost
*	Unlimited	10.6M	0	10.6M
$	2M/YR	12.3M	0.2M	12.5M
+	100K/YR	0.7M	14.6M	15.3M

FIGURE 15 Network PCI-budget comparison.

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The views of the author do not purport to reflect the position of the U.S. Department of the Army or the U.S. Department of Defense.

Design Specifications and Implementation Requirements for State-Level Long-Term Pavement Performance Program

ATHAR SAEED, JOSE WEISSMANN, TERRY DOSSEY, AND W. R. HUDSON

The procedures for developing a state-level long-term pavement performance (LTPP) program for rigid pavement performance modeling for future pavement management efforts are documented. The program builds on principles already developed for the national-level Strategic Highway Research Program LTPP program. A method used to develop an experiment design that meets current pavement design standards is described. The description of the experimental design is followed by a discussion of the type of data that should be collected. The data items to be collected are divided into two categories: (a) inventory data items and (b) monitoring data items. Inventory data item sources are identified. The human and financial resources required to establish the data base and the resources required to maintain and monitor this data base periodically are reported. In addition to developing the design specifications and implementation requirements for a state-level LTPP program, the implementability of the procedure is evaluated through a case study LTPP program for Texas. The results are an important contribution to providing a basis for collecting the data items needed for the analysis to update current pavement design standards, direct needed research covering climatic and geographic needs, and improve pavement management system efforts overall.

The United States spends approximately $30 billion annually on highway and bridge infrastructure (*1*). This has received considerable attention at the public and private levels, and as a result significant research has taken place at the state and federal levels to address the problems of preserving this large investment.

In spite of the national concern about substandard highway condition, the United States has not systematically studied highway performance since the AASHO Road Test in 1958 to 1960 (*2*). That test was a massive experiment that provided a better understanding of the behaviors of pavements under load applications, but that also left many unanswered questions.

Under the sponsorship of FHWA and with the cooperation of AASHTO, TRB undertook in 1983 a study to investigate the effect of expanded research on improving highway transportation (*3*). The results of the study were reported in *TRB Special Report 202; America's Highways, Accelerating the Search for Innovation* (*3*). That study recommended six important research areas combined under one program called the Strategic Highway Research Program (SHRP).

The SHRP long-term pavement performance (LTPP) program, which has test sections in every state, can be used as a basis for developing a state-level LTPP program. The resulting data base would be used to develop and update distress prediction models, which are the basis of every pavement management system (PMS) and also contribute to revision of current design and construction procedures.

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EXPERIMENT DESIGN

The development of distress prediction models is based on the collection of distress data over a period of time. To support the data collection procedures an experiment design is required.

Given the large number of different factors that affect pavement performance, a factorial experiment design needs to be used to ensure that the effects of various factors can be investigated simultaneously. In factorial experiment design procedures the effects of each factor can be studied individually and their interactions with other factors can be examined. More information can be gathered about the true effects, with this methodology, than with experiments that test one factor at a time (*4–6*).

It has been well documented that the performance of continuously reinforced concrete pavements (CRCPs) is affected by moisture, temperature, subgrade swell characteristics, traffic, percent steel, and slab thickness, among others. These variables naturally become candidate factors to be included in a factorial experiment design (Figure 1.)

On the same principles, factorial experiments can be designed for all the pavement types that exist in a state. In summary, the factorial experiment design should accommodate all possible combinations of different variables judged to be of significance for the long-term performance of a particular pavement type.

MAIN FACTORS AND VARIABLES FOR DATA BASE

The manner in which a pavement performs in the field depends largely on the design concepts that were used to generate the pavement specifications. The construction quality and subsequent maintenance and rehabilitation activities carried out after construction to ensure a continuous level of performance comparable to the one when the pavement was new also have significant impacts on pavement performance (*7,8*).

A properly designed data base should provide easy access to processed data and information. The data base should be able to support the development of required models to explain the relationship between significant independent variables and the occurrence of deterioration and distress in the pavements.

To determine these required variables in the data base, empirical and theoretical models need to be analyzed. AASHTO design equations and mechanistic models, representing the two methods respectively, were evaluated to determine the significant variables that affect the performance of rigid pavements. Furthermore, analysis of variance (ANOVA) performed on existing data bases also guided the selection of significant variables (*9,10*).

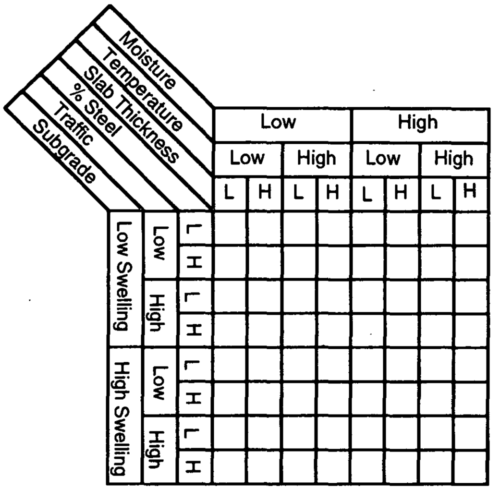

FIGURE 1 Factorial experiment design for CRCP.

TABLE 2 List of Candidate Variables from Mechanistic Models and ANOVA

Variable Type	Variable
Climatic	Temperature Drop
	Moisture Change
Design	Coarse Agg. Type
	Subbase Type
	Slab Thickness
	Traffic (18-kip ESAL)
	Steel Elastic Modulus
	Conc. Elastic Modulus
	Coef. of Load Transfer
Performance	Crack Width
	Crack Spacing
	Traffic (18-kip ESAL)

By evaluating the empirical and mechanistic equations combined with the statistical analysis discussed previously a list of candidate variables that have a significant effect on pavement performance was compiled. The list of candidate variables for rigid pavements on the basis of empirical models and ANOVA is presented in Table 1.

Table 2 lists the candidate variables obtained on the basis of theoretical or mechanistic models.

It can be inferred from Tables 1 and 2 that the data items belong to two broad categories. The first is the inventory data items and the second is monitoring data items.

Inventory data include such data items that remain the same during the monitoring period. There is only a very slight probability that the inventory data will change during the life of the pavement. The inventory data items include information required for the proper identification of the test section, construction material properties, geometric details, environmental conditions, construction dates, and costs. If a rigid pavement is overlaid, usually near the end of its service life, the pavement type then changes to composite pavement, so the inventory data need to be updated to consider these changes (*11*).

Monitoring data include the variables that change with time, that require periodic evaluation and measurements during the monitoring period, and that require constant updating to keep the data base current. Information concerning distress, serviceability, and deflection measurements should be a part of the monitoring data, which should also include traffic and axle load data. Maintenance and rehabilitation costs incurred during the monitoring period are also included. Data are collected on an annual basis most of the time but could also be based on some other reasonable time period. All this information should be collected in a historical data base required to study the relationship between distress, performance, age of the pavement, traffic and axle loading, and maintenance and rehabilitation procedures.

TABLE 1 List of Candidate Variables from AASHTO 1986 Design Guide

Variable Type	Variable
Climatic	Temperature
	Moisture
Design	Coarse Agg. Type
	Soil Type
	Subbase Type
	Slab Thickness
	Traffic (18-kip ESAL)
Performance	Roughness
	Cracks
	Patches
	Traffic (18 kip ESAL)

TEST SECTION IDENTIFICATION AND MONITORING COSTS

The test sections must be located on the ground so that the data collection crews can perform the data collection process adequately. Data collection costs can present a financial burden on the budget within the agency if they are not managed properly. Because of financial constraints priorities should be set to collect required data first and optional data later.

Test Section Identification Costs

The identification and location of the test section physically on the ground remain problems. Some permanent form of test section identification should be used, such as the reflectorized signs being used by the SHRP LTPP program. Besides using these signs, the beginning and end of the test sections should also be marked with white paint. The cost of paint, the manufacturing costs of the sign, and the cost of placing the sign at the proper location combined constitute the test section identification costs.

Inventory Data Collection Costs

State departments of transportation (DOTs) maintain a comprehensive set of project construction plans. Almost all the inventory data requirements can be met from these records and require little or no effort on the part of DOT personnel.

Monitoring Data Collection Costs

Monitoring data to be collected relates to the distress and the performance of the pavement. It also includes the traffic data and deflection measurements required to evaluate behavior. The distress and performance data collection requires visual distress surveys. Deflection measurements require testing with a falling weight deflectometer (FWD) or similar instrument. Traffic data are also collected on a regular basis. Traffic control is required when visual distress surveys and FWD tests are being conducted, and this adds to the data collection cost.

TABLE 3 Activities That Contribute To Test Section Setup and Cyclic Monitoring Costs

Activity
I. Test Section Set Up Costs
Identification signs manufacturing cost
Install test section identification signs
Test section paint details marking
Test section material sampling
First time monitoring
Traffic data collection
II. Test Section Cyclic Monitoring Costs
Test section monitoring
Traffic data collection
Test section maintenance

Field Materials Sampling Costs

A comprehensive plan is followed by the SHRP LTPP program to obtain field material samples. Coring and augering are conducted at the test site to collect field material samples (*12*). It is recommended that a similar plan be used to obtain field samples for the state LTPP program. Traffic control will be required while the material samples are being collected.

Traffic Data Collection Costs

To collect detailed traffic data, weigh-in-motion (WIM) stations would be desirable at all the test sections. Data from these WIM sites could be supplemented by the installation of automatic vehicle classifiers (AVCs) at test sections that lack a WIM setup. Detailed weight data are provided by the WIM station for each class of vehicles. Classification data from the AVC device are then supplemented with WIM data from nearby locations to estimate the equivalent single axle loads (ESALs) at a particular site.

Travel Time Costs

Test sections may not be located in every district of the DOT. The size of the state may make travel time an important consideration when scheduling personnel and equipment for data collection. Distances should be calculated along the most direct route to the test section.

Certain other factors such as unforeseen weather, equipment breakdown, the effect of fatigue on personnel efficiency, and the time spent on locating the test section physically in the field on arriving in the general area could not be predicted accurately. To take into account all these factors, the estimated total time was increased by 5 percent (*10*).

Table 3 lists all data collection activities that, when combined, constitute the initial setup costs and the cyclic monitoring costs. Some of these costs are one-time expenses, such as the material sampling and testing and the costs incurred to set up signs for test section identification. Other expenses will be required periodically to maintain and monitor the test sections at a regular interval as selected by the state DOT.

CASE STUDY: TEXAS LTPP PROGRAM FOR RIGID PAVEMENTS

By using the principles discussed earlier, an LTPP program was developed for rigid pavements in Texas. A factorial experiment design was developed for each of the rigid pavement types in Texas. This sampling template or experiment design was used to select the parameter test sections after the determination of predominant pavement types in each Texas DOT (TxDOT) district. The data items to be collected were determined, and the costs associated with collection of data items were also estimated in terms of man hours required.

Factorial Experiment Design

On the basis of the variable determination procedure described earlier, three experiment designs were proposed, one each for CRCP, jointed plain concrete pavement (JPCP), and jointed reinforced concrete pavement (JRCP). Figures 2, 3, and 4 show the proposed factorial experiment designs for the three pavement types, respectively. The traffic factor midpoint for these experiments is 1.7 million 8.2-ton (18-kip) ESALs/year. A slab thickness of 22 cm (8.5 in.) and an age of 15 years are the respective midpoints for the factors of slab thickness and age in the CRCP factorial experiment design. A thickness of 25.4 cm (10 in.) demarcates between high and low thicknesses for JPCP and JRCP experiment designs.

Data Items To Be Collected

Empirical and theoretical models besides ANOVA on existing Texas data bases (*9,10*) were used to determine the data items to be collected for the Texas LTPP program for rigid pavements. Tables 4 and 5, show the inventory data items and the monitoring data items, respectively, to be collected for the Texas LTPP program.

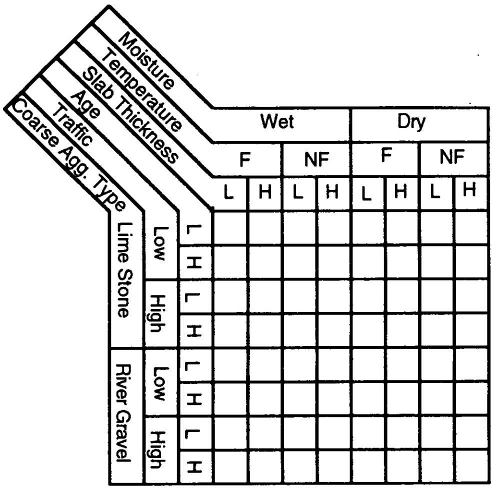

FIGURE 2 Proposed CRCP factorial experiment design for Texas LTPP program.

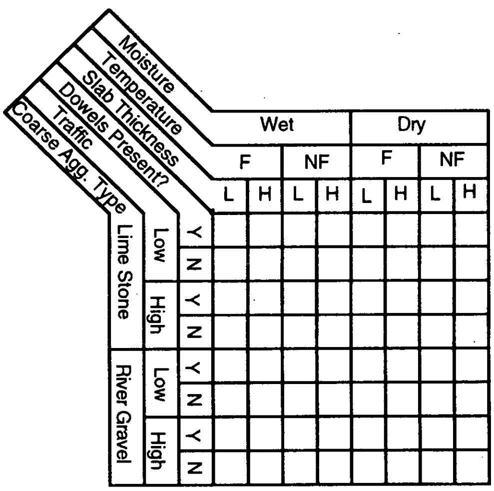

FIGURE 3 Proposed JPCP factorial experiment design for Texas LTPP program.

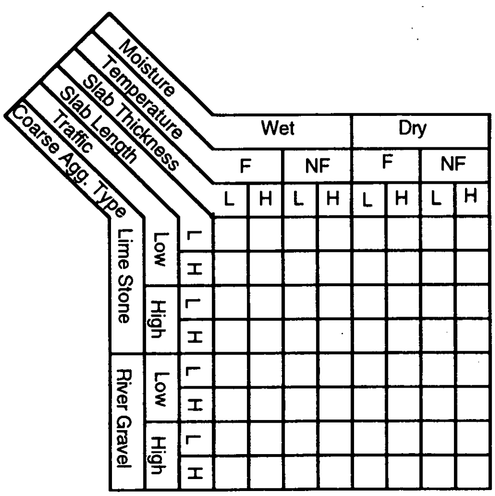

FIGURE 4 Proposed JRCP factorial experiment design for Texas LTPP program.

TABLE 4 Inventory Data Items To Be Collected

Data Type	Data Items to be Collected
Identification	Functional Class of Highway
	Number Designation, Direction
	Pavement Type
	Rural / Urban
	Test Section Location, No. of Lanes
	Construction Date
Geometric Details	No. and Width of Lanes
	Shoulder Presence, Type and Widths
	Drainage Effectiveness
	Joint Spacing
	Dowel Presence, Diameter, Spacing
	Severity and Extent of Existing Distress
Climatic	General Type (Dry Freeze etc.)
	Annual, Monthly Rainfall
	Highest, Lowest Mean Monthly Temperatures
	Freeze Thaw Cycles per Year
	Freeze Index and Thorntwaite Index
Accumulated Traffic	Total and Mean AADT for previous years
	18-kip ESAL, % Trucks
	No., Distribution of Tandem Axles
	No., Distribution of Single Axles
	No., Distribution of Triple Axles
Material Properties	Layer Thicknesses
	Subgrade Soil Type, Classification (especially swelling or not)
	Subbase Soil Type, Classification
	Stabilization Presence, Type
	PCC Moduli of Rupture & Elasticity
	PCC Steel Content, Steel Modulus of Elasticity,
	PCC Coarse Aggregate Type
Accumulated Costs	Initial Construction Cost
	Maintenance & Rehabilitation type, Date Performed and Costs

Test Section Identification and Cyclic Monitoring Costs

TxDOT will carry out all PMS activities in-house with its own personnel and resources. The costs are calculated in terms of the number of man hours, required, and no equipment costs have been estimated.

To manufacture a sign measuring 30 × 38 cm (12 × 15 in.) the cost is approximately $6.00. To put on the white reflectorized paint the cost is usually $5.00. The lettering costs $5.00 per linear meter (3.28 ft). Furthermore, it usually takes two persons half an hour to place the sign in the ground. So, for test section identification purposes the total cost is approximately $18.00 along with one man hour for sign placement in the ground.

Figure 5 shows the average time required to conduct a visual distress survey and FWD testing. The number of man hours required to conduct these activities is given in Table 6.

TABLE 5 Monitoring Data Items To Be Collected

Data Type	Data Items to be Collected
Distress, Performance	Transverse, Longitudinal and Slab Cracking, D - Cracking
	Joint Faulting, Pumping
	Roughness, Patches, Skid Resistance
	Joint, Crack Deterioration
	Lane - Shoulder Separation
Traffic	AADT, Percentage Trucks
	18-kip ESAL for the Time Period
	No., Distribution of Tandem Axles
	No., Distribution of Single Axles
	No., Distribution of Triple Axles
FWD/Deflection Tests	Mean Max. Deflection Under Load
	Deflection Observations, Basin, Loading
	Pavement Temperature

Figure 6 shows the time required to conduct field material sampling. A total of 18 cores are recommended at the beginning and end of the test section (*12*). Table 7 shows the man hours required to obtain field material samples. The cost in man hours required to monitor traffic at a test section is presented in Table 8.

Test sections are not located in all TxDOT districts primarily because not every district has rigid pavements. The size of Texas makes travel time an important consideration when scheduling personnel and equipment for data collection. The data collection party consisting of four persons, two for visual distress survey and two for FWD testing, must travel 11,363 km (7,058 mi.) to collect data. The travel times are calculated at a constant 80 km/hr (55 mph), although most of these are on an Interstate, to compensate for brief stops, for example. A total of 514 man hours must be added to the total time required to complete all the tasks for the network to take into account the time spent in traveling to and from the districts. Table 9 calculates the total man hours required to set up and monitor a total of 100 test sections in Texas.

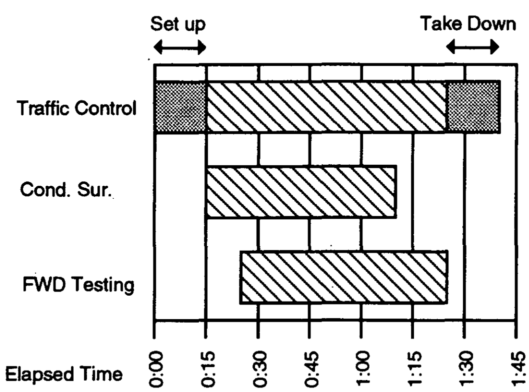

FIGURE 5 Average time required to conduct condition survey and FWD testing.

TABLE 6 Man Hours Required To Conduct Visual Distress Survey and FWD Testing

Testing

Activity		Time Required	Persons Required
Traffic Control		0:30	3
Condition Survey		0:55	2
FWD Testing		1:00	2
	Total	2:25	7
	Total Cost	**2:25 x 7 =**	**16.92 man-hours**
			17.00 man-hours

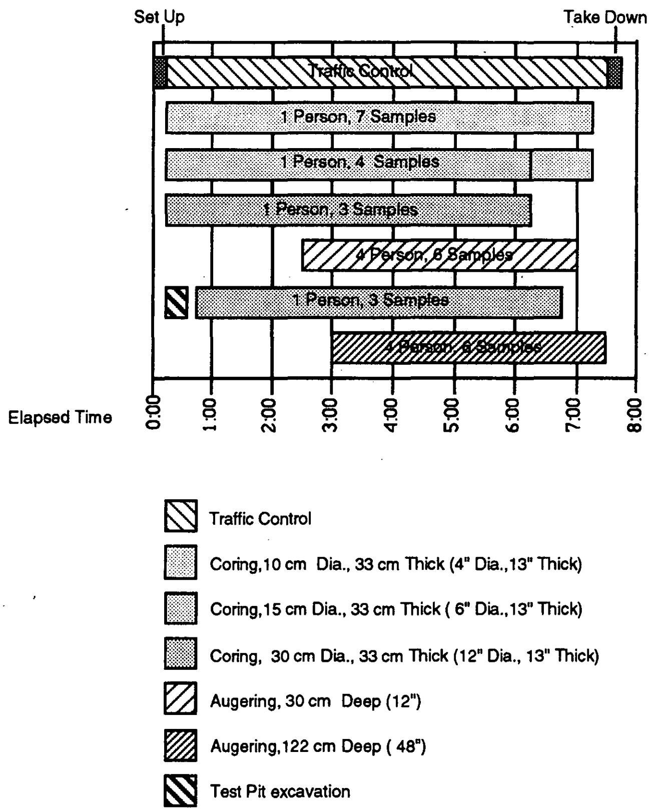

FIGURE 6 Time and persons required to obtain field material samples.

TABLE 7 Man Hours Required To Conduct Materials Sampling

Activity		Time Required	Persons Required
Traffic Control		8:00	3
Coring		8:00	4
Augering		8:00	8
	Total	8:00	15
	Total Cost	8:00 x 15 =	120 man-hours

TABLE 8 Traffic Data Collection Costs

Method	Man-hours Required
I. portable WIM Site	6 persons X 2 - 8 hour shifts
	Total 96 man-hours
II. Radian WIM Site	3 persons X 4 - 8 hour shifts
	Total 96 man-hours
III. Permanent WIM Site	Data down loaded by the Department main frame automatically

CONCLUSIONS

This paper documents a procedure that can be used to set up a state-level LTPP program or to modify or set up a LTPP program in any state or national highway agency. The program is based on guidelines that were originally developed for the national-level LTPP program conducted in the United States beginning in 1987 and continuing under FHWA leadership. Implementation of a state-level LTPP program should provide data that will improve models and thus facilitate better pavement management to safeguard the large investment that each agency makes in its highway pavement infrastructure.

The procedures presented here have been used to develop balanced factorial experiment designs for the predominant pavement types used in Texas. If an agency uses other pavement types, the experiment laid out here will not be applicable, but the procedure can be used to define the experiment design for other pavement types.

If a state is part of the national LTPP program, this may not be sufficient, and indeed probably will not be sufficient to produce a model for direct use in the state, because the national LTPP program is balanced nationally and not within a given state.

One may also find that some of the specifications for developing test sections in the national LTPP study are not applicable to state-level study. In this case one must modify those specifications to accommodate the geographic and climatic needs of an area. Because great thought was given to the national LTPP protocol, however, care should be taken before it is cast aside. In the present study it was determined early that the costs of some of the national LTPP testing protocols were far too expensive for continuation in the long term within Texas. Thus, it was necessary to simplify the effort.

Notwithstanding the conclusions outlined, it is the summary conclusion of the authors that it is possible for any state or other agency to set up an LTPP program within an agency by properly allocating resources and experiment design. Such a well-designed and developed LTPP study should be carried out diligently to provide the core information needed to update pavement management models within the agency. Continuity in the program is far more important than extravagant detail. This is one of the major problems from which the national LTPP study currently suffers.

TABLE 9 Summary of Test Section Setup Costs and Annual Section Monitoring Costs in Terms of Man Hours Required

Activity	Man-Hours Required per Test Section
Man-Hours Required for Test Section Set Up	
100 Test Sections X 238	= 23,800 man-hours
100 Test Sections X $ 18	= $ 1,800
Travel Time	= 514 man-hours
Climatic Factors (5%)	= 1,216 man-hours
Total	= 25,530 man-hours plus $ 1,800
Man-Hours Required for Test Section Monitoring per Round	
100 Test Sections X 115	= 11,500 man-hours
Travel Time	= 514 man-hours
Climatic Factors (5%)	= 600 man-hours
Total	= 12,614 man-hours

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Impact of Different Economic Criteria on Priorities in Pavement Management Systems

VERA MIJUŠKOVIĆ, DRAGAN BANJEVIC, AND GORAN MLADENOVIC

The results of a recent study conducted at the Faculty for Traffic and Transport Engineering, University of Belgrade, Belgrade, Yugoslavia, are presented. The study was performed to investigate whether there is any theoretical principle that could be implemented as a guiding rule when defining a capital pavement maintenance strategy. Road network pavement deterioration and repair were described by the controlled nonhomogeneous Markov process. Six guiding rules were defined and a theoretical proof was provided, that is, that the rule "the sections in the worst state have the first priority" could never be the best strategy when considering the effects on a network as a whole when budgets are restricted. The results of a computer program based on these principles are presented.

The basic dilemma of spending a restricted budget during a longer period could be defined as whether

1. To repair a greater extent of the network in 1 year with the measures of a lower standard or
2. To repair a minor extent of the network in 1 year, but achieving at once an excellent state.

The results of many studies have shown that the staged construction could almost never be the best strategy; only a severe budget restriction could impose the achievement of a long service life of pavements in two or three steps. So we considered only Orientation 2 of the maintenance strategy. An additional question was imposed: In what way does the sequence of repair work influence the speed of improving the overall quality of the network? This question is treated in the Pavement Management Forecasting System (PMF) (*1*), which considers only the effects on the quality of the network itself without an analysis of costs to users or of accidents.

One of the most serious obstacles for the implementation of the modern software for pavement management in countries with poor economies is the lack of reliable data. The devices for measuring a bearing capacity and for the calibration of surface deficiencies are expensive. Moreover, it takes years to quantify the parameters describing pavement behavior under local climate conditions and the conditions of the physical environment. This can postpone the introduction of a modern management system in many regions with poor economies.

Homogeneous Markov chains are frequently used to describe pavement deterioration over time on a network as a whole. The probability that a certain section would remain in the same state or pass into a worse state in regions where periods of intensive road construction are followed by periods of almost total absence of investment depends on the moment considered. That is why the homogeneous process is not convenient for the description of the behavior of such a road network.

For these reasons the present study had the following objectives:

- To establish a mathematical model that will present the impact of the amount of the available budget on the network quality as a whole in a simple and as real a way as possible;
- To analyze the effects of strategies defined as principles to obtain instructions, that is, guiding rules, for the definition of the projects in more detailed models;
- To establish criteria that will separately reflect the road manager's and users' interests and quantify their mutual relations under various circumstances;
- To develop a computer program that will contain all the stated theoretical assumptions and enable the use of currently available data, but at the same time allow the use of more precise data to be acquired in the future; and
- To determine the trigger values of the traffic volume for which particular strategies are competent on the basis of such a program.

MATHEMATICAL MODEL

The mathematical model was established in two phases:

- A simple alternative was set up to make clear whether any regular relationship existed between the sequence of repair works and their effects, and
- An alternative closer to practice was also set up.

Starting Assumptions

Because these considerations were primarily of a theoretical character and a very poor data base was available, the mathematical model was based on the following assumptions:

1. The change in the condition of the road network can be described by nonhomogeneous Markov chains. The probability of a change in the condition of a certain road section depends on the relation between the length of the service life and the time spent in operation. Because no systematically collected road network condition data were available, the only reliable data—the year of

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construction, the length of the design period, and the type of intervention during the past period—had to be used.

2. Road network sections were classified into a finite number of states (four) on the basis of an indicator that was decisive (or in correlation with the decisive indicator) for the maintenance treatment. Because the model was based on pavement performance curves, which were also used for design, and served primarily for the choice of principles, not for the choice of the treatment type, only four states were adopted. The pavement states were delimited in such a way that improvement to the excellent state might be obtained by the following interventions:

- From a good to an excellent state: surface treatment and thin overlay without increasing the bearing capacity,
- From a fair to an excellent state: strengthening, and
- From a poor to an excellent state: reconstruction (similar to the PMF model).

3. All road network sections behave according to the same deterioration model provided by a standard design procedure. Although different design methods were applied in the previous period, the performance curve expected by the standard method of pavement design was adopted.

4. Interventions either turn pavement back into an excellent condition or do not influence the pavement such that it changes into any other condition. This means that routine maintenance costs depend on the pavement condition but do not influence the probability of the change of condition. If small repairs are not performed in time, pavement deterioration is accelerated, but no reliable quantification could be done with the available data. If appropriate information on those relations were available, that information could be entered into the program by adapting the input data.

Description of Road Deterioration and Repair

The entire road network condition can be described by the state vector of the network:

$$\alpha_k^i = (\alpha_{1k}^i, \alpha_{2k}^i, \alpha_{3k}^i, \alpha_{4k}^i) \qquad \sum_{j,k} \alpha_{j,k}^i = 1 \tag{1}$$

where α_{jk}^i presents the participation of sections in j state of roads in class k in the ith year on the entire road network, where j is 1 for excellent condition, j is 2 for good condition, j is 3 for fair condition, and j is 4 for poor condition (Figure 1).

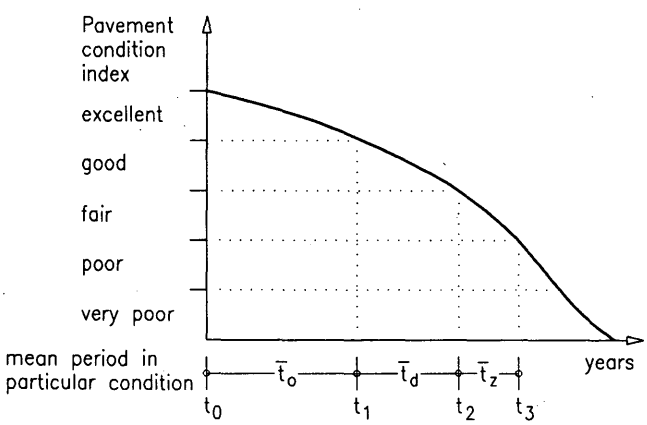

FIGURE 1 Average time of pavement service in particular condition.

The classes of roads present the road network classification according to traffic volume categories expressed by the average annual daily traffic (AADT). The limits among individual classes have been determined on the basis of a traffic survey, so that each class (k = 1, 2, 3, 4) covers a typical traffic composition (Figure 2) in the following way (where vpd is vehicles per day):

AADT1:		AADT	>10,000 vpd
AADT2:	10,000>	AADT	> 5,000 vpd
AADT3:	5,000>	AADT	> 2,000 vpd
AADT4:	2,000>	AADT	

If p_{jk}^i presents the probability that the road network section in class k of traffic volume in the year i will remain in the state j and $1 - p_{jk}^i$ is the probability that it will pass into a worse state, then the transition matrix for the network without interventions (except routine maintenance) will look like

$$P_{ik} = \begin{vmatrix} p_{1k}^i & 1-p_{1k}^i & 0 & 0 \\ 0 & p_{2k}^i & 1-p_{2k}^i & 0 \\ 0 & 0 & p_{3k}^i & 1-p_{3k}^i \\ 0 & 0 & 0 & 1 \end{vmatrix}$$

where p_{1k}^i, p_{2k}^i, and p_{3k}^i are natural conditions, that is, the consequence of the general state of pavement on the part of network in the kth class. Because interventions on the network either change it into an excellent condition or do not influence its condition at all, the transition matrix for the network expected to be improved will be

$$P_{ik} = \begin{vmatrix} p_{1k}^i & 1-p_{1k}^i & 0 & 0 \\ a_{2k}^i & p_{2k}^i \cdot (1-a_{2k}^i) & (1-p_{2k}^i)\cdot(1-a_{2k}^i) & 0 \\ a_{3k}^i & 0 & p_{3k}^i \cdot (1-a_{3k}^i) & (1-p_{3k}^i)\cdot(1-a_{3k}^i) \\ a_{4k}^i & 0 & 0 & 1-a_{4k}^i \end{vmatrix}$$

where

a_{2k}^i = participation of network length of the kth class in good condition improved at the ith step (year) to excellent condition by surface treatment or overlay $d \leq 4$ cm,

a_{3k}^i = participation of network length of the kth class in fair condition improved at the ith step to excellent condition by strengthening, and

a_{4k}^i = participation of network length of the kth class in poor condition improved at the ith step to excellent condition by reconstruction.

The elements a_{jk}^i must be determined by the optimization process in the context of the available budget.

If the average time that the pavement remains in excellent condition is t_o years, and if t_d and t_z are periods for good and fair condition, respectively, then

$$p_{jk}^i = \frac{\sum_{l=t_{j-1}}^{t_j-1} s_{lk}^i}{\sum_{l=t_{j-1}}^{t_j} s_{lk}^i} \tag{2}$$

where s_{lk}^i is the participation of road network length of class k being in operation l years considered in ith year.

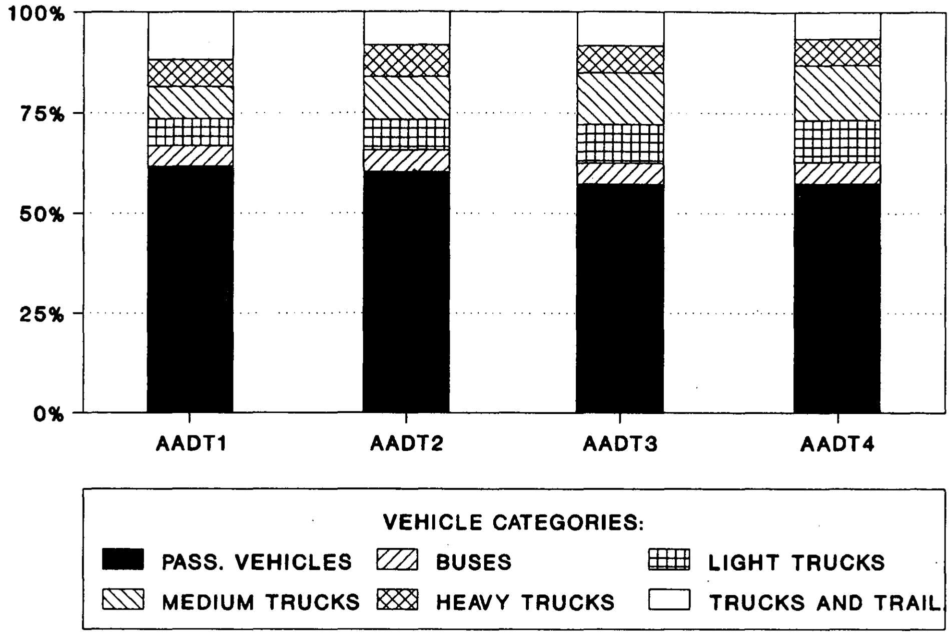

FIGURE 2 Traffic composition depending on traffic volume.

Definitions of Objectives and Criteria

The basic aim is a high-quality network. That means that

α_{1k} is >1.0 and α_{2k}, α_{3k}, and α_{4k} are >0.

The criterion of a network quality from the investor's point of view is the total sum B^i of funds needed to bring the whole network into an excellent condition (backlog) in the ith year.

$$B^i = \sum_{j,k} \alpha^i_{jk} \cdot G_{jk} \cdot L_k \tag{3}$$

where

G_{jk} = the average repair cost to bring 1 km of pavement in class k and in state j into excellent condition,
L_k = the length of roads in class k of the network considered, and
i = current year of the period considered.

The highest network quality is obtained when B^i is equal to 0. The best strategy is the one that reduces B^i to the minimum in the quickest way, with the assumption that the total network length does not change significantly during the period considered. The adopted sum B^i

- Most clearly reflects the effects of the repair strategy on the quality of the network from the investor's point of view,
- Contains the comparative rating of particular road conditions through unit repair costs,
- Uses data in a form typical for the proposed mathematical model, and
- Can be adapted to different evaluation systems.

The primary criterion from the users' point of view is minimal vehicle operating costs. For the whole network considered these costs are

$$S^i = 364 \cdot \sum_{j,k} \overline{\mathrm{AADT}^i_k} \cdot \alpha^i_{jk} \cdot T_{jk} \cdot L_k \tag{4}$$

where

S^i = total users' costs on the whole network in the ith year of the period considered;
T_{jk} = users' costs per vehicle kilometer pondered for average traffic composition in class k of traffic volume ($k = 1, 2, \ldots, m$) on pavement in condition j, where T_{1k} is T_{min} on pavement in excellent condition; and
$\overline{\mathrm{AADT}^i_k}$ = average annual daily traffic on roads of class k in the ith year.

The users' costs were calculated by means of the vehicle operating cost (VOC) (*2*) model, which is a part of the HDM-III program, so that results could be compared with the results of similar methods. The data and standards on vehicles and pavements taken from related studies were used without any statistical verification of their reliability. The quantification and validation of parameters will be done in the final phase, after examining the model behavior under hypothetical conditions. To make the data manipulation easier, the following expression was mostly used:

$$\Delta S^i = 364 \sum_{j,k} \overline{\mathrm{AADT}^i_k} \cdot (T_{jk} - T_{1k}) \cdot \alpha^i_{jk} \cdot L_k \tag{5}$$

which represents the additional users' costs because of the imperfect state of the pavement, that is,

$$S^i = 364 \sum_{jk} \overline{\mathrm{AADT}^i_k} \cdot T_{1k} \cdot L_k + \Delta S^i$$

Accident costs have not been included up to now since the available evidence could not be rapidly adapted to the needs of this model, but a modification of the model could easily be done.

The routine maintenance costs r_k^i are a very important measure of the effects of particular strategies. They are several times smaller than backlog or users' costs, but they have the same structure.

$$r_k^i = L_k \cdot \sum_j \alpha_{jk}^i \cdot r_{jk} \tag{6}$$

where r_{jk} is an average cost of routine maintenance for 1 km of road in class k and in state j. Only additional routine maintenance costs because of an imperfect state of the pavement are a part of the total effects presented here.

Definitions of Strategies

Strategies were defined as guiding rules concerning priorities. The program searched for the best sequence of repairs in each step, that is, year. Because the analysis of effects should give also an answer to the question: Does a consistent application of a certain basic rule (Strategies I to IV) give better results than the sequence of interventions obtained by optimization at each step? the following strategies were defined.

Strategy I (the best first). Resources are spent primarily to repair roads in better condition, and the rest is spent on roads in worse condition.

Strategy II (proportional). Resources are spent in proportion to the length of roads in particular condition categories.

Strategy III (combined). Resources are spent on the part of network whose state is below the minimal standards, and the rest is spent according to Strategy I.

Strategy IV (the worst first). Resources are spent primarily on repairs for the worst sections, and the rest is spent on sections in a better state.

Strategy V (investor's point of view). Resources are spent according to the sequence determined by optimization at every step (year), with the maximal benefit in network quality (backlog) as the primary criterion.

Strategy VI (users' point of view). Resources are spent according to the sequence determined by optimization at every step, applying minimal users' costs as the primary criterion.

FINDINGS

Optimal Sequence of Repair Works

The state of the network before the implementation of particular strategies was described by α_{k0}. The transition matrix of the road network condition in the ith year for the class of roads k was defined by P_{ki}, so the road network state vector after n years of application of some strategy would be

$$\alpha_k^n = \alpha_k^{n-1} \cdot P_{nk} = \alpha_k^0 \cdot P_{1k} \cdot \ldots \cdot P_{ik} \cdot \ldots \cdot P_{nk}$$

For each step the elements a_{jk}^i were calculated from the available budget b_g^i in the ith year

$$b_g^i = \sum_{j,k} a_{jk}^i \cdot \alpha_{jk}^i \cdot G_{jk} \cdot L_k \tag{7}$$

according to the strategy considered. Strategies I to IV have a fixed sequence of work; only the effects backlog B^i, users' costs S^i, and routine maintenance costs were calculated. When the optimization had been performed (Strategies V and VI) the model searched for the sequence of work with minimal backlog or minimal users' costs at every step (year). The optimal sequence was obtained in the following way:

1. *Backlog as an optimization criterion*: The trigger value for the choice of a strategy was found to be

$$r = F(\max a_2) \,/\, F(\max a_3)$$

For $r > 1$ the sequence of repairs is A for good, B for fair, and C for poor roads. For $r < 1$ the sequence of repairs is A for fair, B for good, and C for poor roads.

where

$$F(\max a_2) = (G_{3k}/G_{2k}) - P_{2k}^i\,[(G_{3k}/G_{2k}) - 1]$$
$$F(\max a_3) = (G_{4k}/G_{3k}) - P_{3k}^i\,[(G_{4k}/G_{3k}) - 1]$$
$$F(\max a_4) = 1$$

$G_{j,k}$ = construction costs of bringing 1 km of road in the jth state and the kth category of traffic volume to excellent condition, and

$P_{j,k}^i$ = the probability for a road section to remain in state j in year i.

The fact that

$$F(\max a_2),\ F(\max a_3) > 1, \text{ and } F(\max a_4) = 1$$

means that roads in poor condition must be repaired only if the other two categories have been accomplished. This is the mathematical proof of a logical conclusion that the basic task must be oriented toward stopping the deterioration of better pavements, regardless of whether the available budget is low or high.

2. *The users' costs as an optimization criterion:* The sequence of repairs was determined by the sequence of magnitude of F', so that the sections in a state for which $F'(\max a_j)$ was the greatest had the first priority.

$$F'(\max a_2) = \frac{Q_k^i}{G_{2k}}(T_{4k} - T_{1k})\left(\frac{T_{3k} - T_{1k}}{T_{4k} - T_{1k}} - p_{2k}^i\,\frac{T_{3k} - T_{2k}}{T_{4k} - T_{1k}}\right)$$

$$F'(\max a_3) = \frac{Q_k^i}{G_{3k}}(T_{4k} - T_{1k})\left(1 - p_{3k}^i\,\frac{T_{4k} - T_{1k}}{T_{4k} - T_{1k}}\right)$$

$$F'(\max a_4) = \frac{Q_k^i}{G_{4k}}(T_{4k} - T_{1k})$$

where Q_k^i is (mean AADT · 365) in year i on the roads of the kth class, and, $T_{j,k}$ is VOC per vehicle kilometer for the traffic composition on roads in class k and in state j. These relations show that the road section in any condition could have the first priority, mostly depending on the traffic volume and the ratio of operating costs/construction costs.

Data Used

It was extremely difficult and tiresome to find consistent and reliable data on capital and routine maintenance from our practice. Different data or default values were taken from the available studies and programs with minimal corrections. Thus, the effects presented here can provide only an idea of the relations. For practical use the whole input had to be reconsidered. The sources of data used in this study were as follows:

1. The mean time of service for pavements in particular conditions and the construction costs were adopted from PMF.
2. Maximal possible value for roughness was adopted from HDM-III to make the vehicle operating costs on poor roads as great as possible.
3. The prices for VOC input were average prices in Belgrade in October 1992.
4. Data for three sets of representative vehicles were used in this model. The two sets were suggested by two independent groups of experts (without any statistical background), and one was taken from HDM-III. The differences were evident, but not so important that the strategy had to be changed.
5. Routine maintenance costs were adapted from different sources.
6. The road network in extremely bad condition (Figure 3) was adopted in the example presented to show the logic of the model.

Although data from different sources were aggregated the results were very stable. Little changes could not influence the choice of strategy. Some of the results are presented in the following.

Results

We expected that the best-first strategy would always be the best one from the investor's point of view and that the worst-first would be the best strategy from the users' point of view only on the roads with high traffic volume. For this reason we searched for such a set of data that would give the opposite strategies from two viewpoints. A very high traffic volume was adopted for two-lane roads to obtain the following results. The consequent application of the best-first and worst-first strategies was also considered to see which sequence prevailed in an optimal choice and to see whether the optimization would yield significantly better results than those from the fixed sequence.

Contrary to expectations, the worst-first strategy was never optimal in the frame of the input data. Although there were only minor differences between the effects obtained by the three good strategies, the participation of road length under particular conditions after a 10-year period was quite distinct (Figure 3). These distinctions might influence the final decision only through routine maintenance costs. The magnitude of these costs (Figure 4), users' costs (Figure 5), and backlog (Figure 6) were not of the same level.

The benefits in relation to the do-nothing alternative were usually considered in the evaluation procedures. This relation is shown in Figure 7. The difference between do nothing and the worst-first strategy was almost the same as those between the worst-first and the three good strategies. Such a promising picture was changed after the number of thin layers had been limited.

The PMF model gave the same advantage to the "best-first" strategy, but in that model only backlog existed as a criterion. The choice of strategy from the users' viewpoint depended on the differences $T_{j,k} - T_{j-1,k}$ of users' costs. A large increase in these dif-

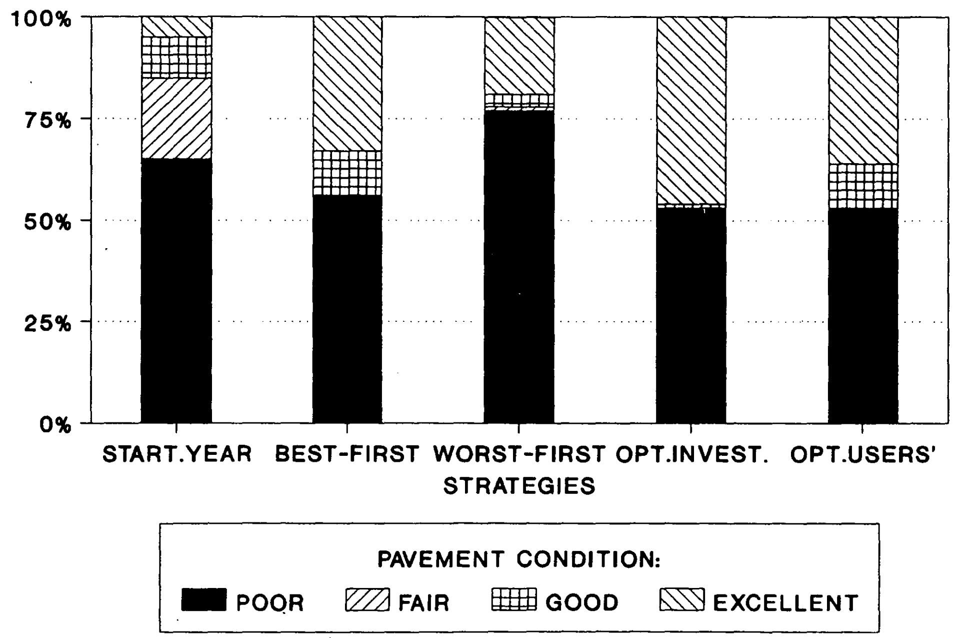

FIGURE 3 Pavement condition before and after 10-year application strategies.

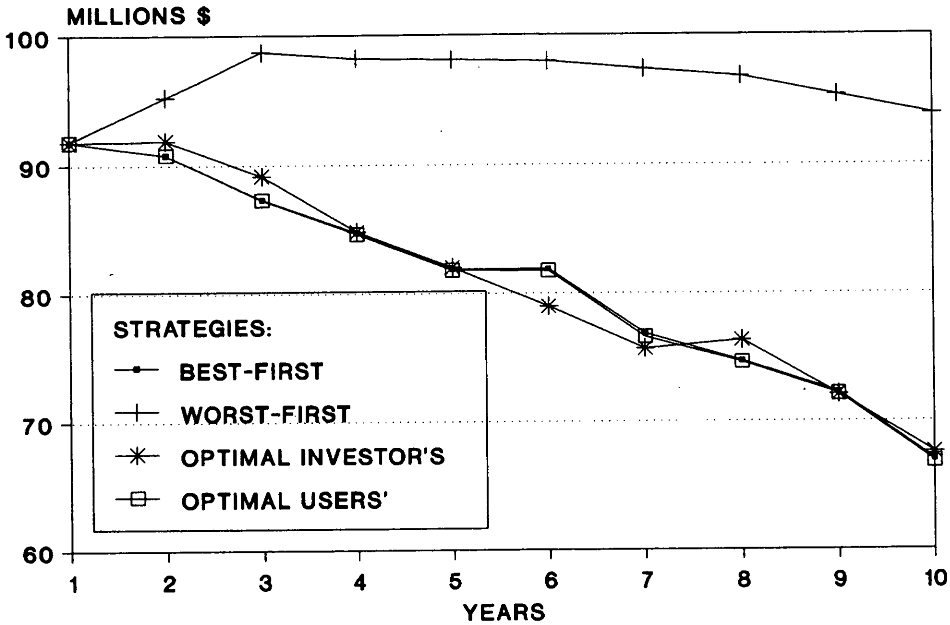

FIGURE 4 Additional users' maintenance costs.

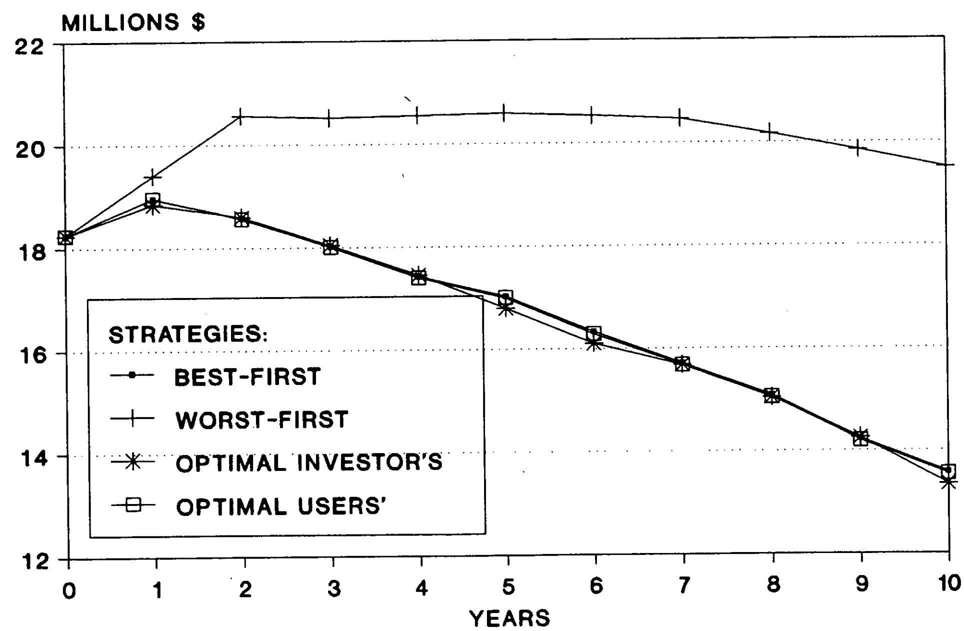

FIGURE 5 Additional users' costs because of pavement deficiencies (ADDT = 12,000 vpd; *L* = 250 km).

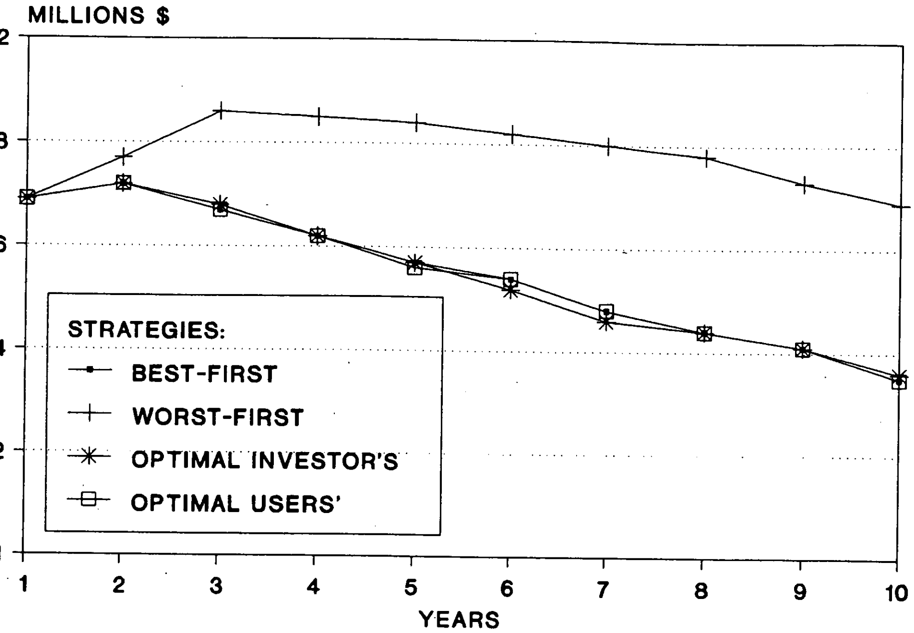

FIGURE 6 Backlog funds needed to bring entire network to excellent state in 1 year (ADDT = 12,000 vpd; L = 250 km).

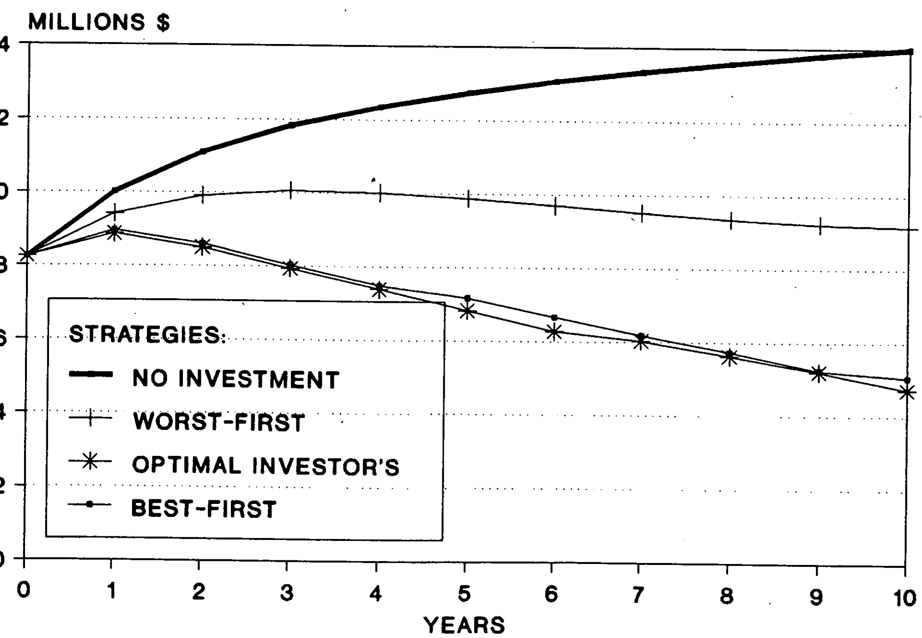

FIGURE 7 Backlog for network with and without investment.

ferences had to be made to change the decision. A detailed investigation of the upper and lower limits of prices in a stable economy is still needed to define more exactly the frames in which particular strategies are competent.

The function of pavement behavior represented here by means of service life in particular states had a great impact on the results. An analysis of these relations under different circumstances is still needed.

In spite of the expected changes in the results in case of some other physical and economic environment, the following conclusions can be drawn:

1. Backlog or some familiar criterion must find its place in the maintenance strategy evaluation for all classes of roads, and
2. Effects of any greater improvement must be observed only on a network as a whole.

INTRODUCTION OF LIMITATION IMPOSED BY PRACTICE

The next step to the real-world situation was the introduction of the principle that after the application two thin layers, that is, after two improvements from a good to an excellent state, the road section must be strengthened. This was performed by introducing several more classes of pavement condition categories, which enabled the network structure to be visible at every step.

The transition matrix had the following shape:

$$\left|\begin{array}{ccccccc}
p^i_{10,k} & & & 1-p^i_{10,k} & & & \\
 & p^i_{11,k} & & & 1-p^i_{11,k} & & \\
 & & p^i_{12,k} & & & 1-p^i_{12,k} & \\
 & a^i_{20,k} & & p^i_{20,k}\cdot(1-a^i_{20,k}) & & (1-p^i_{20,k})\cdot(1-a^i_{20,k}) & \\
 & & a^i_{21,k} & & p^i_{21,k}\cdot(1-a^i_{21,k}) & (1-p^i_{21,k})\cdot(1-a^i_{21,k}) & \\
a^i_{3,k} & & & & & p^i_{3,k}\cdot(1-a^i_{3,k}) & (1-p^i_{3,k})\cdot(1-a^i_{3,k}) \\
a^i_{4,k} & & & & & & 1-a^i_{4,k}
\end{array}\right|$$

The vector of the network state is

$$\alpha^i_k = (\alpha^i_{10,k}, \alpha^i_{11,k}, \alpha^i_{12,k}, \alpha^i_{20,k}, \alpha^i_{21,k}, \alpha^i_{3,k}, \alpha^i_{4,k})$$

where, for the part of network in the kth class of traffic volume in a year i

- $\alpha^i_{10,k}$ = participation of new, strengthened, and reconstructed road sections in an excellent state;
- $\alpha^i_{11,k}$ = participation of new, strengthened, and reconstructed road sections in an excellent state after one treatment with thin layers;
- $\alpha^i_{12,k}$ = participation of new, strengthened, and reconstructed road sections in an excellent state after two treatments with thin layers;
- $\alpha^i_{20,k}$ = participation of new, strengthened, and reconstructed road sections without surface treatment in a good state;
- $\alpha^i_{21,k}$ = participation of new, strengthened, and reconstructed road sections with one thin layer in a good state;
- $\alpha^i_{3,k}$ = participation of roads in a fair state; and
- $\alpha^i_{4,k}$ = participation of roads in a poor state.

To observe the consequences of the introduction of such a matrix the same data were processed in the basic model and in the improved model. For that reason we had to adopt a longer design period to make up the changes. In practice, the starting state vector would reflect the interventions done in the previous period. The results are presented in Table 1 and Figures 8 and 9.

The results are logical and expected. The only surprise is the stationary state in some strategies (Figure 8, years 17 to 20). It means that the process becomes homogeneous after several years. In that case no increase in quality can be expected without an increase in budget resources. It also means that a stable investment in a very poor network leads to a homogeneous process and that conditions for such a development could be defined. The most important is that the worst strategy achieves homogeneity the first.

CONCLUSIONS AND FINAL REMARKS

No significant deviations from the results obtained in the basic alternative (Table 1) were found in the improved alternative of the model. Thus, the worst-first strategy had to be rejected under the circumstances considered in the paper. We also underline two facts.

1. Investor's and users' criteria produced the effects whose functions had the same shape.

TABLE 1 Effects of Particular Strategies in Basic and Improved Simulation Model

STRATEGY	YEAR	BACKLOG (thousands $)	ADDITIONAL USER'S COSTS (thousands $)	ADDITIONAL ROUTINE MAINT.COSTS (thousands $)
Effects of good strategies without limitation for thin layers				
BEST-FIRST	20	10589	51611	957
INVESTOR'S	20	10389	51317	936
USER'S	20	10574	51536	956
Effects of good strategies with the limitation imposed				
BEST-FIRST	16	12542	61957	1130
	17	12557	59225	1141
	18	12793	61551	1169
	19	12360	63141	1121
	20	11933	59550	1076
INVESTOR'S	16	12372	61043	1115
	17	12475	58615	1139
	18	12564	60551	1149
	19	12473	61122	1134
	20	12096	61433	1092
USER'S	16	12525	61876	1129
	17	12542	59144	1141
	18	12775	61485	1168
	19	12354	62976	1120
	20	11927	59610	1076

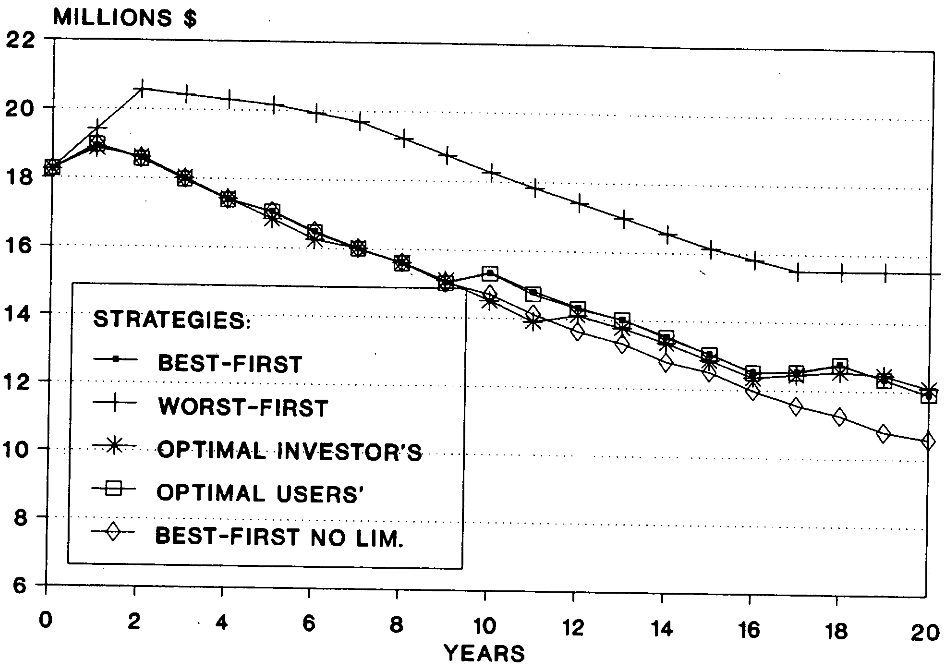

FIGURE 8 Backlog for different strategies with and without limitations of maximum of two thin layers.

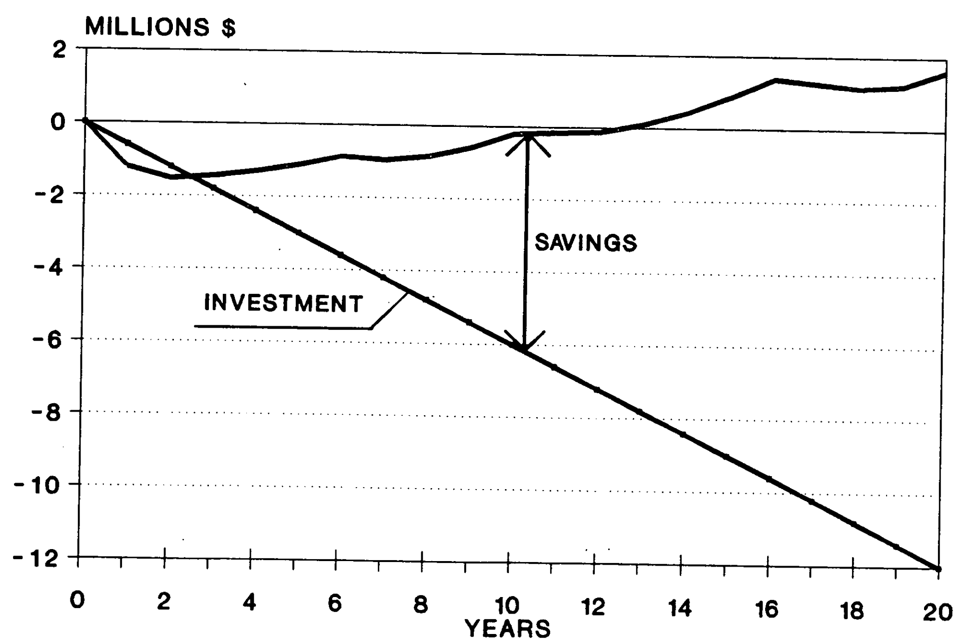

FIGURE 9 Savings (backlog plus routine maintenance) versus investment.

2. Differences between the effects of particular good strategies were under the level of accuracy of the input of this model.

Actually, the consequences of the limitation were reflected in greater oscillations of the effects. For this reason the backlog of the optimal investor's strategy in the 20th year was greater than the backlog of the users' strategy. Cumulative savings could help the final decision, but no further economic analysis has been performed (like internal rate of return or net present value). Such an analysis would be needed for the opposed strategies or when the accuracy of the parameters in the model and the input data had been checked.

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