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Transportation Research Record 1442

Contents

Foreword

The 19 papers in this volume address highway maintenance management, safety, structures, and snow and ice control. These papers should be of interest to engineers, managers, and practitioners involved in maintenance management, pavement maintenance and management, pavement markings, bridge management systems, bridge inspection and deck repair, work zone safety, and winter maintenance activities.

The management section contains four papers, which deal with municipal management systems, a maintenance facility location model, pavement management systems that use geographic information systems, and an economic analysis of pavement preventive maintenance. The safety section contains four papers, which describe speed relationships in work zones, the capacities of roadways with lane closures, the use of glass beads in pavement marking reflectorization, and the use of fly ash in traffic paint. The nine papers in the structures section present information on benefit-cost analysis in bridge management systems and maintenance, rehabilitation, and replacement of bridges; bridge deterioration models; computer-automated bridge inspection; span-based network characterization; the behaviors of reinforced concrete slab bridges; polymer concrete deck overlays; and chemical reactions involving corrosion-inhibited deicing salts. The snow and ice control section contains two papers, which discuss winter maintenance user costs and anti-icing chemicals.

PART 1

Management

Framework for Municipal Maintenance Management Systems

SALEH AL-SWAILMI

Since the 1960s pavement engineers have been working on developing pavement management systems (PMSs). Any PMS includes several major activities such as planning, budgeting, construction, maintenance, and rehabilitation. To date experience has indicated that the system could be successful from a technical standpoint even if it is unsuccessful in the implementation process. Numerous techniques have been proposed and evaluated, but none has proved to be very effective. Agencies vary in their usage of maintenance management systems (MMSs), from not adopting a program to employing sophisticated techniques. The focus is on MMSs for cities. There is a significant difference between urban and rural highway networks. In cities several groups share the streets and the roads. Moreover the provision of an effective communication and coordination system among related groups has a great influence on the success of the MMS. The authorities that should be considered in the MMS include gas, water, telephone, electricity, sewer, and storm authorities as well as traffic departments. A framework for municipal MMSs was developed, with an emphasis on two points: (a) integration of a city's MMS with other related authorities provides better implementation, and (b) communication among related groups has a significant role in the success of any MMS. The conclusion reflects the importance of the communication channels to MMSs and the reliability of using the subsystems technique in integrating the municipality's MMS with other related authorities.

At present there is no universally acceptable definition of a maintenance management system (MMS). However the MMS can be viewed as the technique of optimizing the available resources to accomplish a predetermined minimum level of service by coordinating and controlling applications of planning, budgeting, scheduling, and evaluation. The MMS in broad terms is part of a pavement management system (PMS) that includes the following five major activities: planning, design, construction, maintenance, and pavement rehabilitation. City pavements require maintenance because of the impact from traffic, cuts in the roadway by utilities (utility cuts), and climatic conditions. Well-planned maintenance and rehabilitation slow the rate at which a pavement deteriorates. PMSs have not received the same desired research as the design and construction of new pavements. During the last several decades the focus of pavement engineers has changed from new construction aspects to pavement management development. This shift has occurred because pavements constructed before 1970 are approaching the end of their design lives (*1*). Therefore the use of maintenance and rehabilitation systems has increased significantly among road agencies.

With time maintenance issues have become top priorities in most countries because of the natural process of deterioration of national road networks. Many of these major pavement networks are approaching the end of their design lives, and the budgets for these networks are less than what they were in the past. Moreover continuous construction of many new roads and highways increases the need for MMSs. In many cases road maintenance is so inadequate that even newly constructed roads are deteriorating faster than normally expected (*1*).

Riyadh Municipality, P.O. Box 27108, Riyadh 11417, Saudi Arabia.

City road networks have a greater need for MMSs than rural road networks because city roads are exposed to more deterioration factors. Cities have experienced greater increases in the volume of utility cuts that cause noticeable deterioration to existing asphalt pavements. Because of utility network expansion and the need to maintain existing lines, utility cuts occur even on newly constructed pavements. Patching these trenches has resulted in a noticeable decline in both riding quality and the structural integrity of these pavements. Utility cuts in cities include those for electricity, water, storm, sewage, gas, and telephone systems. Trench depths and widths vary within each utility type as well as among the various utility types (*2*).

STUDY SCOPE AND OBJECTIVES

The MMS provides reliable information that agencies can use to develop a budget and to produce maintenance and rehabilitation schedules. When the MMS is absent the budget and activity schedules are based on on-site inspections. This approach results in subjective decisions that are generally influenced by the previous year's experience.

NCHRP Synthesis of Highway Practice 110 (*3*) has shown that the MMS has been used for pavements for approximately 40 years in the United States and has often been considered a good technique for optimizing resources and planning schedules. The most common problem associated with MMSs is that the MMS experience has indicated that the system can be successful from a technical standpoint but perhaps unsuccessful in terms of implementation. Numerous techniques have been proposed and evaluated, but none has proved to be very effective. Agencies vary in their usage of MMSs, from not adopting a program to employing sophisticated techniques. Between these two extremes some agencies are in the process of developing an MMS. Pavement evaluation is generally determined by some rating procedures, which consider surface distress, roughness, skid resistance, and structural capacity. The purpose of this paper is to develop a framework for a municipal MMS. The following points are suggested to be part of the development of an MMS:

1. Integration of a municipal MMS with other related authorities as subsystems would provide a means for the better implementation of an MMS. The subsystems include traffic departments as well as water, power, gas, and telephone authorities. Utility cuts in pavements account for the greatest amount of pavement distress

in most cities. The PMS is managed by pavement engineers, traffic systems are controlled by traffic engineers, and the MMS is controlled by maintenance engineers. Utility systems are handled by the utilities' technicians and engineers. The integration of these groups as subsystems within the MMS is an essential factor for a successful municipal MMS.

2. The implementation of an effective PMS within any agency requires effective internal communication and cooperation. Communication among related groups has a significant role in the success of any MMS.

GAPS IN CURRENT KNOWLEDGE

The optimization concept of most MMSs needs to be integrated with utility works and management systems that include water, gas, electricity, sewage, storm, and telephone systems. These systems usually include some sort of maintenance management or data base system that can be linked to the master data base of the MMS.

At the network level an optimization methodology needs to be developed by comparing repair alternatives for each project with other project alternatives. This methodology should consider the effects of alternatives on a project and the impacts of possible alternatives on the network.

Sampling systems for pavement evaluation vary in the number of samples per unit length. This technique is effective for rural highways where there are no utility cuts and the serviceability and structural deterioration are distributed uniformly along the pavement. However city or urban roads are exposed to utility cuts that create significant condition variations within the same road. This situation requires a new sampling technique to give a representative road evaluation. The technique considers the deterioration caused by utility cuts and reports related utility information that could help to justify and simplify coordination with utility authorities.

Although most of the MMSs have been developed adequately in terms of technical aspects, there are several problems associated with implementation phases that need to be characterized and resolved to improve the efficiency and effectiveness of the MMSs. Pavement engineers should differentiate between a municipal MMS and a rural MMS by considering the administrative problems. These problems involve the fact that several groups share road works and that there are technical problems associated with utility cuts. Generally the effects of utility cuts on pavement life and pavement restoration standards should be incorporated into the MMS evaluation and prediction models.

In examining the pavement cost impact of utility cut restorations, Emery and Johnston (*4*) found that there were many investigations that indicated the negative impacts of utility cuts. All of the studies in this area indicated that the impact of utility cuts significantly reduces the functional life of the pavement. Anani and Al-Swailmi (*2*) found that there is a significant difference between the deflection readings obtained with a falling weight deflectometer (FWD) meter for patched and unpatched pavements (i.e., 80 and 40 for patched and unpatched pavements, respectively). Figure 1 illustrates the deflection profiles obtained from one street tested in that study. The tests were performed with a constant loading of 4000 kg (9,000 lbf). One profile represents the FWD readings from inside the utility-patched trench, and the other profile represents the unpaved part of the pavement 1 m (3 ft) from the trench edge. The significant increase in deflection readings for the utility-patched pavement verifies that the patched section is significantly weaker than the unpatched part of the pavement. Some other studies have claimed that the utility impact accounts for up to two-thirds of the service life (*4*). Shahine and Crovetti (*5*) have shown that patching of utility cuts has a significant effect on both pavement performance and load-carrying capacity. They have shown that the effect is more severe for pavements that are patched while they are in good to excellent overall condition. Heavy traffic accelerates the deterioration process because of weakened support around the patched section and the pavement edges. Figure 2 (*6*) shows the general deterioration trends of pavements as a result of the impact of traffic, time, and other factors such as utility cuts. Any delay in timely and proper maintenance results in greater costs. Those costs include the repair cost and added user operating costs from low serviceability and interest rates on money.

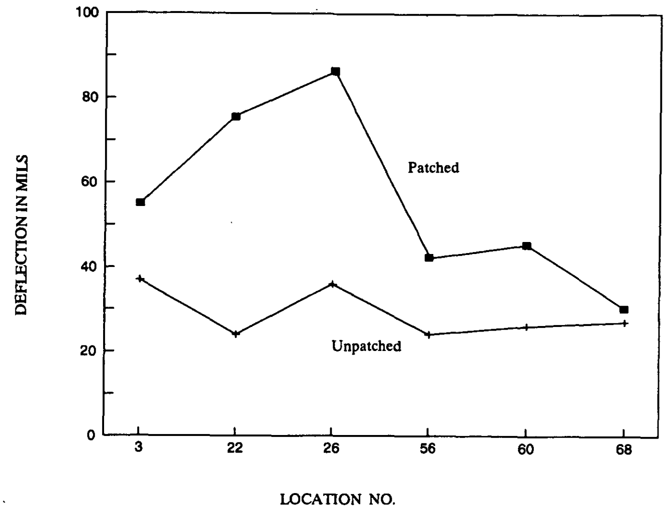

FIGURE 1 Maximum deflection comparison between patched and unpatched pavements (1 mil = 0.001 in.) (*2*).

COMPARISON BETWEEN MUNICIPAL AND RURAL MMSs

There are significant differences between urban and rural roads. Beneath city roads a tremendous number of utility lines run parallel to and cross the roads. The only means of access to construct and maintain these utility lines is to dig up the road pavement. Among the problems associated with utility repairs are the achievement of an adequate backfill compaction and the provision of a smooth finished surface on the asphalt patch. As a result of this situation urban roads experience a significant deterioration rate. Therefore the development of an MMS for urban roads is more complicated than the development of one for rural roads. To develop an effective MMS for a city, all related technical, political, and administrative factors must be considered. Considerations should include the utilities embedded in the pavement layers such as sanitary manholes, inspection chambers, drainage catch basins, and all of the utility lines beneath the pavement layer such as water and storm, telephone, electric power, and gas lines. Figure

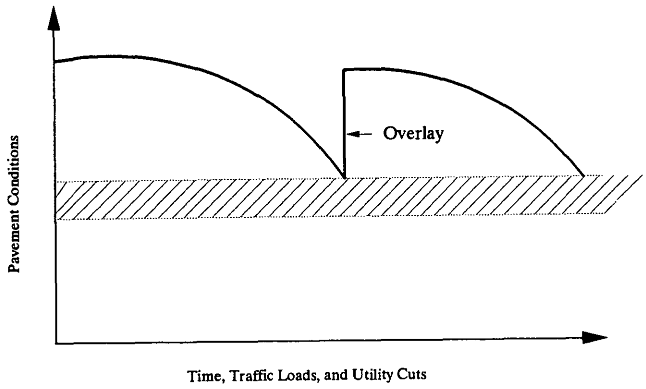

FIGURE 2 Effect of maintenance on pavement performance (*6*).

3 compares typical rural road and municipal road cross sections with utility lines. Each utility line is associated with a unique method of construction in terms of backfill, utility protection, space from adjacent utilities, and depth from the pavement surface. For example a telephone cable should not be closer than a certain distance from a high-voltage electric power cable. Main sewer and storm lines could be as deep as 9 m (30 ft), depending on the topography of the city, because flow is by gravity. Therefore a minimum slope should be maintained to provide an appropriate flow. These differences between urban and rural roads indicate the importance of developing an MMS to meet urban road requirements.

A patch is considered a defect regardless of its performance. A patched area or the area adjacent to the patch usually does not perform as well as the original pavement section. Most utility contractors are specialized in particular utility work but have limited experience in road construction. This results in pavement patches with poor structural and serviceability conditions. Since utility cuts are increasing because of rising utility maintenance, it is necessary to develop a performance impact methodology within the MMS for municipal roads and streets. The effect of asphalt patching on the quality of highways is recognized by the AASHTO *Maintenance Manual* (*7*).

DEVELOPING A NEW SYSTEM

Because of the possible interactions in an urban MMS, it is necessary to develop an MMS with appropriate mechanisms for handling utility maintenance activities. The objective of this technique is to link utility subsystems with the MMS to make an effective MMS by eliminating conflicts associated with utility activities. Therefore the development of an MMS for municipal roads and streets is associated with special conditions that result from the interactions of utility activities. The system should include the

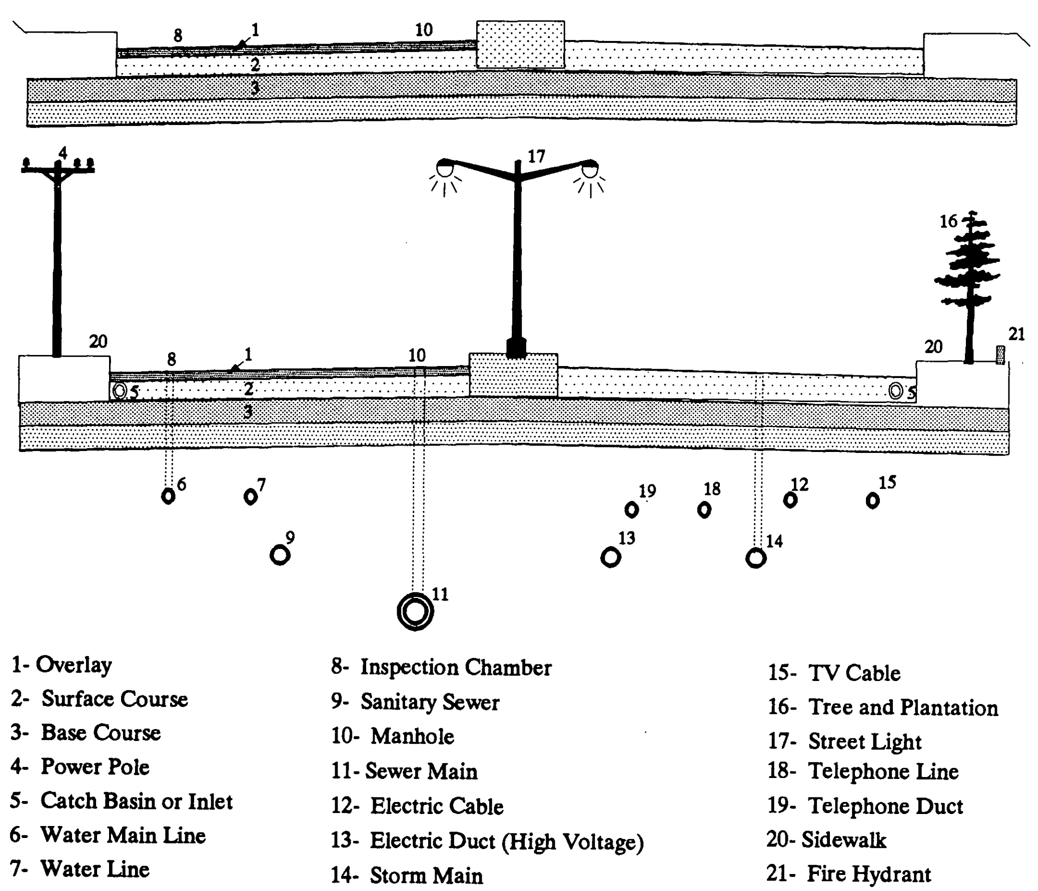

FIGURE 3 Comparison between rural (*top*) and urban (*bottom*) road cross sections.

proper information management system. The data should be sorted according to several categories (i.e., utility type, stress type, severity level, etc.). Occasionally the required data have been gathered and stored previously, but the main problem remains: where and in what format will it be used?

The *NCHRP Synthesis of Highway Practice 135* (*8*) discussed the development and implementation of a PMS. Before an MMS is developed, four preliminary tasks must be accomplished. Figure 4 shows a recommended sequence for those tasks.

INTEGRATING THE CITY'S SUBSYSTEM WITH AN MMS

This section gives a general overview of each external agency, known as a subsystem, that has some activity on the roads and streets of a city. This overview will help to emphasize the role of each subsystem in the MMS. It is always a difficult task to coordinate pavement maintenance activities with utility maintenance activities. Moreover there are coordination problems among the utility agencies as well. Although coordination with these utility agencies is often a very complicated procedure, it is an essential part of maintenance management activities. Therefore if maintenance work is to be accomplished efficiently the coordination problems associated with the external agencies must be overcome. Agencies may organize multiagency committees to provide better coordination for city maintenance activities (*9*). Every city has the major utilities that play a role in the study as subsystems within the city's system. These subsystems include water and storm, telephone, electric power, and gas networks as well as the traffic department. On the basis of city's size, cities vary in how many subsystems are included in the development of an MMS. These subsystems vary in their political organizations. Some of them are departments within the organization of the city, such as water and storm departments. Some of them are public agencies but not within the city organization, such as the department of transportation (DOT). The third type consists of private companies that

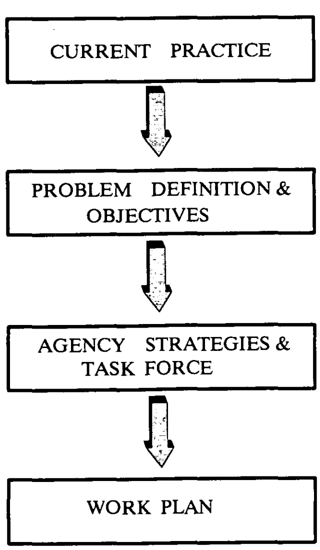

FIGURE 4 Basic tasks toward developing an MMS.

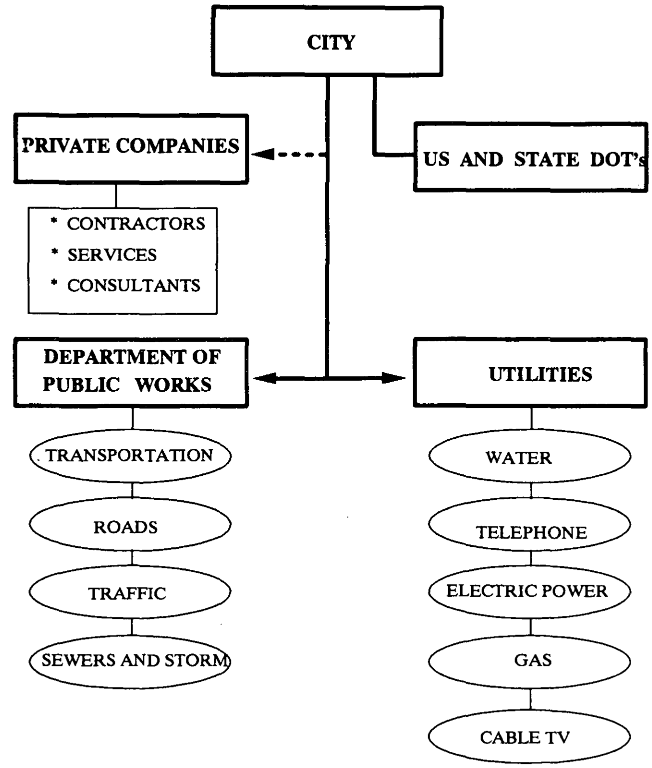

FIGURE 5 Typical organizational relationship for a city's subsystems.

are regulated by either the city or the state, such as electric power and telephone companies. Figure 5 shows a typical organizational structure for the subsystems. The variations among these agencies in terms of political aspects and technical interests make conflicts very likely. A brief description of each subsystem is discussed in the following sections.

Water Network Subsystem

Each street or road contains at least one if not two main and lateral water lines. By looking at the sizes of these water networks and the associated daily activities, one can realize the magnitude of the interaction with road maintenance and rehabilitation activities. Tremendous conflicts can result if this interaction is done with a lack of coordination.

Telephone Network Subsystem

The telephone network covers all streets and roads of the city. In some cities telephone contractors do all the work, including trench backfilling and asphalt patching. In such situations backfilling and asphalt patching are of the lowest quality because of the lack of experience and interest. This results in poor riding quality and continuous failures and settlements. Telephone cable work requires special techniques in terms of cable protection, cable joints, and other technical aspects. Therefore the construction crew is focusing primarily on their technical work with the telephone cable. Instead of hiring a pavement subcontractor for road works, utility contractors prefer to contract out the complete project because this saves them coordination time and markup differences. As a result some telephone contractors are licensed to do road construction in addition to their telephone work. Other telephone contractors have sister companies that are licensed to do road work. To overcome this problem the city should regulate the pavement-related activities done by companies that do telephone construction or maintenance work by requiring preapproval of pavement contractors on the basis of their license and experience. Alternatively the city can bid out road construction and telephone work as two projects, with each bid having its own requirements on the basis of the nature of the work.

Electric Power Network Subsystem

The electric power network is very similar to the telephone network in terms of its complexity. The electric power networks like telephone networks require contracting and construction supervision revisions. This is accomplished by modifying the contracting system and linking electrical construction and maintenance activities with the city's MMS.

Gas Network Subsystem

Although the gas network is not very extensive, it involves continuous activities because of new gas line construction and maintenance of the existing network. To achieve effective coordination and minimize conflicts, these activities must be linked with the city's MMS.

Cable Television Lines

The network of cable television lines contributes the least to conflicts because cable television lines make up the smallest network in the streets. This results in less maintenance activity. Cable television construction and maintenance activities, however, must also be incorporated into the MMS.

Traffic Control Subsystem

Traffic police departments play an important role in municipal MMSs. A number of states have issued written policies and procedures on lane closure and full road closure requirements (*9*). These policies are used to protect against conflicts between the routing of road traffic and maintenance activities. The use of lane closure requests is very effective in providing an organized procedure for pavement and utility companies to accomplish their planned and scheduled activities effectively. The lane closure request form should include information about the date and time of the closure, the name of the street, the duration and type of maintenance activity, and any other necessary information such as the name of the contractor or site engineer. Accident prevention and safety programs for maintenance activities should be considered a part of the lane closure request. The AASHTO *Maintenance Manual* (*7*) provides detailed information about the safety requirements that should be followed during maintenance work.

Sewer and Storm Catch Networks

Sewer and storm catch networks differ from other networks because of their large main lines. Any major maintenance activity with these main lines, such as relocation, replacement, or major repair, requires full closure of the road. In addition one of the major problems associated with sewer and storm catch networks is the difficulty of adjusting pavement service with sewer manholes and storm catch basins. Although these networks are not as big as telephone or water networks, they cover most of a city's road network.

FRAMEWORK FOR NEW MUNICIPAL MMS

As discussed earlier municipal road maintenance activities are combined with several coordination, administrative, and technical problems. These problems have been considered in developing a framework for a municipal MMS. Figure 6 shows the components of the recommended framework for a municipal MMS for urban roads and streets. These components are discussed in the following sections.

Inventory and Pavement Evaluation

An inventory of the roads and their structures should be established. The inventory should include all necessary data related to construction, materials, traffic, structural and serviceability con-

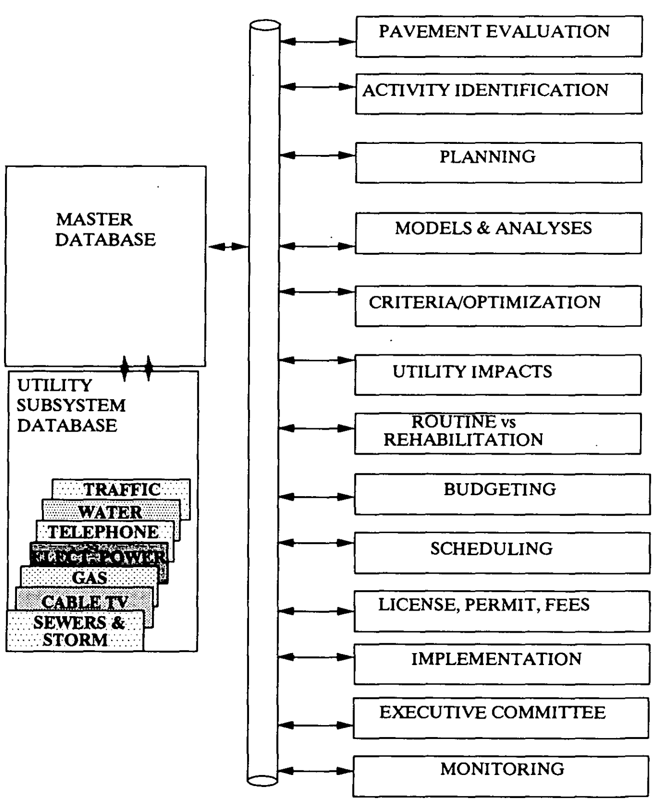

FIGURE 6 Framework for a municipal MMS.

ditions, and utility cuts. For accurate pavement evaluation periodic measurements and site tests of the structural and serviceability conditions of the pavement should be performed at certain distances. Utility cut information should be included in the inventory along with identification of the utility that made the cut.

Identification System (Coding System)

To make data manageable and accessible an identification system should be developed for the pavement evaluation, data collection, analysis, and reporting process. Since the main frame of the system is integrated with each utility agency (as shown in Figure 6) by the subsystem technique, a means of identifying each road distress is required. The identification system should indicate the location, distress severity, serviceability, and cause of the distress (utility cut or environment and traffic). The beginning and end of the section should be clearly defined. Therefore the network should be divided into blocks that are small enough so that they can be used to identify most of the utility repairs by location. At the same time to make data inventorying and information processing activities manageable, the block should not be too small. Figure 7 shows a recommended identification system. The original direction is from south to north and from east to west. Each street is divided longitudinally into stations, with a 15-m (50-ft) station length. Each street is divided into lanes, and each lane is divided into two halves, with right and left sides. This technique divides the paved area into rectangular units of 15 m (50 ft) in length by 1.2 m (4 ft) in width, which is half of the lane width. Parking and emergency lanes, sidewalks, and medians are identified by numbers, and their full widths are used. For each distress resulting from a utility repair the identification should include the utility type and contractor name, as shown in Figure 7. Although this identification technique seems complex, it will provide maintenance engineers with technical justifications for utility repairs and other information necessary for effective coordination. As a result the city will avoid paying for utility repairs that should be conducted by the utility agencies. At the same time the utility contractors will be more concerned about the quality of repaving the utility trenches, because they realize that they will be forced to redo their repairs if the quality of the patched area does not meet the minimum standard specifications. On the other hand different information is not always needed for each block on all roads. This means that the actual number of blocks in the network for which data entry is required is much less than the calculated number obtained by dividing a network area by the standard block area of 19 m^2 (200 ft^2). In other words for roads with similar conditions the computer program would have the capability (as a default setting) to enter their combined evaluations at once. This would be accomplished by entering the start and the end stations of a particular road instead of entering the stations block by block.

Planning

Planning involves an assessment of road performance on a networkwide basis. Generally the level of concern about the network exceeds the level of concern about individual projects. Planning involves the assessment of deficiencies in the network, the ranking of priorities, and the development of a schedule according to

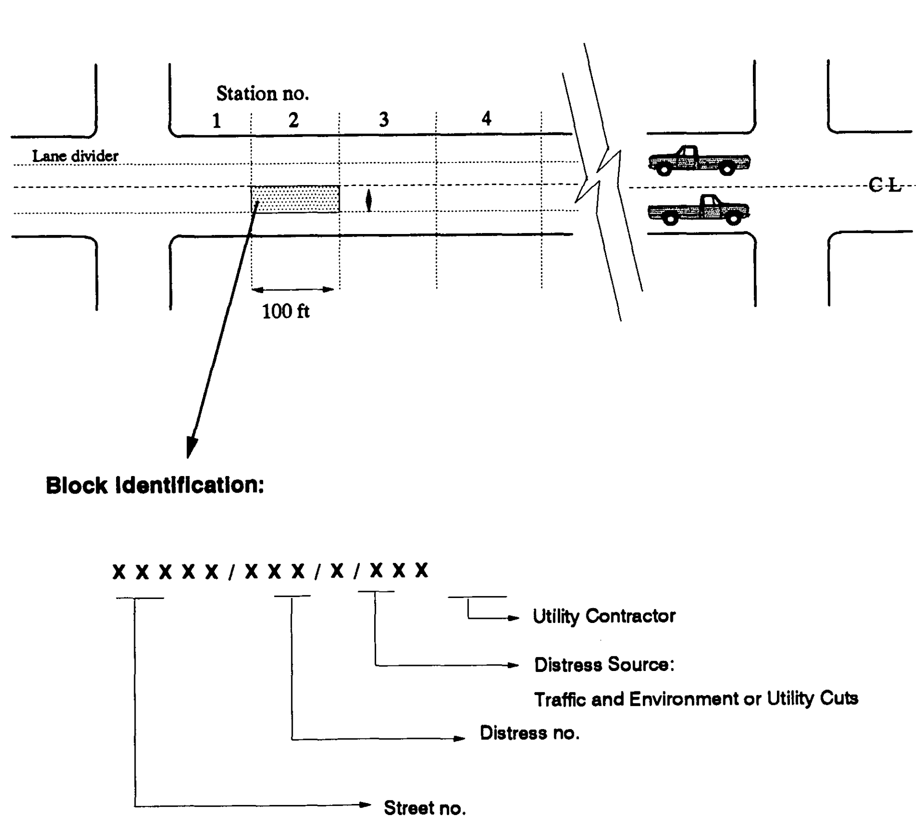

FIGURE 7 Identification system (1 ft = 0.3 m).

available budgets. If utility cuts or traffic loads increase or environmental conditions are extreme, the need for repairs occurs sooner than predicted. This indicates the importance of inspection frequency and interpretation by maintenance engineers.

Models and Analysis

Analyses should include a procedure for analyzing data over time to predict performance. The procedure should include a regression analysis to process repair alternatives and strategies in a format that addresses cost-effectiveness and prioritization in consideration of resource constraints.

Criteria and Optimization

The program establishes preventive maintenance and rehabilitation criteria. Criteria are based on structural and serviceability conditions. The pavement condition index (PCI) is recommended as an evaluation technique. PCI has been selected over other techniques because of its capability of balancing pavement evaluation between pavement serviceability and structural conditions. Criteria for rehabilitation is usually a PCI of between 55 and 70, and preventive maintenance takes place for pavements with PCIs of more than 60. The optimization procedure is based on the most cost-effective alternative.

Since the objective of this paper is to develop a mechanism to link utility agencies with the MMS, criteria and optimization are not discussed.

Impacts of Utilities

Various management systems that have the common objective of identifying the most cost-effective decision are used by utility agencies. Since the agencies share urban roads with municipalities, it is necessary to recognize the contributions (either positive or negative) of those agencies and integrate them into the MMS analysis process. The lack of recognition of the roles of utilities results in costly maintenance activities, plan conflicts, and controversial relationships among the involved agencies. In the absence of an MMS it is common for a utility company to dig a trench in a newly overlaid road because neither the utility nor the road agencies were aware of the others' schedules. The linking of utility agencies as subsystems in the MMS would result in a continuous interchange of information. Such awareness about utility schedules could generate rescheduling of rehabilitation activities to permit utility contractors to finish their repairs. Lack of an MMS results in confusion, contradiction, and poor construction quality.

Maintenance and Rehabilitation Alternatives

The MMS organizes road repairs on the basis of distress type and severity into either routine or rehabilitation repairs. The optimization procedure could change the types of repairs that are made. Because of funding constraints the road agency may be forced to substitute rehabilitation repairs, such as an overlay, by routine maintenance. The impacts of utilities on the MMS could lead to a delay in overlay activities because scheduled utility cuts must be done.

Budgeting

The development of maintenance budgets for a network should be based on the actual needs predicted from the MMS and previous experience. The budgeting process allows agencies to forecast major maintenance and rehabilitation requirements to maintain a particular condition level. The forecasting accuracy is highly dependent on the accuracy of the inputs. The inputs include pavement condition criteria, repair costs, and the expected present worth, which in turn depends on the inflation rate during the analysis period. By changing the rehabilitation criteria the budget forecast changes. Since the budget can be forecast for long-range rehabilitation it can also be estimated for current year maintenance as well.

Scheduling

Scheduling develops actual programs (schedules) for the maintenance and rehabilitation activities. Schedules are developed on the basis of budget constraints and the pavement evaluation process. Data processing models should be capable of predicting when a highway will require maintenance or rehabilitation. A certain level of variation between the actual and the predicted needs should generally be anticipated and adjusted. Maintenance engineers should reduce the effects of those variables that significantly influence long-term planning.

License and Fees

Because several parties are involved in municipal MMSs a license should be issued for each maintenance or rehabilitation activity. The license or permit will centralize the pavement repair process for either utilities or road agencies. At the same time the fees, if any, will be collected through the licensing process.

Implementation

Once the system has been developed to the level that it can be demonstrated, it is necessary to apply it to a pilot network. Such a demonstration project is important for refining the system and exposing any issues that have been overlooked. Through such a demonstration project the development engineers will be able to recognize areas where debugging is necessary or focus on areas for potential improvement. The task force will be more knowledgeable about the practicality of the system and will be receptive to the new format. The main objective of preliminary implementation is to investigate the adaptability of the new system to an actual layout.

At the completion of the trial application and after the final revision the system should be ready for full-scale implementation. The efforts of the technical and administrative teams should be organized to achieve a successful implementation. The implementation phase should be considered a training process for all personnel involved in the system.

Executive Committee

The support of top management is an essential factor for the MMS to remain effective. To maintain that support an executive committee representing all involved agencies should be established. After completing development of the system the task force members could become members of the executive committee. Such a committee would have a low-profile link to the system. The role of this committee is to maintain the communication channels that were established during the development process. This committee will resolve any conflicts between project groups. The committee should receive periodic reports about the activities of the previous period and the plans for the next period. These reports should include the problems associated with the previous period and the technical team's recommended solutions. The frequency of executive committee meetings depends on the size of the city and the necessity of their involvement in resolving conflicts.

Monitoring

Although monitoring is an easy task it is usually overlooked by most road agencies. The MMS is an ongoing development and updating process in terms of data and technology. The MMS team should monitor the efficiency of utilizing the system as it was planned. The monitoring of pavement performance should be a continuous process. This periodic monitoring includes determination of pavement characteristics such as structural capacity and serviceability conditions. Pavements on roads with normal conditions are commonly evaluated every 1 or 2 years. If a road is exposed to major utility cuts an updated evaluation is necessary regardless of the time since the last evaluation. The monitoring process should include an assessment of the reliability of the MMS and an evaluation of any deficiency associated with the system.

Master Data Base

The new MMS recommended here is associated with an increase in data sources because of the inclusion of utility agencies as subsystems within the master data base. With the increasing power of modern computers this will not be a computer processing problem as it was in past years. The cost of computers is rapidly declining. The equivalent hardware capabilities that cost $100 in 1960 would cost $10.74 in 1970 and $0.12 in 1990. At the same time software capabilities have been improved by about 60 percent (*10*). The difficult aspect is how to identify the data and analyze them while retaining the full identification code for each output report. The master data base includes the data along with a coding system that gives the system the ability to view reports for all of the involved agencies and groups. In addition to the typical analysis process the system can identify the responsibility of each utility agency and contractor in the maintenance and rehabilitation budgets. The subsystem technique links the inputs and outputs to the coding system, which results in a data base for each utility.

CONCLUSIONS

1. The development of related methodologic and analytical aspects has received most of the pavement engineers' attention, whereas system implementation requires more research and development.
2. Maintenance work activities on urban roads cause major traffic congestion and rerouting schedules. The nature of maintenance conditions therefore requires extensive planning and scheduling with the other groups that share the roads.
3. Conflicts between road and utility construction and maintenance activities result in a substantial dilution of resources and the diversion of capital from what should be the goal of the MMS. Municipal MMSs recognize the organizational and political problems associated with urban networks. The municipal MMS is a promising technique for protecting urban roads and provides reliable information and justification to show each group its rights and liabilities.
4. A municipal MMS will provide effective and efficient coordination, communication, and working environments instead of the fragmented situation that results when groups work independently.
5. A municipal MMS will provide a general concept of managing maintenance activities with a rational cost-effective approach. Because the groups that share urban roads are greater in number than the groups that share rural roads, this kind of technique has more operational potential for urban roads. This technique will assist all of the groups that share the road network in achieving their goals in a systematic manner.
6. Because of the urban environment, municipal MMSs require greater efforts and a larger budget than MMSs for rural roads.

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From Single-Stage to Two-Stage Facility Location Model of Connecticut's Highway Maintenance System

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The development of two facility location models for use in optimizing the locations of maintenance facilities for the Connecticut Department of Transportation (ConnDOT) is described. In the ConnDOT system vehicles and other equipment are stored at 55 roadway garages and are maintained at 13 repair facilities. In the single-stage model the optimum number of repair facilities is sought without changing the existing configuration of 55 roadway garages. In the two-stage model the optimum numbers of both repair facilities and roadway garages are sought. In both cases the solutions are achieved by using a mixed-integer programing formulation. The objective is to minimize the sum of annual transportation costs plus annualized overhead and expansion costs while maintaining acceptable levels of service. On the basis of the results from the single-stage model and external considerations, two options were recommended to ConnDOT management. The first option closes three repair facilities with an estimated net present value of savings of \$5.0 million. The second option closes two repair facilities and has an estimated net present value of savings of \$3.1 million. In addition to an explanation of how the single-stage model is built on to attain the two-stage model, a detailed description of the data gathering required to achieve this extension is given.

The Connecticut Department of Transportation (ConnDOT) operates 55 garages serving 8900 centerline-km (5,500 centerline-mi) of roadway. The vehicles and other equipment stationed at these garages and at 21 other locations are maintained at 13 repair facilities. This paper describes the development of two related optimization models to represent the maintenance system. The first model considers consolidation of repair facilities without changing the existing configuration of 55 roadway garages. Details of the solutions and the consequent recommendations made on the basis of this single-stage model are given. Then the modifications required to build on the single-stage model to develop the two-stage model are described. In the two-stage model consolidation of the 13 repair facilities and reconfiguration of the 55 roadway garages are both allowed.

SINGLE-STAGE MODEL

Existing System

In 1990 a report issued by a special state commission (*1*) suggested that economies might be realized by a consolidation of ConnDOT's 13 vehicle repair facilities. In response to that suggestion ConnDOT commissioned a study by the Transportation Institute of the University of Connecticut to determine the optimum configuration of vehicle repair facilities. A detailed description of that study and the resulting recommendations have been given by Campbell and Davis (*2*). This section summarizes that work.

The assignment of roadway garages to repair facilities in the existing highway maintenance system is shown in Figure 1. Note that the repair facilities also serve vehicles housed at various locations (shown by triangles) other than the garages, such as vehicles used for electrical repair and bridge repair. All repair facilities are assumed to perform essentially all types of repairs.

A wide variety of equipment is housed at the garages represented in Figure 1. Overall there are approximately 700 heavy trucks, more than 500 light trucks, almost 800 pieces of heavy equipment such as tractors and sweepers, approximately 1,000 pieces of light equipment such as hand mowers and chain saws, and more than 2,000 other pieces of equipment requiring some maintenance.

Single-Stage Formulation

The model used for the present study is a version of what Aikens (*3*) refers to as the "static, capacitated facility location problem." The model's key decision variables relate to whether or not facilities are kept open. Also included are sets of variables that allow for the capacity expansions at open facilities that are necessitated by the reassignment of equipment. In the single-stage mixed-integer programing model shown here, f indexes over repair facilities and i indexes over roadway garages.

Minimize

$$\sum_{f=1}^{13} \sum_{i=1}^{76} C_{if}X_{if} + \sum_{f=1}^{13} (K_fY_f + MA_f + NB_f) \qquad (1)$$

Subject to

$$\sum_{i=1}^{76} D_iX_{if} \leq (P_fY_f + 2RA_fRB_f) \qquad (f = 1 \text{ to } 13) \qquad (2)$$

$$\sum_{f=1}^{13} X_{if} = 1 \qquad (i = 1 \text{ to } 76) \qquad (3)$$

$$A_f \leq Y_f \qquad (f = 1 \text{ to } 13) \qquad (4)$$

$$B_f \leq 8A_f \qquad (f = 1 \text{ to } 13) \qquad (5)$$

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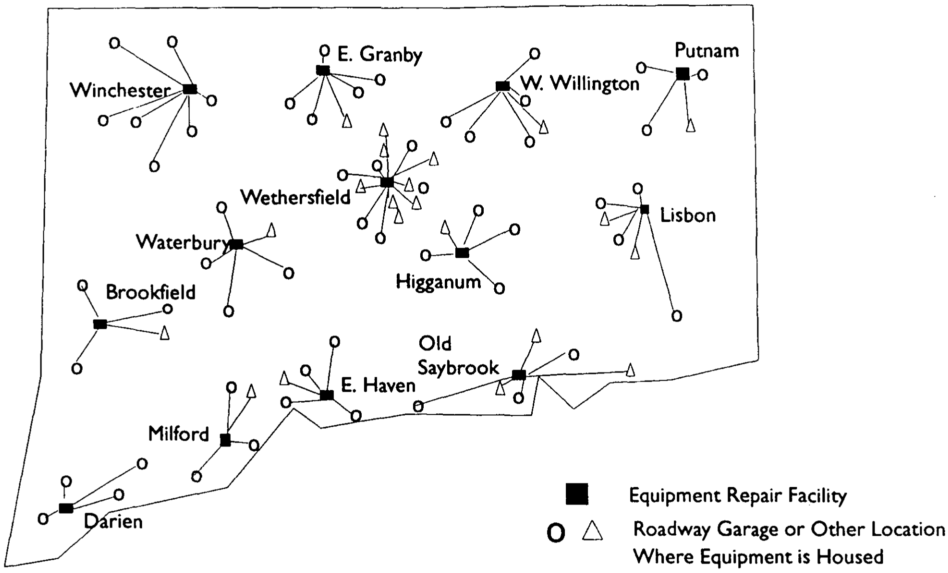

FIGURE 1 Assignment of roadway garages to repair facilities in ConnDOT's existing highway maintenance system.

$$B_f, X_{if} \geq 0 \qquad (i = 1 \text{ to } 76, f = 1 \text{ to } 13) \tag{6}$$

$$Y_f, A_f \in \{0,1\} \qquad (f = 1 \text{ to } 13) \tag{7}$$

where

- Y_f = 1 if facility f is kept open and 0 otherwise;
- A_f = 1 if two work bays are added at facility f and 0 otherwise;
- B_f = number of work bays added, above two, at facility f;
- X_{if} = the proportion of garage i's repair requirements satisfied by facility f;
- C_{if} = total annual cost of transporting all equipment from garage i to facility f for repairs;
- K_f = annualized cost of keeping facility f open;
- M = annualized cost of adding two work bays at a facility;
- N = annualized cost of adding each work bay above two at a facility;
- D_i = total equipment repair requirements at garage i;
- P_f = repair capacity, in terms of equipment serviceable, if facility f is kept open; and
- R = repair capacity added by adding a work bay at a facility.

Objective Function

The objective function, shown in Equation 1, represents a minimization over the sum of annual transportation costs plus annualized overhead and expansion costs. Each overhead cost parameter, K_f, represents the cost of keeping a repair facility open. The one-time overhead costs of the repair facilities include three parts: (a) salvage values of sellable buildings and land, (b) savings from the elimination of planned building renovations, and (c) savings from reductions in inventories of spare parts. The annual components of overhead costs are (a) building expenses and (b) the salaries of support personnel. The one-time overhead costs were put on a common basis with the annual components by assuming a 20-year horizon and an interest rate of 7.125 percent.

Expansion costs also have one-time and annual components. The one-time component is the cost of building new work bays, and the annual part reflects additional building expenses resulting from expansion. When the total costs for expansions of various sizes were estimated, it was found that they could be accurately generalized by a function corresponding to the portion of the objective function that includes the A_f and B_f decision variables.

Transportation costs are the third type of cost included in the model's objective function. The total annual costs incurred if all equipment from roadway garage i is sent to repair facility f for repair, C_{if}, were estimated by using cost functions that consider equipment quantities, travel distances, travel times, and expected numbers of visits per year.

Constraints

Before explaining the problem's constraints, a brief description of the quantification of repair capacities and requirements is in order.

The capacity of a repair facility, in terms of equipment serviceable, is limited primarily by its number of work bays and number of mechanics. Since the total equipment repair requirements do not change when equipment is reassigned to different repair facilities, the systemwide number of mechanics remains fixed and capacities are based strictly on numbers of work bays.

Figure 2 shows the equipment served versus the number of work bays for each active repair facility. There are dozens of different types of equipment within the system, and each has different maintenance requirements. To quantify the service requirements, six equipment categories were defined on the basis of similarities of maintenance requirements. Each "equipment served" value shown in Figure 2 represents the weighted average quantity of equipment served by a repair facility.

From Figure 2 it is apparent that excess capacity exists at some repair facilities. By choosing as benchmarks the repair facilities that serve large amounts of equipment for their numbers of bays,

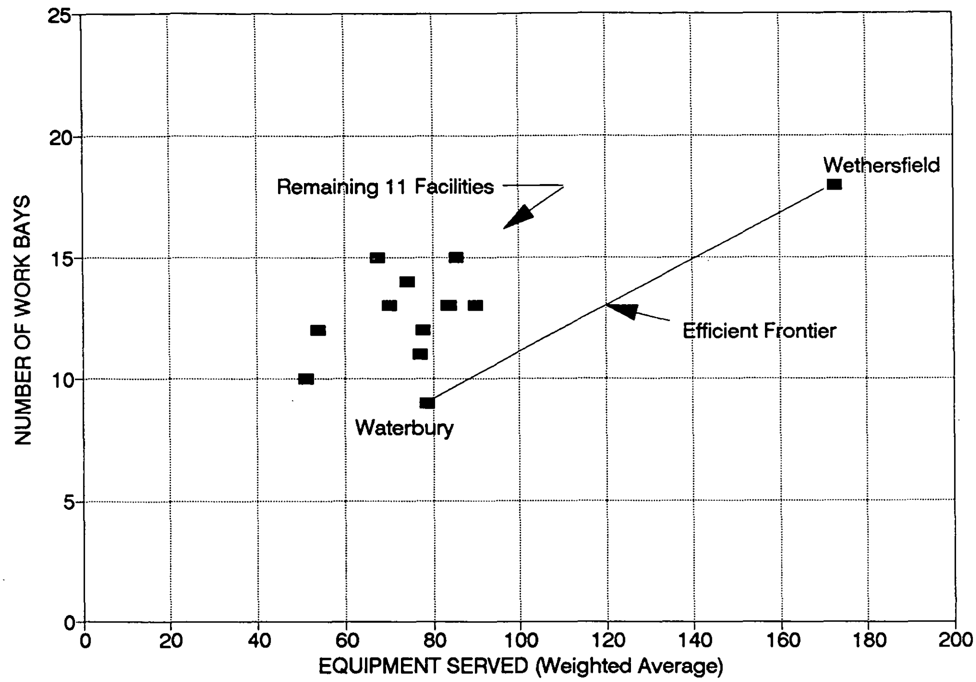

FIGURE 2 Equipment served versus number of work bays for each active repair facility.

an efficient frontier was established. For each repair facility not on the efficient frontier, the frontier was used to find the value of "equipment serviceable" associated with the number of work bays at the facility, as illustrated in Figure 3.

Constraint Set 2 (Equation 2) states that for each repair facility the total repair requirements served cannot exceed the available capacity in terms of equipment serviceable. Constraint Set 3 specifies that all repair requirements must be satisfied for each roadway garage.

Constraint Sets 4 and 5 pertain to expansions at repair facilities. According to Constraint Set 4 an expansion of two work bays cannot be done unless the repair facility is kept open. Constraint Set 5 ensures that an expansion beyond two work bays cannot be done unless the initial two bays are added, and it also limits the number of bays added above two to a maximum of eight.

Constraint Sets 6 and 7 are standard nonnegativity and binary variable constraints, respectively.

Single-Stage Results

The optimal solution of the single-stage problem called for elimination of 6 of 13 repair facilities, with a net present value of savings of $7.1 million. Such a dramatic change from the existing system was unexpected, particularly since many of the model's underlying assumptions are conservative and favor the status quo.

In practical terms the closing of almost half of the repair facilities would cause considerable disruption. Therefore a set of runs to investigate the savings offered by solutions with more than seven repair facilities was designed by adding a constraint to the model that specified an allowable number of repair facilities. The problem was then resolved with the allowable number of repair facilities fixed at levels ranging from 8 to 13. The results of these runs are shown in Figure 4.

External Considerations

Consideration of factors outside the context of the model was important because the model could not capture all aspects of the real problem. The model's objective function, with its net present value criterion, reflects an emphasis on minimizing costs. This was practical, given the impetus for the study, but it must be recognized that other criteria are important as well. Ease of implementation, the levels of service provided, and robustness in the face of uncertainties are all factors not reflected in the model's objective function. Nevertheless such factors had to be considered as recommendations were being developed.

Single-Stage Recommendations

On the basis of the results from the single-stage model and external considerations, two options were recommended to ConnDOT. The first option consisted of closing three repair facilities for an estimated savings of $5.0 million. Note from Figure 4 that this solution captures about 70 percent of the savings offered by the seven-facility solution, although only half as many repair facilities would be closed. In addition each of the closed facilities could be sold, because there are no other major operations on those sites. This is not true for the other repair facilities closed in solutions with fewer than 10 facilities. However this option has two disadvantages. First it calls for closing East Haven, which is one of the finest facilities in the system. Second it calls for expansion at Wethersfield, which would be extremely difficult because of its suburban location. The second option retains East Haven and calls for no expansion at Wethersfield, but saves only $3.1 million.

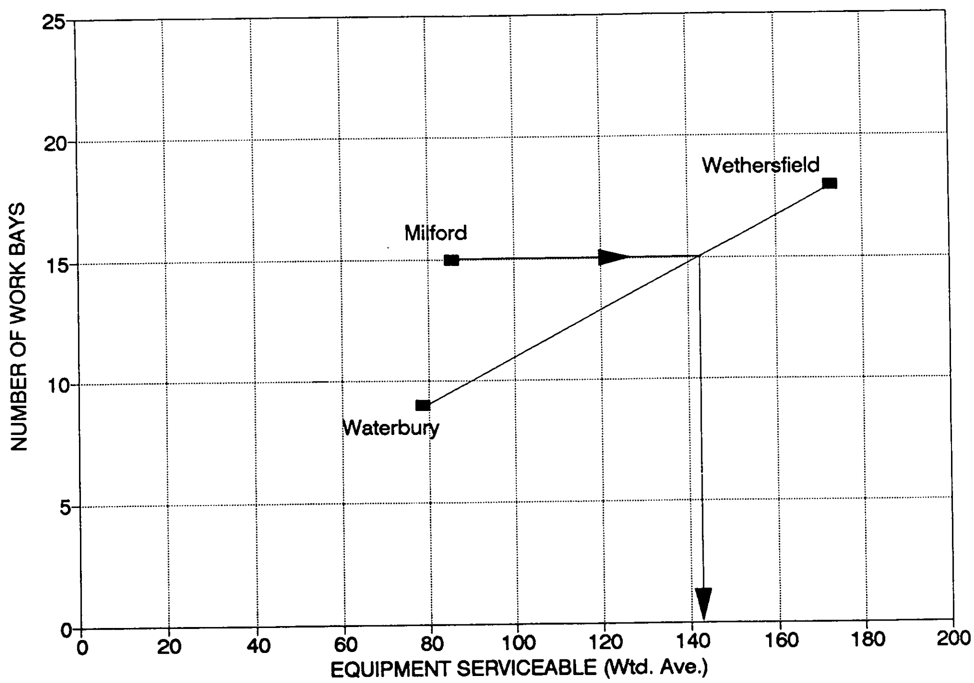

FIGURE 3 Use of "efficient frontier" to determine repair facility capacity.

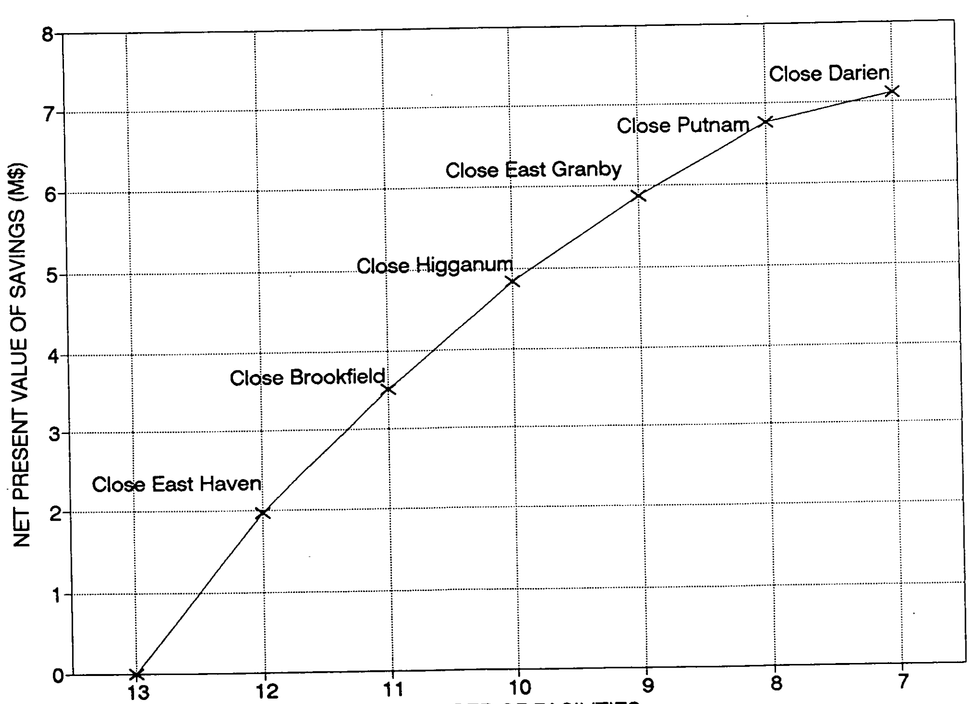

FIGURE 4 Savings offered by solutions with seven or more repair facilities.

Implementation

From the outset great care was taken to involve ConnDOT management in the investigation. Their assistance was vital in a number of areas, including cost and parameter estimation and input regarding issues external to the mathematical model. Nevertheless at the beginning of the single-stage study ConnDOT was in a position of having to react to the initial commission recommendations within a limited time frame. At the conclusion of the single-stage study it was obvious that a study that is more proactive and that at the same time recognizes the potential for consolidating roadway garages would have been more desirable. Therefore rather than proceeding directly to implementing the results of the single-stage study a second study was undertaken to examine the two-stage problem.

TWO-STAGE MODEL

The two-stage facility location model builds on the single-stage model to allow for consolidation of the roadway garages as well as the equipment repair facilities. In this section the formulation of the two-stage model is described.

The two stages in the newly developed model correspond to the two types of facilities mentioned above—that is, equipment repair facilities and roadway maintenance garages. As before all repair facilities are assumed to have the same repair capabilities in terms of the types of repairs that they perform. Similarly all roadway garages are assumed to provide the same types of roadway maintenance.

Besides the two stages represented by the repair facilities and roadway garages, a third level exists within the maintenance system, namely the roadways that require state-provided maintenance. Predefined snow removal "runs," which number about 300, were adopted as the basic roadway sectors for the study.

Previous Work

The prior study most relevant to the location of roadway garages is that of Bell and Rainer (*4*). That study dealt with the location of roadway garages for the state of Alabama. Although they used a single-stage model, their methods are relevant to the current study because they modeled garages serving roadway sectors, which is the key new feature of the two-stage optimization model.

A description of prior research that has been done to extend single-stage facility location models to represent multiple stages is included in the survey by Aikens (*3*). In that survey two multistage facility location formulations are presented for systems without capacity constraints. The formulation by Kaufman et al. (*5*) is based on triple-subscripted variables (x_{ijk}) that correspond to the amount of demand in zone k served by plant i through warehouse j. The formulation by Tcha and Lee (*6*) extends the formulation given by Kaufman et al. (*5*) to allow for any number of stages in the system. The emphasis of both of these multistage studies is on efficient solution procedures, not necessarily on modeling any specific application.

The formulation described below takes certain elements of its structure from previous multistage models, but it also adds some key new features of its own. The way that it simultaneously allows for facility closings, capacity expansions, garage and sector reassignments, and equipment transfers takes it a step beyond previously developed formulations.

Two-Stage Formulation

The two-stage mathematical programing model of ConnDOT's maintenance facility location problem is shown below. As in the single-stage formulation, f indexes over repair facilities and i indexes over roadway garages. The j subscript, which is new in this formulation, indexes over roadway sectors.

Minimize

$$\sum_{f=1}^{13} (K_f Y_f + MA_f + NB_f) + \sum_{i=1}^{55} (L_i YG_i + MG\,AG_i + NG\,BG_i) + \sum_{f=1}^{13}\sum_{i=1}^{76} C_{fi} X_{fi} + \sum_{f=1}^{13}\sum_{i=1}^{55} U_{fi}(T_{fi} - TP_{fi}) + \sum_{i=1}^{55}\sum_{j=1}^{300} TCOST_{ij} XS_{ij} \tag{8}$$

Subject to

$$\sum_{i=1}^{76} D_i X_{fi} + \sum_{i=1}^{55} (T_{fi} - TP_{fi}) \le (P_f Y_f + 2RA_f + RB_f) \quad (f = 1 \text{ to } 13) \tag{9}$$

$$\sum_{j=1}^{300} DS_j XS_j \le \left[D_i + \sum_{f=1}^{13} (T_{fi} - TP_{fi}) \right] \quad (i = 1 \text{ to } 55) \tag{10}$$

$$\sum_{f=1}^{13} T_{fi} \le (2RG\,AG_i + RG\,BG_i + XSPACE_i\,YG_i) \quad (i = 1 \text{ to } 55) \tag{11}$$

$$\sum_{f=1}^{13} X_{fi} = 1 \quad (i = 1 \text{ to } 76) \tag{3}$$

$$\sum_{i=1}^{55} XS_{ij} = 1 \quad (j = 1 \text{ to } 300) \tag{12}$$

$$\sum_{j=1}^{300} XS_{ij} \le 20YG_i \quad (i = 1 \text{ to } 55) \tag{13}$$

$$A_f \le Y_f \quad (f = 1 \text{ to } 13) \tag{4}$$

$$B_f \le 8A_f \quad (f = 1 \text{ to } 13) \tag{5}$$

$$AG_i \le YG_i \quad (i = 1 \text{ to } 55) \tag{14}$$

$$BG_i \le 8AG_i \quad (i = 1 \text{ to } 55) \tag{15}$$

$$TP_{fi} \leq D_i X_{fi} \quad (i = 1 \text{ to } 55, f = 1 \text{ to } 13) \tag{16}$$

$$\sum_{f=1}^{13} \sum_{i=1}^{55} T_{fi} = \sum_{f=1}^{13} \sum_{i=1}^{55} TP_{fi} \tag{17}$$

$$T_{fi}, TP_{fi}, BG_i, B_f, X_{fi}, XS_{ij} \geq 0 \tag{18}$$

$$YG_i, Y_f, A_f, AG_i, \in (0, 1) \tag{19}$$

where

YG_i = if roadway garage i is kept open and 0 otherwise;
AG_i = 1 if two storage bays are added at roadway garage i and 0 otherwise;
BG_i = number of storage bays added, above two, at roadway garage i;
XS_{ij} = the proportion of roadway section j's maintenance requirements satisfied by roadway garage i;
T_{fi} = additional equipment going from roadway garage i to repair facility f because of expansion at roadway garage i (in terms of weighted units of equipment added);
TP_{fi} = reduction in equipment going from roadway garage i to repair facility f because of downsizing at garage i;
U_{fi} = total annual cost of transporting a unit of equipment from roadway garage i to repair facility f for repairs (this equals C_{fi} divided by D_i);
$TCOST_{ij}$ = total annual cost of servicing of all of sector j requirements from roadway garage i;
L_i = annualized cost of keeping roadway garage i open;
MG = annualized cost of adding two bays at a roadway garage;
NG = annualized cost of adding each bay above two at a roadway garage;
DS_j = demand in sector j in terms of the equipment required to service that sector;
RG = capacity added by adding a bay a roadway garage; and
$XSPACE_i$ = extra space available at roadway garage i.

Objective Function

The objective function (Equation 8) represents a minimization of all relevant costs. The cost components of the objective function are of three types: (a) the costs of keeping the repair facilities and roadway garages open, (b) the costs of expanding repair facilities and roadway garages, and (c) the costs of providing service. The cost components added beyond the single-stage model are elaborated upon below in the discussion of the input data requirements for the two-stage model.

Constraints

Supply constraints ensure that the total service being provided from a location does not exceed that location's capacity. Constraint Set 9 considers the capacity of repair facilities. The left-hand side of the inequality is the total equipment being serviced by a repair facility, and the right-hand side is the repair facility's capacity on the basis of its existing number of work bays and the numbers of work bays, if any, that are being added. Constraint Set 10 ensures that the service provided by each roadway garage is less than or equal to the capacity of the equipment at that garage, including any equipment that is being added to (or subtracted from) the garage. Constraint Set 11 ensures that the amount of equipment being added to a roadway garage does not exceed the garage's supply of available space.

Demand constraints are included in the model to ensure that all service requirements are met. Constraint Set 3, which is the same as that in the single-stage model, handles the repair of equipment from the roadway garages, and Constraint Set 12 handles roadway maintenance for all sectors. Constraint Set 13 states that a sector must be served by a roadway garage that is open. In that constraint set a value of 20 represents the maximum number of sectors that can be assigned to a single garage. This value was chosen because it appears to be large enough to be nonrestrictive for any realistic solution.

Constraint Sets 4, 5, 14, and 15 enforce logical relationships among expansion variables. As in the single-stage formulation, Constraint Set 4 states that a repair facility cannot be expanded unless it is kept open, and Constraint Set 5 ensures that expansions by more than two bays cannot happen unless the initial two-bay expansion occurs. Constraint Sets 14 and 15 represent similar constraints for the roadway garages.

Constraint Set 16 is included in the model to ensure that the amount of equipment being subtracted from a repair assignment does not exceed the amount that was assigned. Constraint Set 17 balances the additions and subtractions associated with reassigned equipment.

The last two constraints in the model, Constraint Sets 18 and 19, are nonnegativity and binary variable constraints, respectively.

Obtaining Required Input Data

The two-stage model presented above was run successfully by using artificial input data. In addition many of the actual data required for the two-stage model were collected for the single-stage model. On the basis of those data 8 of the 15 parameter types required to build the two-stage model have been estimated: C_{fi}, D_i, U_{fi}, K_f, M, N, P_f, and R. The estimation of the remaining seven parameter types is discussed below.

Demand in Sector *j* Demand in Sector j, DS_j, is expressed in terms of the equipment required to serve that sector. It can be calculated as follows:

$$\mathrm{DS}_j = \frac{D_i SR_j}{\sum_{j \in \{Si\}} SR_j} \tag{20}$$

where

D_i = total equipment at roadway garage i;
SR_j = amount of snow removal equipment required for sector j;
$\sum_{j \in \{Si\}} SR_j$ = total amount of snow removal equipment for all sectors assigned to roadway garage i in existing system; and

$\{S_i\}$ = set of sectors served by roadway garage i in existing system.

As mentioned earlier, D_i values were already estimated for the previous study. Data pertaining to the SR_j and $\{S_i\}$ values are readily available on the basis of information contained in the "snow books" produced by ConnDOT's Office of Highway Operations. Therefore, estimation of DS_j values should present little difficulty.

Total Cost of Servicing All Sector *j* Requirements from Garage *i* The total cost of servicing all sector j requirements from garage i, $TCOST_{ij}$, is the largest set of parameters in the model. There are 55 existing roadway garages and hundreds of roadway sectors, but fortunately an estimate is not required for all possible garage-sector pairs. On the basis of the rules of thumb provided by the Office of Highway Operations, a set of candidate roadway garages will be defined for each roadway sector, so that $TCOST_{ij}$ values will need to be estimated only for garage-sector pairs that correspond to such sets.

Another piece of the transportation cost function will be the number of trips made between a roadway sector and a roadway garage. On the basis of discussions regarding this aspect of the cost function, it was concluded that estimates can be provided by those most familiar with roadway maintenance operations. However these estimates will be subject to some degree of error. It will therefore be important for the second phase of the study to include a sensitivity analysis that investigates the effects of errors in the $TCOST_{ij}$ estimates.

Annualized Cost of Keeping a Roadway Garage Open The annualized cost of keeping a roadway garage open, L_i, will be estimated for 25 to 40 roadway garages that will be candidates for closing. These candidates will represent those roadway garages that have not been identified as "untouchable" by ConnDOT management. There are three major components to the L_i values: (a) salvage values of land and buildings, (b) savings in building expenses such as for heat and electricity, and (c) savings in salaries for support personnel.

Estimation of the salvage values of land and buildings for the roadway garages is a complicated matter. Three estimation alternatives have been identified: (a) field appraisal of each site, (b) estimates made on the basis of accounting records, or (c) not including property salvage values in the L_i estimates. The third alternative offers advantages in that it is the most conservative, requires the least amount of data gathering, and is consistent with the methods used in the study by Bell and Rainer (*4*).

Regarding the second major component of the L_i values, detailed building expense records for roadway garages are readily obtainable. For the third major component the Office of Highway Operations will be consulted regarding which positions could be eliminated if a roadway garage were closed and what the estimated cost savings would be.

Capacity-Related Parameters Capacity-related parameters include RG, the capacity added by adding a bay at a roadway garage; and $XSPACE_i$, the extra space available at garage i. Although the estimation of values for these parameters may appear to be straightforward, it requires clarification of certain assumptions related to policy, such as whether or not each major piece of equipment requires its own storage bay.

Annualized Costs of Adding Bays at a Roadway Garage The annualized costs of adding bays at a roadway garage are represented by MG and NG. Because the Office of Highway Operations has cost estimates for expansions of various sizes, the development of estimates for these parameters should be straightforward. The model allows for a maximum of 10 bays added at each roadway garage. This simple type of constraint may be appropriate for a first-pass analysis, but it would be better if the model could be based on realistic site-specific limitations on expansions. The trade-off, of course, is that a significant amount of work is required to estimate the potential for expansion at 55 garage sites.

Use of Two-Stage Model

Once the data required to complete the two-stage model have been obtained the model will be used to develop specific recommendations regarding the following five types of decisions:

1. Which repair facilities to keep open and which to expand,
2. Assignments of equipment to repair facilities,
3. Which roadway garages to keep open and which to expand,
4. Assignments of roadway sectors to roadway garages, and
5. Assignments of equipment to roadway garages.

Given the close relationship between the two models, the results of the two-stage model are expected to be consistent with those of the single-stage model. However the recommendations made on the basis of the two-stage model will be more comprehensive. These recommendations are likely to include cost-saving measures related to the consolidation of the roadway garages that could not be identified by using the single-stage model.

SUMMARY AND CONCLUSION

Two related optimization models representing Connecticut's system of roadway maintenance and vehicle repair facilities have been described. The single-stage model was developed to investigate the possible benefits that would result from consolidation of vehicle repair facilities. On the basis of that model two options were recommended, with estimated savings of \$5.0 million and \$3.1 million, respectively. The extension of the model to a two-stage formulation has also been described, and the nature of the additional data required to estimate the parameters for the two-stage model has been presented. Considering the results of the single-stage study, the follow-up study done on the basis of the two-stage model is expected to result in significant savings for the state of Connecticut.

ACKNOWLEDGMENTS

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Geographic Information Systems as Platform for Highway Pavement Management Systems

OMAR OSMAN AND YOSHITSUGU HAYASHI

Although existing highway pavement management systems (HPMSs) have so far served their intended specific purposes, they still have limitations. Many of these limitations come from the rigid mode of use of these systems, inefficient coordination between subelements of the system, and the lack of relevant information. Adaptation of geographic information systems (GISs) into this area, however, can help tackle many of these limitations. This is particularly true because of the realistic representation of real-world entities, the organized data structure, and the powerful analysis and presentation capabilities provided by GISs. A prototype HPMS is coupled with a GIS containing a land base of a selected study region. The resulting system is composed of (a) a spatial data base, (b) an attribute data base, (c) a general and a specific analysis module, and (d) an output generation module. Possible applications of such a system are then investigated. Several outputs representing different analysis stages and techniques mainly related to maintenance decision making are also presented. It is shown how this system provides a flexible tool for interactive analysis of policy tradeoffs, clear display of results, and coordination between related activities. The system can also improve data availability by spatially relating relevant information and facilitating data exchange between different administrative offices.

Highway management is a process, with several highway-related activities involving planning, design, construction, operation, maintenance, and research developments. Each of these activities requires frequent decision making to tackle various problems that are neither well structured nor unambiguous. Such problems are not unique, so a one-shot effort would be justified; neither do they recur frequently enough with sufficient similarity to subject them to rigid mathematical treatment. Moreover because of the mixture of uncertainty in the scientific aspects of these problems and the subjective and judgmental elements in their sociopolitical aspects, there is no wholly objective way of finding a best solution (*1,2*). Thus to effectively carry out the management process some sort of a decision support system (DSS) is essential (*1,3*). The functions of a DSS range from information retrieval and display, filtering and pattern recognition, extrapolation, inference, and logical comparison to complex modeling. Unlike information systems that are based on a sequential structure of analysis and decision support resulting in unique answers or at most scenario analysis, DSSs emphasize the importance of interactiveness and the direct involvement of the end user. This implies feedback between the different elements of the system. Under this definition many of the existing highway management systems would fall under the information system category (*2*). The use of DSSs can, however, vastly improve the decision-making quality and can move management toward achieving better use of limited highway resources.

One technology that is particularly promising for the highway management process is geographic information systems (GISs). These systems are gaining increasing importance and widespread acceptance as tools for decision support. GISs can assist in the preparation, analysis, display, and management of geographical data. It is in the analysis and display functions that GISs meet DSSs.

In this paper the applicability of GISs to the highway management process is discussed. A prototype GIS-based highway pavement management system (HPMS-GIS) was developed, with an emphasis placed on maintenance-related issues. This system couples highway performance evaluation and repair prediction models with GIS analysis and display capabilities. The results of a case study performed with this system are also presented to clarify some of the potential applications and advantages of implementing such a system.

WHY GIS FOR HIGHWAY MANAGEMENT?

Road networks are inherently geographic because they extend over wide areas and interact with various land topographies such as rivers, mountains, buildings, and other roads. Network components and events are also locational in nature. For example the extent and shape of a link, road intersections, accidents, and pavement condition cannot be completely defined unless the geographic location of the component or event is given. Thus spatial considerations in the analysis of different road activities are essential and can vastly improve the quality of the decision-making process (*4–8*). Analyses of such spatial considerations are difficult, inaccurate, and time-consuming without a GIS.

However highway management systems are usually based on a central data bank in which only descriptive data are handled. Moreover in most of these systems the data are not referenced to any geographic coordinate system, and thus diverse data types cannot be related in most cases. More advanced systems are also supported by computer-aided design and drafting (CADD) systems for generating maps. Neither of the two systems permits spatial operations on the data. GIS as a system with spatial analysis capabilities, besides having the attributes of the abovementioned systems, particularly matches the geographic nature of road networks.

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DEVELOPMENT OF PROTOTYPE HPMS-GIS

The application of an HPMS-GIS described here is not intended as a complete system but rather as an investigation of how to apply a system and the advantages of applying GIS to the highway management area. However the development procedure and elements of the described prototype system are equivalent to those required for a complete system.

The system described here was developed within the ARC/INFO environment (*9*) installed on a UNIX-based engineering workstation. This system integrates geographic analysis and modeling capabilities with a fully interactive system for the acquisition, management, and display of spatial and attribute data.

The Aichi Region in central Japan was selected as the geographic area of the prototype system. The city of Nagoya is located within this region and was selected for detailed representation in the system. An HPMS-GIS was then developed for this region. The system that was developed consists of several modules that interact with each other to carry out the required analyses and presentations. These modules are (Figure 1) (a) the data module for the spatial data base, (b) the data module for the attribute data base, (c) an analysis module, and (d) an output generation module. The contents of and the development procedure for each module are described in the following sections.

Spatial Data Base Module

The spatial data base module includes data describing the spatial distributions of geographic features in the study area. The basic features included are a selected road network and the region's boundaries. Other features were also included to represent a sample of the different land topographies that interact with the road network. Thus the final land base in the system contains the following features:

1. Borders of the Aichi Region,
2. Administrative boundaries within the Aichi Region,
3. National trunk roads within the region representing the study road network,
4. Other main roads located within Nagoya,
5. Land use in the area of Nagoya Port where two major trunk roads pass, and
6. Main water supply lines in the Nagoya Port area representing public utilities.

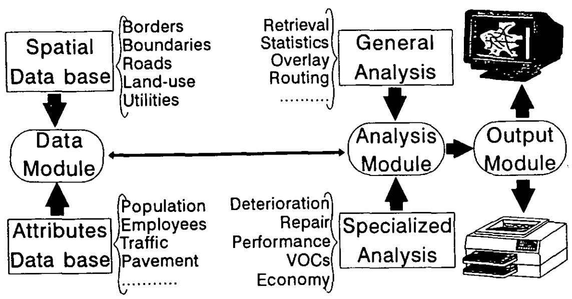


FIGURE 1 Modules of developed HPMS-GIS.

The only appropriate base maps that could be obtained were sets of paper maps; each provides a representation of one of the features to be included in the land base. These maps are originally produced by different authorities for different purposes and thus have different scales and geographic extents. Moreover some of these maps are not even referenced to any standard geographic referencing system, such as latitude-longitude. Thus to establish a common referencing system, common landmarks were selected and marked on each of the paper maps to be used later as reference ticks.

Each of the features was then digitized and stored in a separate coverage by using a suitable feature class (arcs or polygons). In the case of the national trunk roads in the city of Nagoya, each 1-km section was digitized as an arc connecting a beginning and an ending node. During digitization a unique identification (ID) number was assigned to each of the arcs representing highway sections, arcs representing utility lines, polygons representing administrative areas, and so on. The system was then run to generate topological relationships between the features in each coverage. During this step the length of each arc and the area of each polygon representing geographic features also were computed and stored. The reference ticks were used to transform all of the coverages so that all of them would have a similar extent and reference and thus could be perfectly overlaid. With this step done the land base is ready for the spatial analysis and display steps. Figure 2 shows a display of the road network within Nagoya as well as the boundaries and administrative borders. This display is obtained by overlaying different coverages by using location as a common key.

Attribute Data Base Module

The attribute data base module includes the representative nongeographic information associated with each administrative area, road section, land lot, and utility line. Each of these features is represented by the attributes described below.

Administrative Areas

Administrative areas are represented by area ID, area name, and road authority name. All of these attributes are entered and saved in one data base file.

Road Sections

Road sections are represented by section ID, route number, link number, road geometric data (such as kilometer post, number of lanes, lane width, and shoulder width), pavement data [such as pavement type, California bearing ratio, structural number, maintenance control index (MCI; an index describing pavement condition on a scale ranging from 10 to 0, with 10 for excellent condition and 0 for failed pavement), distress amounts, and last repair date and type], and traffic data (such as traffic volume, road capacity, percentage of trucks, directional distribution, and speed). These data sets are entered and stored in three separate data base files. Each record in any of these files contains one set of road data (geometric, pavement, or traffic) besides section ID, route number, and link number. These three variables are used as keys

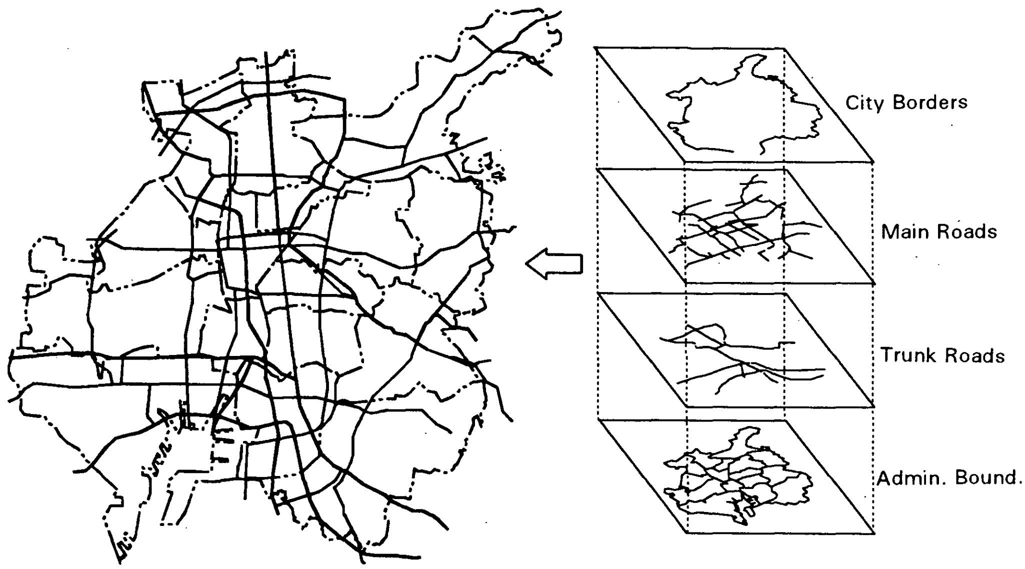


FIGURE 2 Overlay of road network and city borders.

for relating the records among these files (Figure 3). This separation is done to expedite the updating of data and to reduce the burden of maintaining and using files with large amounts of data. This can also reduce data redundancy in a well-structured data base.

Land Lots

Each land lot is represented by an ID number, land use type, and average land price. All of these attributes are stored in a single data base file.

Utility Lines

Each main water supply line is represented by an ID number, pipe diameter, the name of the authority responsible for its repair, and the year of planned future repair. One data base file is used.

Attribute Data Entry

Most of the attribute data were entered by using the conventional tabular format for data entry provided by the INFO data base manager (*10*). However some attribute data could be entered by first referring to a geographic feature on a screen displaying the corresponding coverage and then entering the attribute. For example the name of each administrative area could be entered by this technique (Figure 4). This allows for the direct use of thematic maps as the source of data without the need for coding. Therefore this technique reduces data preparation time and coding errors.

Special care was taken to ensure that the ID given to each data record in the attribute data base matched that given to the corresponding spatial feature. Once this match is accomplished the records in the attribute files can be linked to their spatial features, and thus to each other, by using location as a common key.

Besides the aforementioned attributes additional variables are included in some of the data files to accommodate output values from the analysis module. For example in the pavement data file, variables for expected future condition, repair type, costs, and after-repair condition are included.

Analysis Module

The analysis module consists of two categories of user-written programs. The first category is programs that run fixed sequences of standard ARC/INFO commands on user-selected coverage(s) or attribute data. This includes several programs written in ARC Micro Language (AML) (*11*) and INFO programing language (*10*). These programs help avoid the tedious work required for the step-by-step processing that must routinely be followed to perform a certain type of analysis or display, such as map composition, overlays, queries, data entry, update and retrieval, and general statistics. An example of this group is programs used to generate thematic maps with standard formats, attribute data, colors, titles, legends, and so on.

Besides the category mentioned earlier, for a GIS to be a useful tool like a DSS user-defined procedures in the form of specialized simulation and optimization models must be represented in the system (*3*). This can be accomplished by using the general-purpose programing language supplied within or outside of the GIS environment to represent these procedures. Therefore the second category of programs provides a specialized analysis related to highway management. This category includes programs for condition analysis, prediction of future deterioration of highway pavements, and the selection and simulation of repair alternatives under different budgets and scenarios. Programs for performance evaluation regarding the surface and service conditions of the highway system and changes in vehicle operating costs are also provided. All of these programs are written in FORTRAN 77 and can be called from within the GIS environment by the procedures written in AML.

Geometric Data

ID	Route	Link	KM	No. of lanes	Lane width	....	No. of lanes	Median	....
1	19	37	21	4	3.75	....	4	1.5	....
2	19	37	22	4	3.75	....	4	1.5	....
3	19	38	23	4	3.75	....	4	1.5	....
.	.	..	..	..	....	....	.	...	....
.	.	..	..	..	....	....	.	...	....

Pavement Data

ID	Route	Link	MCI	Pavement Type	SN	....	MCI	Repair type	....
.	..	..	...	..	.	....	...	.	....
1	19	37	7.8	AS	6	....	7.2	1	....
2	19	37	7.2	AS	6	....	7.0	1	....
3	19	38	6.5	AS	6	....	6.0	1	....
.	..	..	...	..	.	....	...	.	....

Traffic Data

ID	Route	Link	Volume	Speed	% Trucks	....	Volume	Capacity	....
.	..	..	.....	..	..	....	.....	.....	....
.	..	..	.....	..	..	....	.....	.....	....
1	19	37	20378	40	11	....	21765	24000	....
2	19	37	20378	40	11	....	21768	24000	....
3	19	38	18875	43	13	....	19666	24000	....

← Current Condition → ← Prediction →

FIGURE 3 Relational structure of attribute data base of road network.

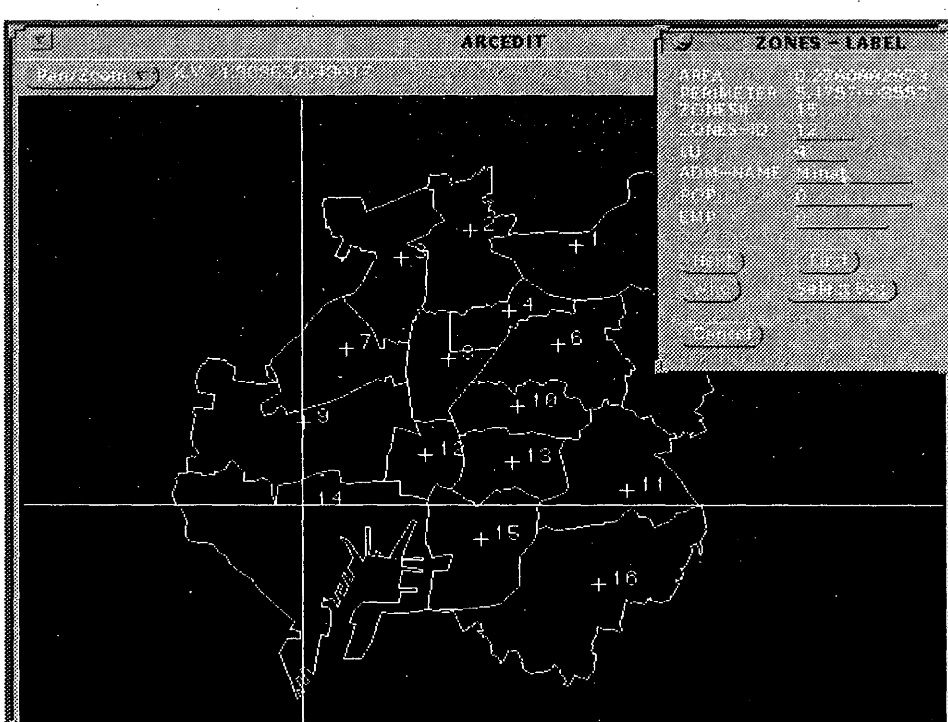


FIGURE 4 Interactive entry of attribute data.

The merit of running these specialized applications from within the GIS environment was realized through another set of programs that take advantage of the spatial analysis capabilities of the GIS. The outputs of these programs are passed to the specialized analysis programs automatically or by the user. Possible spatial manipulations that can be carried out by these programs involve (*12*):

1. Determination of patterns of data associated with locations and the manipulation of location-related data to derive new information from existing data, for example, the analysis of clusters of certain pavement deficiencies by the determination of patterns that indicate nonrandom occurrences and therefore require investigation for other relationships to understand the reasons for the clustering. The result of such an analysis can be fed back to the condition analysis program for example; and
2. Manipulation of data such as overlays, address matching, and routing topologically to create new information from the relationships among the data and the topological structures related to them. An example is finding the best detour path between two nodes on a network to avoid maintenance work on a certain link or overlay land use and road network coverages to get information on the land required for widening a certain route. Almost all of the specialized analysis procedures can benefit from the results of this type of spatial manipulation of the data.

Other required analyses that are not supported by any of the programs described earlier must be carried out interactively. This requires a certain level of skills with GIS. The development of a user interface can vastly facilitate this task. However the developed prototype system does not provide such an interface at the current stage. Nevertheless the ARC/INFO general-purpose menu interface built in to the system, known as ARCSHELL (*13*), can provide an easy pathway for accomplishing many such tasks.

Output Generation Module

The main function of the output generation module is to produce screen displays or hard copies of data and information generated by the analysis module. These outputs are usually on a thematic map or in tabular formats or data files. In general output generation can be done by using one or more of the analysis programs, textual queries, or direct geographic queries (by pointing at a feature on a screen map by using a mouse for example).

POSSIBLE APPLICATIONS AND ADVANTAGES OF SYSTEM

The prototype system that was developed can benefit the different activities within the maintenance management process as well as other highway-related activities. Some of the possible applications and merits of implementing the system are discussed below.

The way in which the data are captured and stored in the system can vastly improve data quality and their availability. Data inconsistency, data redundancy, and coding effort can be minimized. The data held within the system can easily be accessed by staff scattered in different places within and outside the local authority maintaining the system. If mutual data access is achieved the benefits from the wider availability of the data will also be reflected in the effort expended to ensure that the data are up-to-date and that their quality is maintained. There is no doubt that improved data quality and availability will positively affect the quality of decisions made on the basis of those data.

Automated cartography can replace the routine process of manually transferring some of the tabular information on road conditions to a base map as a step in understanding the data. With the powerful cartographic capabilities of GISs, this step can be done more extensively, accurately, and even more cheaply. On the other hand summary reports on conditions, problems, policies, impacts, and achievements that are typically presented in tabular and graphical formats can be supported or even replaced by thematic maps generated by the system. A picture or a map is easier to comprehend than a report. Such an understanding is the basis for more sensible judgment and decision making. Showing information in a map format is most suitable for clearly presenting abstract information to high-level decision makers, politicians, and groups of citizens. Without a GIS it is impractical and time-consuming to extensively show information by manual cartography.

The spatial analysis capabilities of the system can help to revolutionize many of the activities carried out within the highway management system. Spatial integration of the data and pattern determination capabilities can help in the development of sophisticated deterioration models that take into consideration a diversity of factors that may affect deterioration rates. These capabilities would also make it possible to enrich the decision-making process by incorporating other types of data that cannot easily be brought into the process without the ability to spatially relate the data. For example repair decisions that take into account accident analysis, the features of the land surrounding the road such as land use and price (for road widening), and the repair and the timing of installation of utilities embedded in the roadbed can be realized. This can vastly improve the cooperation and coordination between the different local authorities within and outside the highway administration, leading to greater efficiency. Even within the maintenance administration better interaction between offices with different tasks can be achieved because all staff can easily access the same data source.

Topological manipulation capabilities can be employed in developing a realistic simulation of traffic flows under different network conditions. This may help to assign traffic to the best detour routes during the repair of certain links of a road network. The capability of performing geographic queries directly from the display of outputs on the computer screen can also help in the interactive examination and evaluation of problems, policies, and consequences. This would achieve much of the interactiveness between the system and its user that is so strongly required.

Another possible application that could not be added to the prototype system because of time and software limitations is the dynamic segmentation of road networks. In this application each road-related attribute is stored in its own representation of the network, separate from the spatial configuration of the network. Segment boundaries in each thematic network would be defined by the variability of each attribute used in a map. Such a construct would minimize data redundancy while capturing data at any desired level of detail (*4,14*). This also eliminates the need for prior road segmentation.

APPLICATION OF DEVELOPED SYSTEM

The following sections present a sample application of the HPMS-GIS that was developed. The objective of the application is to show how this system can help in the decision-making process by showing various outputs and analysis results produced interactively.

The developed system was used to examine the performance and repair needs of the study road network by assuming different budget levels and repair policies. Several maps representing different stages and types of analysis were produced throughout the application. In general the outputs can be classified into three categories: data retrieval, data analysis, and data integration maps.

Data Retrieval Maps

Data retrieval maps are the counterparts of summary reports generated directly by querying a data base containing raw road data. Two examples of data retrieval maps are shown in Figures 5 and 6. Figure 5 shows the current road (pavement) condition in terms of MCI. Each road segment is highlighted by using different colors or patterns according to the category of its MCI. Summary statistics are also shown on the map. Such maps are routinely produced manually as a preliminary analysis step. The developed system can efficiently perform such tedious work.

Figure 6 shows the current traffic condition in terms of the volume-to-capacity (v/c) ratio. A different technique is used to display segment attributes in Figure 6, in which the buffer width around the road center is used. The buffer width varies according to the v/c ratio, with wider buffers for higher v/c values. By using such a map it is easy to discover locations with low service levels that need widening or improvements. Also the nature of the traffic distribution throughout the network can be understood and better traffic management alternatives can be suggested.

An unlimited number of similar maps can readily be generated by using different data items or a combination of items from the road data base. Three-dimensional maps and overlays would also have been possible if the elevation of features were given related to a fixed datum.

The advantages of this category of maps are that they are clear, are easy to comprehend, can be intensively produced, and can facilitate data consistency checks.

Data Analysis Maps

Data analysis maps can be divided into two types. The first is conceptually similar to and has the same advantages as data retrieval maps, but data analysis maps use information resulting from the analysis module. Figures 7, 8, and 9 show examples of this type. Figure 7 shows the results of a maintenance needs assessment and project selection in which a limited repair budget equals 80 percent of full needs. Two data items are displayed for each segment: the type of required maintenance assessed at the end of year 1991 (shown numerically) and the segment if it is selected for maintenance in 1992 or is deferred (shown as a pattern). The evaluation of such maps by the end user can give good feedback from the system, leading to a fairer and more logical budget allocation. Figure 8 shows a comparison of the predicted surface condition over time if two budgeting scenarios are considered. A higher average MCI is predicted in the case of a decreasing allocation of the available limited budget over the budgeting period (front-loaded investment). Visual comparison of the maps shows that only those segments with high traffic volumes

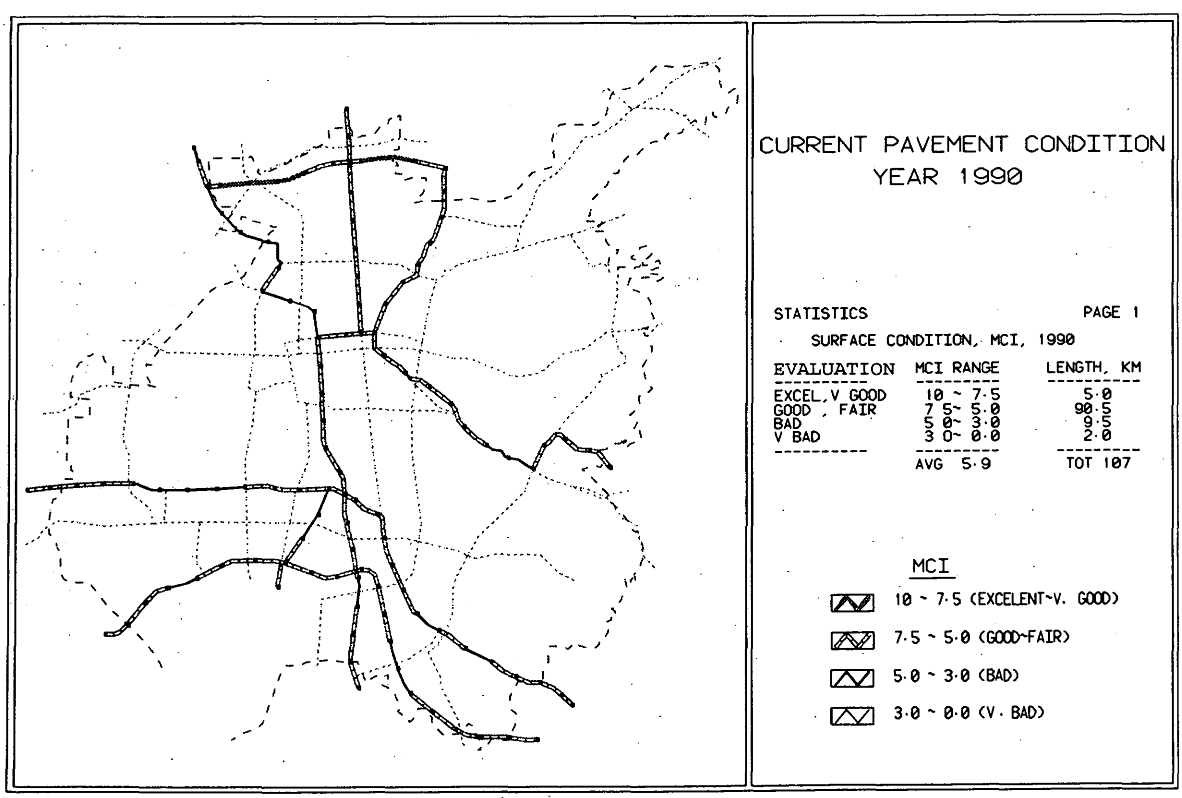


FIGURE 5 **Current pavement condition.**

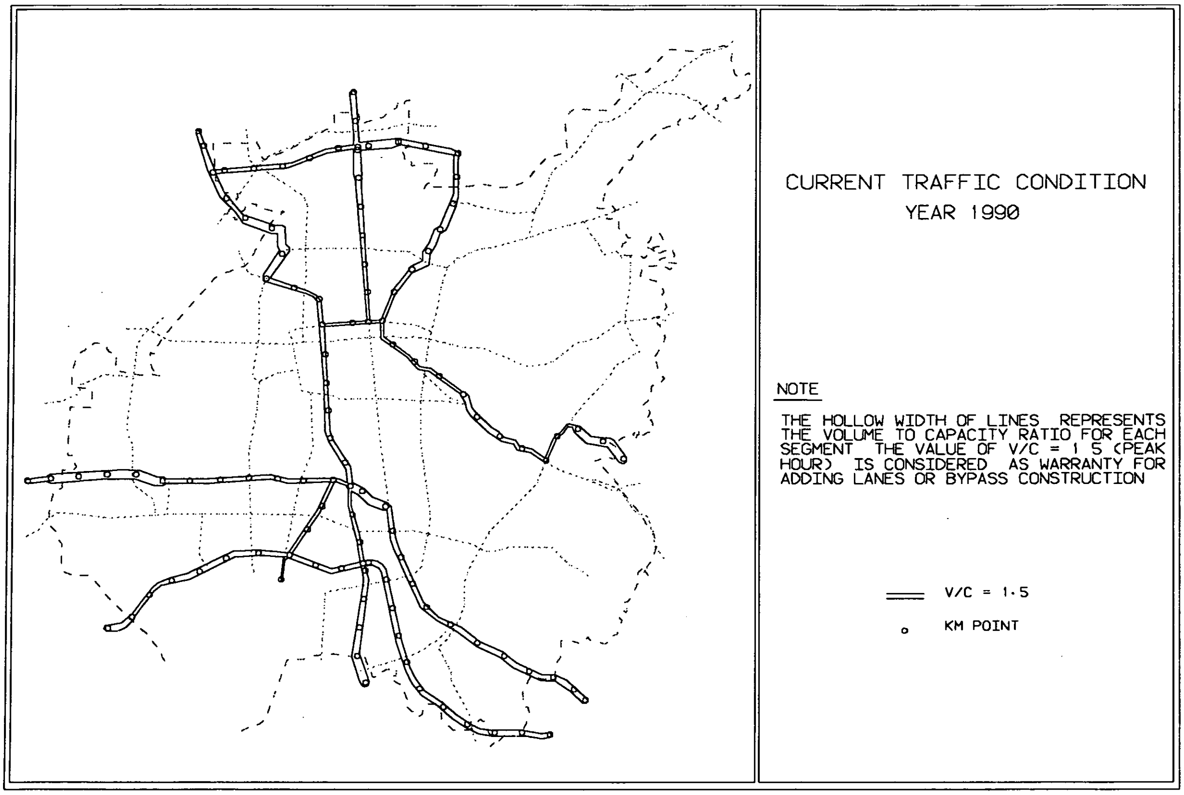


FIGURE 6 Current traffic condition.

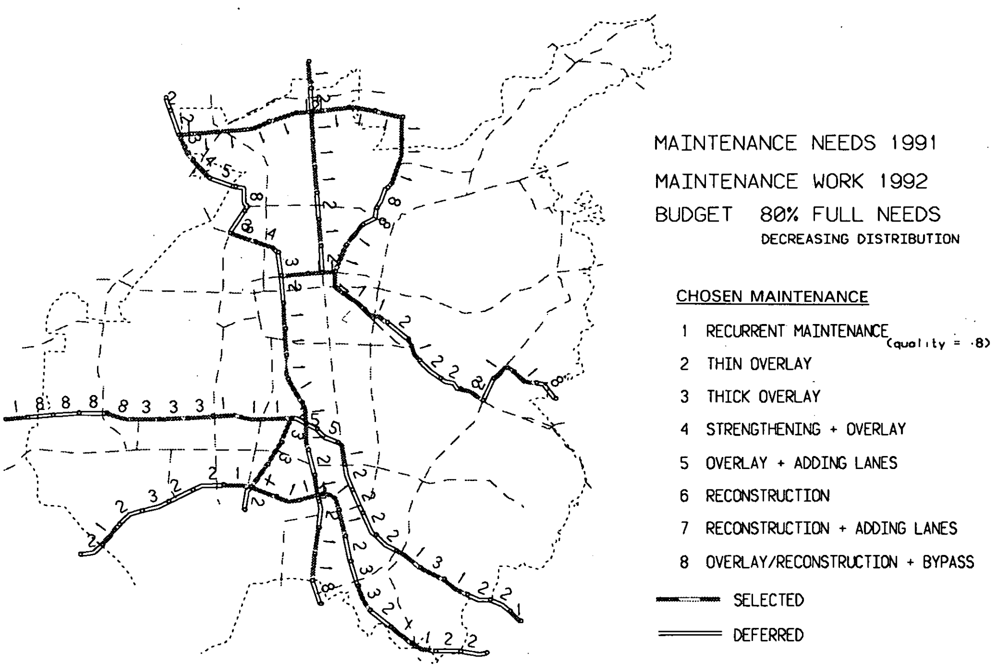


FIGURE 7 Display of predicted repair needs and selection.

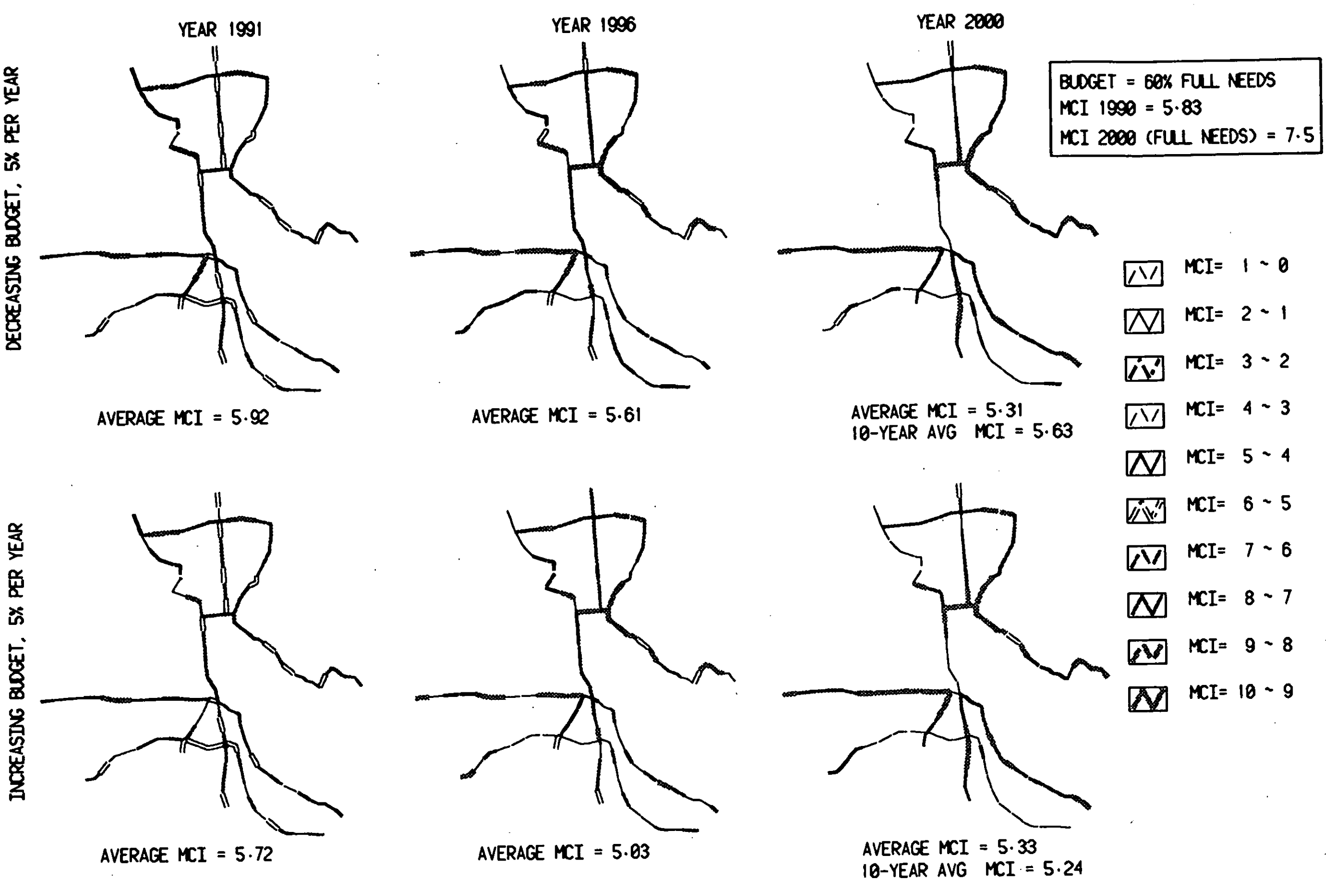


FIGURE 8 Comparison of predicted pavement condition under two different budgeting scenarios over time.

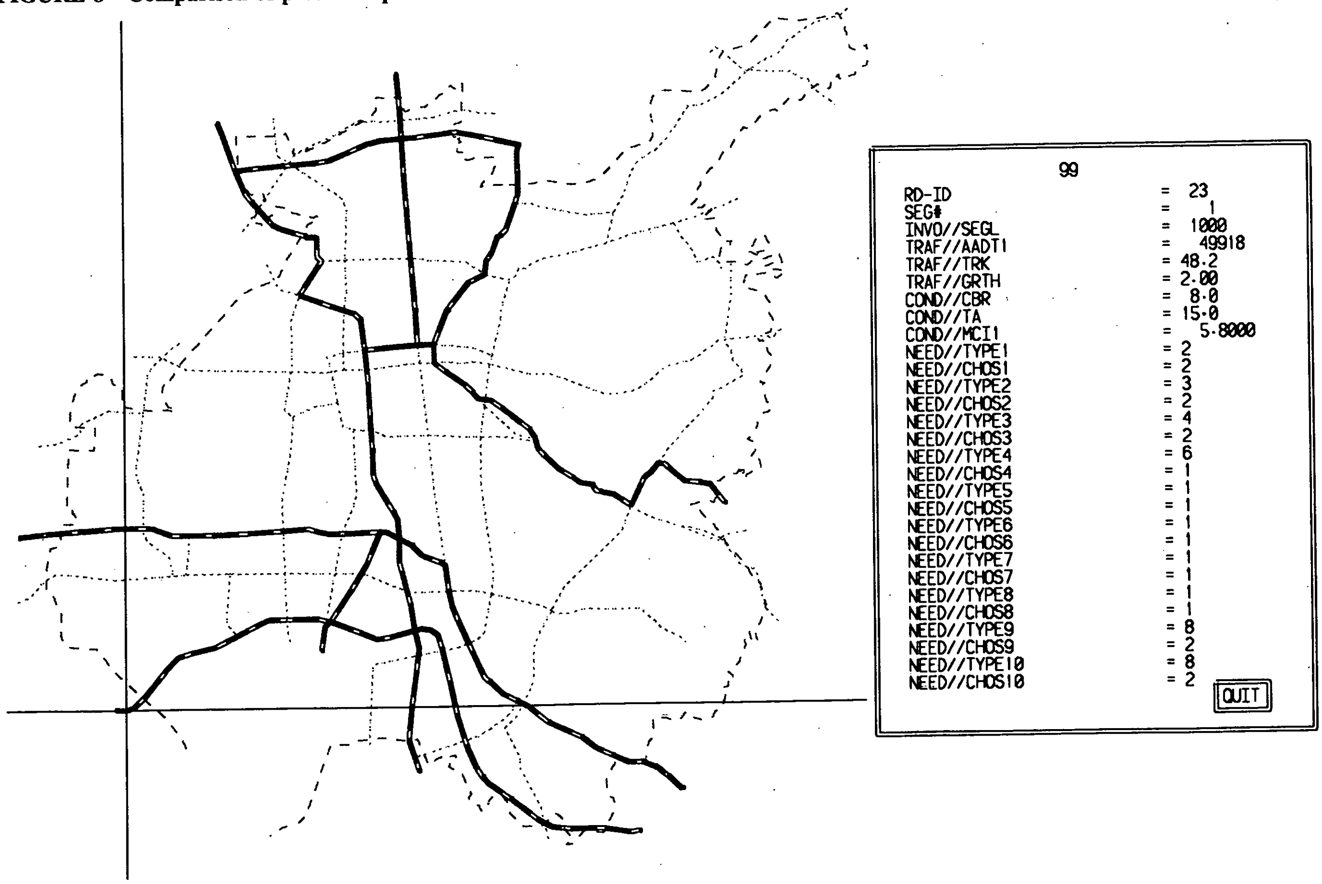


FIGURE 9 Geographic query of maintenance information for a selected road segment.

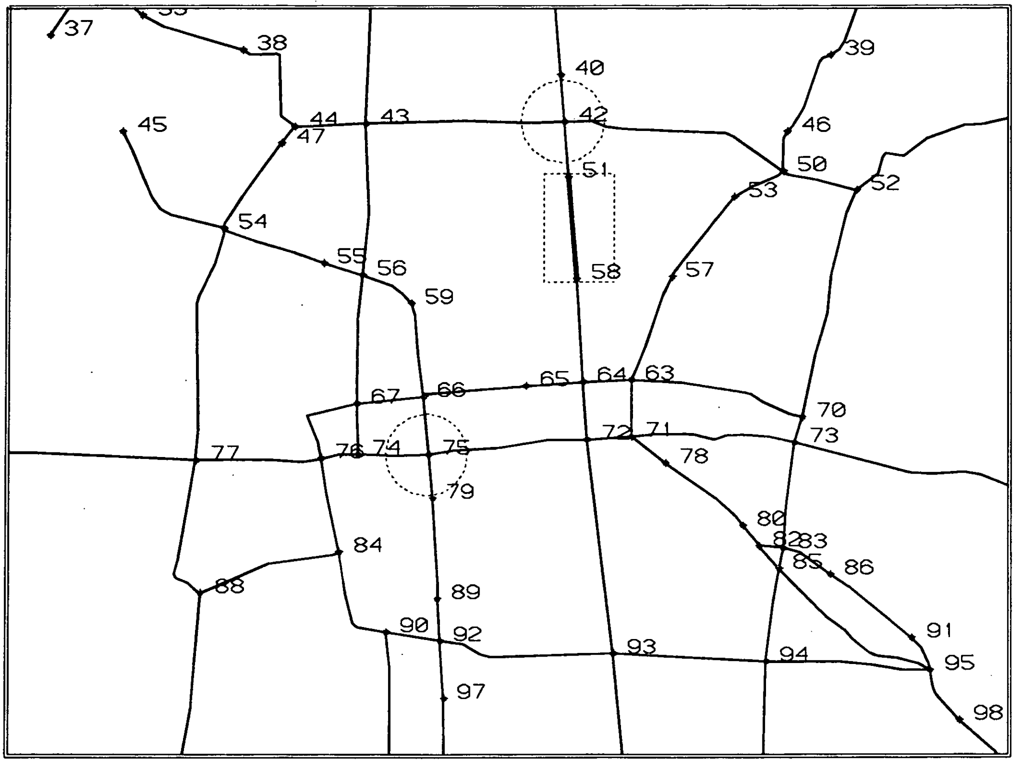
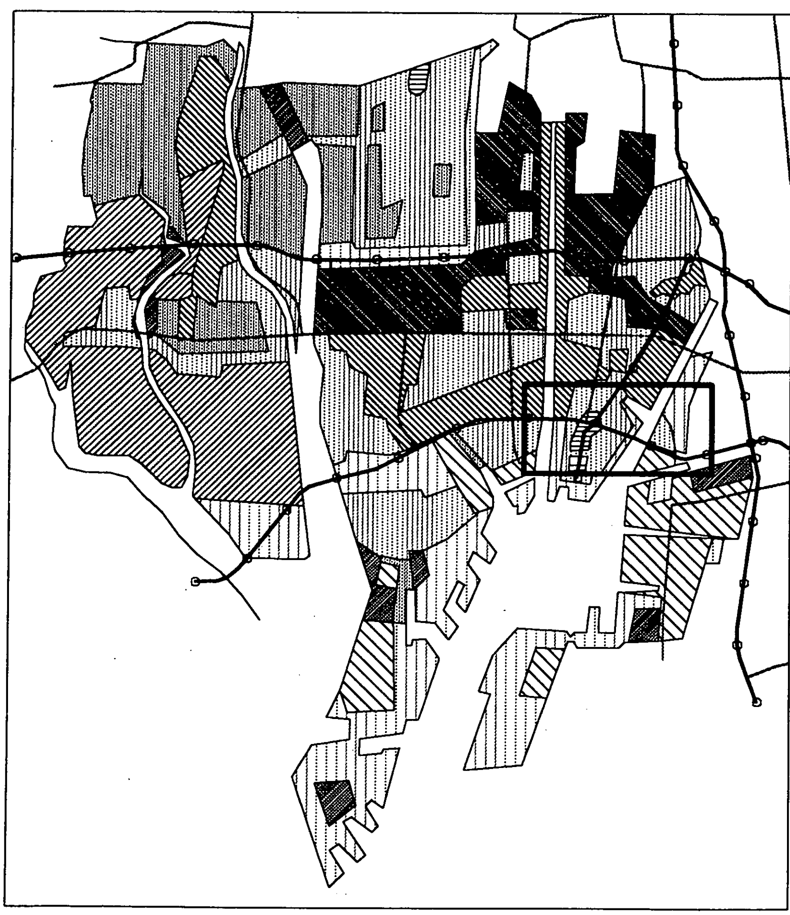


FIGURE 10 Search for best detour route during repair of a link.



LAND ACQUISITION COST IN THOUSAND YEN

RD-ID	SEG#	RD-LENGTH	LU-CODE	COST/M2	TOTAL COST
23	7	281·48	10	200·00	450377·6
23	7	226·84	10	200·00	362950·4
23	7	209·24	7	181·50	303819·4
23	7	198·09	4	330·00	522976·1
23	8	107·50	4	330·00	283807·9
23	8	453·31	7	181·50	658211·9
23	8	100·53	6	194·70	156588·6
23	8	77·29	7	181·50	112238·1
23	9	280·13	7	181·50	406758·9
23	9	108·09	9	116·30	100568·8
		--------			---------
		2042·55			3358297·7

FIGURE 11 Land acquisition requirement for road widening.

have different MCI under each scenario. Figure 9 shows a different type of data query. A road segment is geographically selected on a map on a screen by simply pointing at the segment. A preselected set of information describing road ID, traffic characteristics, pavement design, and maintenance assessment and selection over the analysis period are displayed on the screen as a response to the query. Such a query technique can easily help in clarifying interrelated facts.

The second type of map under this category is that which uses an analysis based on the topologic features maintained by the spatial data base. A possible application is finding and evaluating alternative routes during the shutdown of a road segment for rehabilitation. This application utilizes information on link connectivity, network resources, and turning impedance between links to represent possible routes, directions, and traffic condition on each link. Figure 10 shows an example of this application when the end user is interacting with the system to search for the best detour routes for traffic between nodes 42 and 75 during the application of a planned repair on link 51–58.

Data Integration Maps

In data integration maps several coverages containing diverse types of data are overlaid, and attribute data are integrated by using location as a common key. The result of the overlay is a new coverage containing all spatial features and attributes originally contained in the overlaid coverages. Such an overlay can be useful for both theoretical analyses as well as practical considerations of the features surrounding the road.

An example of possible practical considerations is shown in Figure 11, which shows an overlay of land use and road coverages. The user is analyzing land use, area, and cost of the land area to be acquired along a road section to which a lane is planned to be added where the right-of-way was assessed and was found to be insufficient for the required widening. Figure 12 shows a map used for an analysis for land acquisition for bypass construction as an alternative to the addition of a lane. Figure 12 displays a three-dimensional map of land value in the proposed construction area. The minimum land acquisition cost can be then searched for by analyzing the profile of the land value along several possible routes.

Figure 13 shows an enlarged part of an overlay of roads and the main water supply network. The purpose of this analysis is better coordination between the timing of road repairs and planned installations and the repair of utility lines embedded under the roadbed. The results of this analysis can be fed back to the system as constraints on repair timing or used for communication with other public authorities.

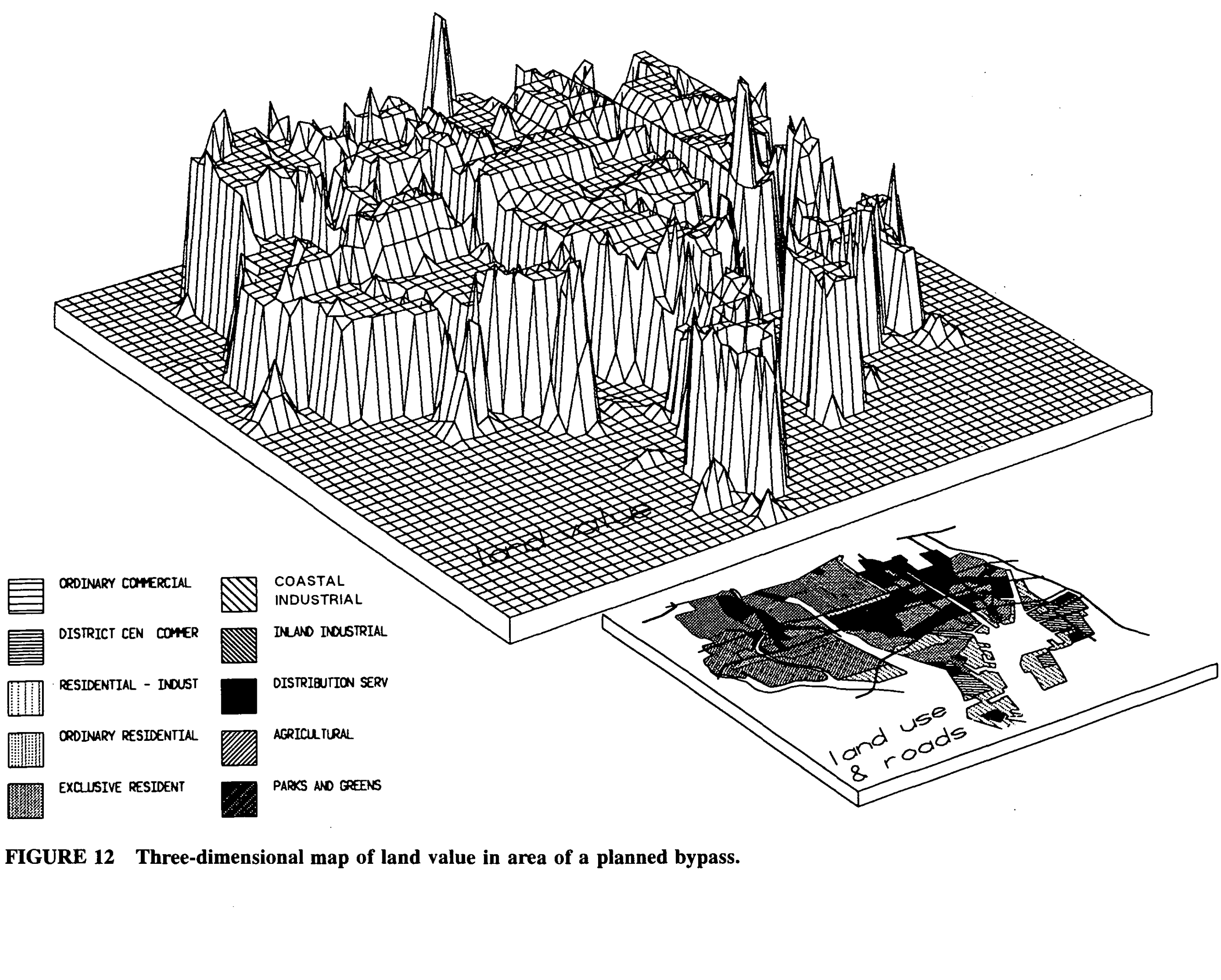
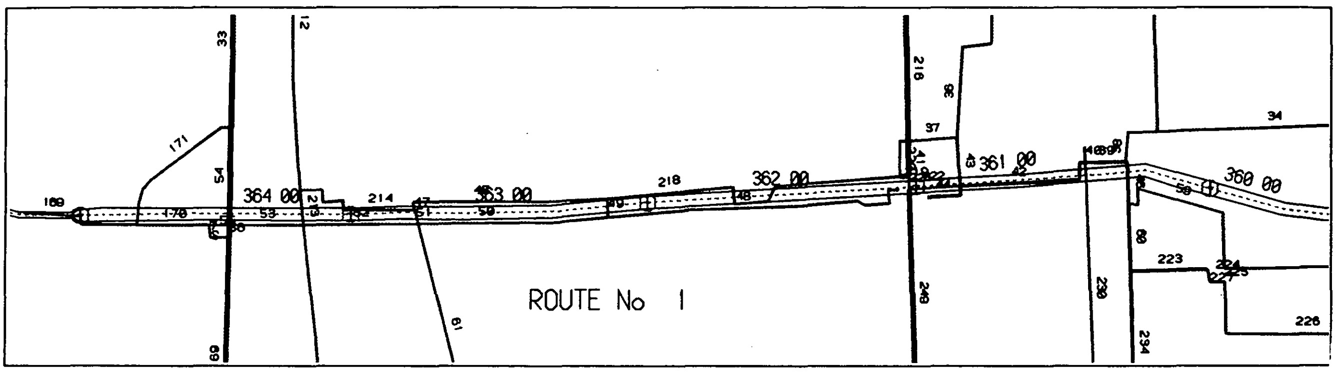


FIGURE 12 Three-dimensional map of land value in area of a planned bypass.

LOCATION OF MAIN WATER SUPPLY LINES



WATER-ID	DIAM(CM)	OVERLAP LENGTH(M)	REPAIR YEAR	AUTHORITY
46	60	45.11	199*	******
230	40	45.02	199*	******
42	60	455.52	199*	******
43	30	30.76	199*	******
222	30	181.39	199*	******
221	40	28.81	199*	******
44	60	14.23	199*	******
249	110	16.25	199*	******
48	30	57.79	199*	******
218	40	50.30	199*	******
218	40	45.00	199*	******
47	40	12.47	199*	******
214	40	248.04	199*	******
49	30	45.13	199*	******
51	40	34.30	199*	******
213	40	45.11	199*	******
171	40	45.15	199*	******
54	140	45.00	199*	******
169	30	225.21	199*	******

DIAMETER 300 ~ 400 mm
DIAMETER 500 ~ 800 mm
DIAMETER 900 ~ 1400mm

FIGURE 13 Coordination between timing of road repair and utility repair and installations.

CONCLUSIONS

The adaptation of GIS in highway management is promising and may vastly improve the quality of the decision-making process. This is particularly true since the problem is inherently spatial. In this paper an HPMS was coupled with a GIS. The resulting prototype system (HPMS-GIS) consists of (a) a spatial data base module that contains several spatial features of a selected study area, (b) an attribute data base module in which representative descriptive data are stored, (c) an analysis module in the form of programs that perform general and specialized analyses, and (d) an output module that produces screen displays and hard copies of the results of the analysis. Possible applications and advantages of the prototype HPMS-GIS were then explored. Results of a case study were also provided.

The main advantages of the system can be summarized as follows;

1. The system can improve the quality and availability of relevant data that will positively affect the quality of the decisions based on them.

2. The system makes it possible to extensively produce automatically generated maps for use in the different stages of the management process, starting with the preliminary analysis and ending with the display of the results and the preparation of reports. Because maps are easier to comprehend, this can be a basis for more sensible judgments and decision making and will be a means of gaining better support from high-level decision makers.

3. The system's powerful spatial queries and its analysis and display capabilities allow for the interaction between the system and its end users for further analysis of outputs. This is a main requirement of an efficient DSS.

4. The spatial integration of data can help users to consider diverse types of data in the analysis, which can enlarge the scope of the decision-making process.

5. Consideration of more features surrounding the road can be given. This can result in better coordination between repair activities and other directly or indirectly related activities.

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Economic Analysis of Effectiveness of Pavement Preventive Maintenance

ABDULLAH I. AL-MANSOUR AND KUMARES C. SINHA

Pavement maintenance can be categorized into two main categories: corrective maintenance and preventive maintenance. The current practices of most highway authorities concentrate on the first category, with minor attention given to preventive maintenance. The main reason for this is the shortage of available funds, which directs some decision makers toward putting the limited funds on corrective measures to satisfy road users, leaving nothing or, at most, negligible portions for preventive maintenance. This situation will continue unless studies show the economic benefits of preventive maintenance. An evaluation of two pavement preventive maintenance techniques, namely, chip seal and sand seal coating, is described. The evaluation process consists of three basic steps. First a data base that included all relevant data on pavement sections was developed. Second a set of models describing pavement deterioration, maintenance costs, and user costs was developed. Finally a comprehensive life cycle analysis was conducted; this included the costs of all items associated with different types and timings of maintenance strategies.

When pavement condition deteriorates below a prescribed minimum level reconstruction or resurfacing activity must be performed. Basic routine maintenance, such as patching, crack sealing, and basic shoulder maintenance activities, tends to slow down the pavement deterioration process, and thus resurfacing or construction can be deferred. However as the pavement ages and its condition deteriorates the cost of basic routine maintenance and the associated user costs increase. Often periodic pavement maintenance such as seal coating is performed to hold the pavement condition above the minimum acceptable level.

Seal coat treatment is a broad term embracing several types of asphalt-aggregate applications placed on any kind of roadway surface. It includes chip seals, sand seals, slurry seals, and fog seals. The most common types, however, are chip and sand seals. Chip sealing involves coating the full width of the roadway section with hot bituminous materials; this is followed by application of a coarse aggregate cover. In sand sealing the cover aggregate is sand rather than coarse aggregate.

The objective of the study described here was to develop an algorithm for evaluating the cost-effectiveness of seal coating activities (chip and sand seals). The algorithm that was developed focuses on the identification of the optimal timing of application of seal coats. This was achieved by using life cycle cost analysis to evaluate the effectiveness of a variety of maintenance strategies by using chip and sand seals.

To accomplish this objective, a data base was developed (*1*). The data base included information on pavement characteristics, pavement routine maintenance and periodic maintenance history, traffic, and pavement performance. These data elements were extracted from the Indiana Department of Transportation (INDOT) data bases. The data were collected over the period from 1984 to 1987. The appropriate data were collected on the basis of contract sections. A contract section is that portion of a highway pavement that is contracted out to one contractor for a specific activity such as resurfacing. Pavement contract sections within 12 of the 37 INDOT subdistricts were included. A stratified sampling scheme was used to select the 12 subdistricts. Four of these subdistricts were located in the northern region of the state, and the other eight subdistricts were located in the southern region of the state.

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PROBLEM FORMULATION

As the pavement section gets old surface roughness increases. User costs as well as basic routine maintenance costs also increase. If at a given point in time a decision is made to seal coat the pavement section, a certain amount of capital is then invested. Seal coating reduces the basic routine maintenance requirements. Because of the resulting improvement in pavement condition, user costs are expected to decline. In addition the service life of the pavement is extended. The main issue here is whether the benefits accrued in terms of reduced basic routine maintenance costs, reduced user costs, and opportunity costs gained because of the deferment of resurfacing equate or exceed the cost of the investment in seal coating. If the savings from seal coating are greater than the investment, the next issues are when is the most economical time to perform seal coating and how many seal coating activities should be performed during the pavement life cycle before the gain from seal coating becomes less than its cost. Figure 1 illustrates this concept. If the seal coating timing is delayed for a certain period of time, say from $t(s1)$ to $t(s2)$, the pavement condition is expected to be worse at $t(s2)$ than at $t(s1)$. Hence the cost of seal coating at a later date would be higher. The benefits from seal coating acquired from reductions in basic routine maintenance and user costs could be less than those from seal coating at the previous time, but there are gains in the added service life. To determine the best seal coating strategy, the costs and benefits need to be discounted to a common base for comparison.

MATHEMATICAL FORMULATION OF LIFE CYCLE COST ALGORITHM

The total life cycle costs, as used in the present study, consist of resurfacing or reconstruction cost, basic roadway and shoulder routine maintenance costs, seal coating costs, and user costs. The resurfacing cost was considered to be a single payment made at

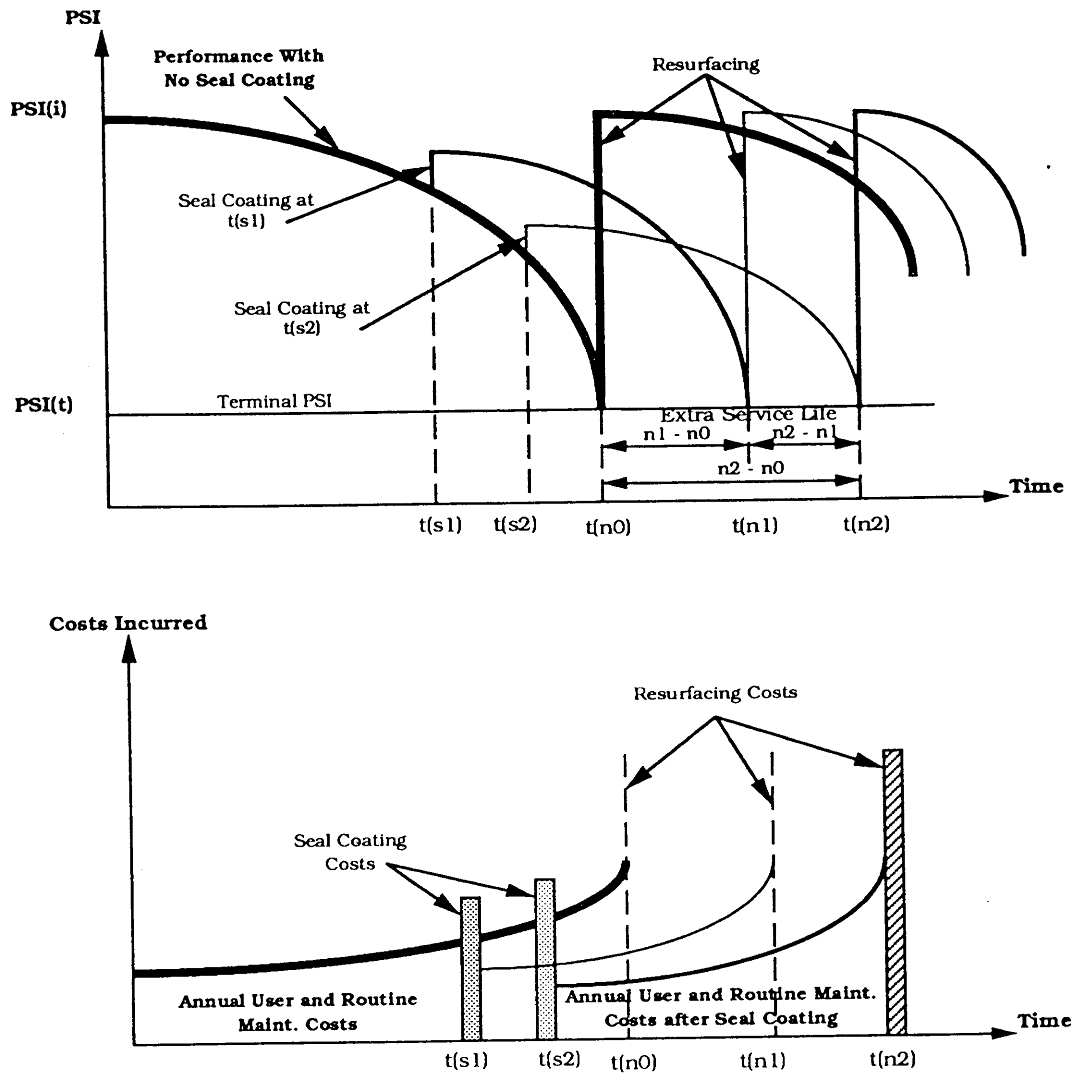

FIGURE 1 Timing effects of seal coating activity.

a future year depending on the given standard for terminal pavement condition. The basic roadway and shoulder routine maintenance costs were considered to be annual single payments at the end of each year. Seal coating costs were treated as single payments made in the years in which sealing was carried out. User costs were accounted as annual single payments at the end of each year on the basis of the pavement condition index in that year.

Because future decisions on the timing of a particular type of maintenance activity are uncertain, it is a common practice in life cycle cost analysis to assume certain sequences and types of maintenance work. This corresponds to the well-known repeatability assumption in financial analysis. After the first pavement's life cycle, the same work sequence and type are assumed to be repeated in perpetuity.

Having established the basic assumptions of the proposed life cycle costing approach, the present worth of resurfacing cost is determined by applying the following formula:

$$\text{PWV1} = \text{RS} \cdot (\text{SPPWF}, i, n) \quad (1)$$

where

PWV1 = present worth of resurfacing [$/1.61 lane-km ($/lane-mi)],
RS = resurfacing cost [$/1.61 lane-km ($/lane-mi)],
SPPWF = single payment present worth factor = $[1/(1 + i)^n]$,
i = discount rate (in decimals), and
n = pavement life cycle (service life).

Similarly the present worth of the periodic maintenance cost (seal coating cost) is determined by using Equation 2:

$$\text{PWV2} = \text{SC} \cdot (\text{SPPWF}, i, t) \quad (2)$$

where

PWV2 = present worth of seal coating cost [$/1.61 lane-km ($/lane-mi)],
SC = seal coating cost [$/1.61 lane-km ($/lane-mi)], and
t = year at which seal coating is performed.

The present worth of annual basic routine maintenance costs is calculated by using Equation 3.

$$PWV3 = \sum_{j=1}^{n} AMC_j \cdot (SPPWF, i, j) \quad (3)$$

where PWV3 is the present worth value of all annual basic routine maintenance costs during the pavement service life [\$/1.61 lane-km (\$/lane-mi)], and AMC_j is the annual basic routine maintenance cost at the jth year [\$/1.61 lane-km (\$/lane-mi)].

Similarly the present worth of user costs is determined by applying Equation 4.

$$PWV4 = \sum_{j=1}^{n} UC_j \cdot (SPPWF, i, j) \quad (4)$$

where PWV4 is the present worth value of all annual user costs [\$/1.61 lane-km (\$/lane-mi)], and UC_j is the annual user cost at the jth year [\$/1.61 lane-km (\$/lane-mi)].

The total present worth value of all life cycle cost components under a maintenance strategy is calculated by using the following summation:

$$TPWV = PWV1 + PWV2 + PWV3 + PWV4 \quad (5)$$

This amount can then be viewed as an outlay to be made in perpetuity every n years. Then the total present worth value in perpetuity can be expressed as

$$TPWV_p = TPWV\{(1 + i)^n/[(1 + i)^n - 1]\} \quad (6)$$

Finally the equivalent uniform annual cost in perpetuity for a maintenance strategy over the pavement service life is determined by applying Equation 7.

$$EUAC_p \text{ (in perpetuity)} = TPWV_p \cdot i \quad (7)$$

DEFINITION OF MAINTENANCE STRATEGIES

The analysis in the present study focused on basic routine maintenance and preventive periodic maintenance activities. Basic routine maintenance included both roadway and shoulder activities. The roadway maintenance consisted of shallow patching, deep patching, sealing of longitudinal cracks, and crack sealing. The shoulder maintenance involved spot repair of unpaved shoulder, blading of unpaved shoulder, and clipping of unpaved shoulder. Periodic maintenance activities are those used as preventive measures to repair minor damage and to hold the pavement condition until higher-order treatments, such as resurfacing, become necessary. In the present study these included chip sealing and sand sealing.

Several pavement maintenance strategies were included in the life cycle cost analyses. Each strategy consisted of one or more maintenance activities. These strategies ranged from resurfacing at the end of a period of no maintenance to routine basic roadway and shoulder maintenance in conjunction with seal coating at different levels of pavement condition. Table 1 lists the various maintenance strategies considered in the life cycle cost analysis.

TABLE 1 Maintenance Strategies Considered in Life Cycle Cost Algorithm

Maintenance Strategies	Activities Performed
Do Nothing	None
Basic Routine Maintenance (BRM)	Patching, Crack Sealing, Spot Repair of Unpaved Shoulders, Blading Unpaved Shoulders, Clipping Unpaved Shoulders
BRM and Chip Sealing	BRM Activities and Chip Sealing before Resurfacing
BRM and Sand Sealing	BRM Activities and Sand Sealing before Resurfacing

RELATIONSHIPS NEEDED FOR LIFE CYCLE COST ANALYSIS

Pavement maintenance life cycle costing requires the determination of the rate of pavement condition deterioration and the timing for resurfacing. The effectiveness of various maintenance activities must also be known. The scheduling of maintenance activities depends on the effectiveness of these activities. To determine the rate of pavement condition deterioration and the effectiveness of maintenance activities, a set of condition prediction models was developed. The timing for resurfacing was estimated by determining the age (number of years) after which the pavement condition reaches a terminal value. The costs of performing different maintenance activities were also needed for the economic analysis. This section presents a brief description of the different models needed for the life cycle cost analysis.

Pavement Condition Prediction Models

The available data base was used to develop a relationship between pavement serviceability index (PSI) and pavement age. Two groups of pavement contract sections were identified. The first group included all pavement contract sections that did not receive any maintenance during the study period (1984 to 1987). The second group included the pavement contract sections that received basic roadway and routine shoulder maintenance activities. To capture the effect of climate on pavement condition, the pavement contract sections within each group were classified according to the climatic region. The pavement contract sections within each climatic region were further subdivided on the basis of their annual average daily traffic (AADT) into two categories: pavement contract sections with high traffic levels (AADT $>$ 2,000) and those with low traffic levels (AADT $\leq$ 2,000). The latest subdivision was applied to account for the effect of traffic level on pavement condition. However because of the limited numbers of pavement contract sections within each climatic region on which no maintenance was performed, the effect of traffic was excluded from this group.

The regression procedure of the Statistical Analysis System (SAS) computer package (*2*) was used to test a large number of models. The best model was found to be in the following form:

$$\text{PSI} = a + b \cdot \text{Age} \tag{8}$$

where

PSI = pavement serviceability index,
Age = pavement age (in years) since construction or last resurfacing, and
a, b = estimated regression parameters.

The statistical characteristics and estimated regression parameters are presented in Table 2.

A PSI value of 2.2 was used as the minimum value for acceptable pavement serviceability. On the basis of this terminal value the pavement service life of contract sections with no maintenance was found to be 16 years in the northern region and 19 years in the southern region. When basic roadway and shoulder maintenance activities were applied the pavement service life was extended to 20 years for the northern region with a high raffic level, 23 years for the northern region with a low traffic level, 22 years for the southern region with a high traffic level, and 24 years for the southern region with a low traffic level.

Gain in PSI Due to Seal Coating

In the available data base a total of 34 pavement contract sections received chip sealing and 20 pavement contract sections received sand sealing. Eleven sections were located in the northern region and the remaining 23 were located in the southern region of the state. All sections that received sand sealing were located in the northern region of the state. Both chip and sand sealing were found to result in an improvement in pavement condition.

To determine a functional relationship between the immediate gain in PSI and the PSI at the time of application of the seal coating, the regression procedure of the SAS package was used. The immediate gain in PSI represents the change in PSI estimated within 1 year of undertaking a seal coating activity. The following form of such a relationship was found to be statistically valid for both chip sealing and sand sealing activities.

$$\Delta\text{PSI} = a \cdot (\text{PSI} - b) \tag{9}$$

where

ΔPSI = gain in pavement serviceability owing to seal coating activities,
PSI = PSI at time of seal coating application, and
a, b = estimated regression parameters.

Table 3 summarizes the results of the statistical analysis and presents the estimated regression parameters for both chip and sand sealing activities.

Routine Pavement Maintenance Cost Models

Many factors could be postulated as affecting annual routine maintenance expenditures, for example, the pavement's condition, the pavement's age, traffic loads, the maintenance procedures performed, and the availability of funds. In the present study the annual amounts of basic routine maintenance on roadways and shoulders were related to pavement condition at the two traffic

TABLE 2 Estimated Regression Parameters of Pavement Condition Prediction Models

PSI = a + b*Age

Climate Region	Maintenance Category	Overall Model Statistics				Estimated Parameters	
		No. of Observ	R^2	Adj R^2	P > F	a	b
North	No. Maint.	13	0.4797	0.3149	0.0084	3.8816	-0.1051
	Basic Routine Main						
	* High Traffic (AADT > 2000)	33	0.4127	0.3943	0.0008	3.9732	-0.0885
	* Low Traffic (AADT < =2000)	43	0.5403	0.5294	0.0001	4.1523	-0.0817
South	No. Maint.	45	0.5407	0.5301	0.0001	4.0135	-0.0978
	Basic Routine Main						
	* High Traffic (AADT > 2000)	48	0.5822	0.5733	0.0001	4.2315	-0.0915
	* Low Traffic (AADT < =2000)	102	0.4081	0.4023	0.0001	4.0736	-0.0773

TABLE 3 Estimated Regression Parameters of Gain in PSI Models

Gain in PSI = a*(PSI - b)						
Seal Coating Activity	Overall Model Statistics				Estimated Parameters	
	No. of Observ.	R^2	Adj R^2	P > F	a	b
Chip Seal	34	0.5453	0.5302	0.0001	0.3325	1.433
Sand Seal	20	0.5588	0.5147	0.0053	0.3728	1.9139

levels. Other factors were assumed to be either constant or confounded with the factors considered.

The pavement contract sections in the data base whose roadways and shoulders received basic routine maintenance were grouped on the basis of their AADTs into sections with high levels of traffic (AADT > 2,000) and sections with low levels of traffic (AADT ≤ 2,000). The average value of the AADT was used as the cutoff point between low and high traffic levels.

The maintenance expenditures versus pavement condition models were developed separately for roadway and shoulder maintenance activities. The functional form of the expenditures versus pavement condition models is given in Equation 10.

$$\text{Log AMC} = a + b \cdot (\text{PSI}) \tag{10}$$

where

AMC = annual roadway or shoulder maintenance expenditure [$/1.61 lane-km ($/lane-mi)],
PSI = PSI at time of maintenance, and
a, b = estimated regression parameters.

The statistical characteristics of these models are given in Table 4.

Sand and Chip Sealing Cost Models

The expenditures for performing chip and sand sealing activities per 1.61 lane km (lane mi) were related to the pavement condition at the time that these activities were performed. It was found that the costs of these activities were higher when performed on sections with poor pavement condition than on those with good pavement condition. This finding was as expected since more materials and human-hours are required to perform seal coating activities on pavements in poor condition than on pavements in good condition. The functional form of the models is given as follows:

$$\text{Log SC} = a + b \cdot (\text{PSI}) \tag{11}$$

where

SC = cost of performing chip sealing or sand sealing [$/1.61 lane-km ($/lane-mi)],
PSI = pavement serviceability index at time of sealing, and
a, b = estimated regression parameters.

The statistical parameters for these models are given in Table 5.

Development of User Cost Models

The life cycle cost algorithm that was developed provides an option for the inclusion of user costs as a function of pavement condition. Basically the user cost models determine the operating costs owing to a decrease in PSI. The consumption rate tables, developed by Zaniewski et al. (*3*) in the FHWA study, are the basis for these models. In the present study the operating costs were given by vehicle type, vehicle speed, pavement condition, and road geometrics. In the present study the cost numbers were updated to 1987 dollars by using FHWA cost indexes for maintenance and operations. The costs were updated for various levels of PSI, different types of vehicles, zero grade, and a speed of 89

TABLE 4 Estimated Regression Parameters of Annual Basic Routine Maintenance Cost Models

Log AMC = a + b*PSI							
Type of Maint.	Traffic Level	Overall Model Statistics				Estimated Parameters	
		No. of Observ	R^2	Adj R^2	P > F	a	b
Roadway Maint.	High Traffic (AADT > 2000)	55	0.5193	0.5141	0.0001	4.0283	-0.4621
	Low Traffic (AADT < =2000)	67	0.5887	0.5824	0.0001	3.7781	-0.4252
Shoulder Maint.	High Traffic (AADT > 2000)	14	0.4099	0.3645	0.0010	3.3221	-0.3547
	Low Traffic (AADT < =2000)	27	0.5693	0.5328	0.0001	3.5323	-0.4573

TABLE 5 Estimated Parameters of Seal Coating Maintenance Cost Models

Log SC = a + b*PSI						
Type of Seal Coating	Overall Model Statistics				Estimated Parameters	
	No. of Observ.	R^2	Adj R^2	P > F	a	b
Chip Seal	34	0.3079	0.2723	0.0018	3.6101	-0.1034
Sand Seal	20	0.4814	0.4597	0.0001	3.3427	-0.0782

km/hr (55 mph). User costs included only vehicle operating costs. Costs arising from accidents and travel time were not included in the analysis.

In the life cycle cost algorithm for the present study a running speed of 89 km/hr (55 mph) and 0 percent road grade were assumed. The performance history of a particular type of pavement during its entire service life was estimated by calculating a PSI value for each year. The linear relationships developed earlier were used to determine pavement deterioration from an initial PSI to a terminal PSI value. If seal coating activities were performed at a given time the PSI value was adjusted for these activities. PSI values for future years were then recalculated by using the same linear models on the basis of the PSI after seal coating.

To convert the user cost values from dollars per 1609 km to dollars per 1.61 lane-km (dollars per 1,000 mi to dollars per 1 lane-mi), the reported AADT was multiplied by 0.5, since most of the state roads are two lanes. The final formula to change user cost unit to cost per 1.61 lane-km (lane-mi) is given by the following equation.

$$\begin{aligned} UC(I) = [(&UC_{pc}(I) \cdot ADT \cdot PPC \\ &+ UC_{st}(I) \cdot ADT \cdot PST \\ &+ UC_{tt}(I) \cdot ADT \cdot PTT)/1{,}000] \cdot 365 \end{aligned} \quad (12)$$

where

$UC(I)$ = user cost for year I [\$/1.61 lane km (\$/lane mi)],
UC_{pc} = operating cost for passenger cars,
ADT = AADT · 0.5,
PPC = percentage of passenger cars,
UC_{st} = operating cost for single-unit trucks,
PST = percentage of single-unit trucks,
UC_{tt} = operating cost for semitrailer trucks,
PTT = percentage of semitrailer trucks.

CODING OF LIFE CYCLE COST PROGRAM

An interactive computer program was developed by using the proposed methodology for the pavement maintenance life cycle cost analysis. The computer program incorporated the pavement condition and maintenance cost models developed in the study and was written in FORTRAN language. The program inputs include maintenance strategy, terminal value of pavement serviceability, traffic characteristics, climatic region, years at which chip or sand sealing should be considered, resurfacing cost, and discount rate. The output of the program includes a list of pavement age and corresponding pavement serviceability, annual routine roadway maintenance, annual routine shoulder maintenance, and annual user cost. The program also prints out the costs of chip sealing and sand sealing if these activities are used. The program output also produces the equivalent uniform annual agency cost in perpetuity, equivalent uniform annual user cost in perpetuity, and equivalent uniform annual total cost of the selected maintenance strategy in perpetuity.

APPLICATION OF LIFE CYCLE COST ANALYSIS

The life cycle cost analysis program was used to determine the most cost-effective pavement maintenance strategy and to find the appropriate timing of chip and sand sealing activities. A terminal serviceability value of 2.2 was used. When pavement condition reaches the terminal value, the pavement was assumed to be at the end of its service life and a resurfacing was needed. The resurfacing cost was estimated to be \$50,000 per 1.61 lane-km (lane-mi). This value was selected on the basis of the information provided by the INDOT (*4*) for the following resurfacing criteria:

1. 20-cm (8-in.) bituminous base for heavy traffic,
2. 3.8-cm (1.5-in.) bituminous binder for heavy traffic, and
3. 2.5-cm (1-in.) bituminous surface for heavy traffic.

The trigger point at which the seal coat was to be applied was varied; the following three PSI trigger point values were tested and analyzed: 3.25, 3.00, and 2.75. The objective of choosing these values was to study the optimal timing for seal coating activities.

For the purpose of calculating user costs, four levels of AADT (3,000, 2,500, 1,500, and 1,000) with a speed of 89 km/hr (55 mph) on a flat roadway were considered. The traffic volume was assumed to comprise 85 percent passenger cars, 5 percent single-unit trucks, and 10 percent semitrailer trucks. For all maintenance strategies resurfacing cost, seal coating costs, annual maintenance costs, and annual user costs were given in 1987 dollars. The discount rate was assumed to be 6 percent. A discussion of the comparison of agency costs, user costs, and the total costs of different maintenance scenarios are presented in the following sections.

Comparison of Agency Costs

The analysis illustrates a consistently declining agency cost as the maintenance scenario changes from no maintenance to basic routine maintenance and as the number of seal coating activities increases during a pavement's lifetime. Agency costs were found to increase as the seal coating decision was postponed. This increase was more noticeable for pavement contract sections with high traffic levels than for sections with low traffic levels. This finding suggests that the optimal timing for performing seal coating activities is when the pavement condition reaches a PSI value of 3.25. On the basis of pavement performance data analysis in the study, this condition would occur when a pavement is about 8 years old in the northern region of the state and about 11 years old in the southern region. The reason for the difference in pavement ages at which seal coating activities become necessary may be related to the effect of the relatively harsher winter weather in the northern zone.

To compare agency cost savings as a result of performing chip and sand sealing, the results of applying these activities three consecutive times at a PSI trigger value of 3.25 were considered. The results indicated that performance of chip sealing instead of sand sealing would result in an average annual saving of 42 percent of agency cost for pavement contract sections with high traffic levels and an average annual saving of 56 percent of agency cost for pavement contract sections with low traffic levels. On the other hand sand sealing activities resulted in an average annual savings of 35 percent for pavement contract sections with high traffic levels and 54 percent for pavement contract sections with low traffic levels.

Comparison of User Costs

The analysis suggests that from the user viewpoint and for AADT of less than 2,000, the optimal maintenance strategy may be to perform basic routine maintenance and one chip sealing or sand sealing activity at a PSI trigger value of 3.00. For pavement sections with higher traffic levels, however, performance of basic routine maintenance and two or three seal coating activities within a pavement's service life is justifiable. To achieve the maximum user cost savings at either high or low traffic levels, the seal coating activities should not be postponed beyond a PSI value of 3.00. The analysis further indicated that if the seal coating activities are not performed at or before the pavement condition reaches a PSI value of 3.00, the best strategy from the user's point of view is to continue with the annual basic routine maintenance to the end of the pavement's service life. In comparing chip and sand sealing the analysis did not detect any major difference in the user cost values between the two activities on any of the traffic levels considered.

Comparison of Total Costs

The comparison between different maintenance strategies was based on the total uniform annual costs in perpetuity, that is, the summation of the total uniform annual agency costs and annual uniform user costs in perpetuity. The annual savings in dollars per 1.61 lane-km (1 lane-mi) per year in perpetuity under different maintenance strategies were calculated for the two climatic regions. The calculation indicated that if seal coating was performed at or before the pavement condition reaches a PSI value of 3.00, the maintenance strategy with the most savings was the application of three consecutive chip seals with basic routine maintenance. This result is true for both traffic levels and both climatic regions. However if seal coating activities are deferred beyond a PSI value of 3.00, the application of basic routine maintenance and one chip seal produced the most savings for both traffic levels and both climatic regions. The calculation also indicated that if seal coating is performed at a PSI trigger value of 2.75, the savings obtained from applying basic routine maintenance and three consecutive sand sealing activities were about equal to the savings resulting from performing basic routine maintenance and two consecutive chip sealing activities. Another observation is that for all maintenance strategies in the southern region the savings achieved at high traffic levels were much greater than those achieved at low traffic levels. However for the northern region a higher level of savings was achieved at high traffic levels if the seal coating was performed at or before the pavement condition reached a PSI value of 3.00. If seal coating was postponed beyond a PSI value of 3.00, pavement contract sections with low traffic levels seemed to produce higher savings than sections with high traffic levels.

SUMMARY AND CONCLUSION

To evaluate the cost-effectiveness of seal coating treatments a life cycle cost algorithm was developed. The components of the life cycle cost included annual routine maintenance costs, seal coating costs, future costs of resurfacing, and user costs. An interactive computer program was then developed on the basis of the life cycle cost algorithm encompassing the developed pavement performance and maintenance cost models.

The application of the life cycle cost program indicated that the optimal timing for performing seal coating, from the agency cost viewpoint, is when the pavement condition reaches a PSI value of 3.25. To achieve the most user cost savings for both high and low traffic levels, the seal coating activities should not be postponed beyond a PSI value of 3.00. As far as the total cost (agency plus user costs) is concerned, the most savings are obtained for both traffic levels and the two climatic regions when basic annual routine maintenance and three consecutive chip sealing activities during the pavement's service life were performed at a PSI trigger value of 3.00. However if seal coating is deferred beyond a PSI value of 3.00, the application of one chip sealing activity during the pavement's service life produced the treatment with the most cost savings.

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PART 2

Safety

Relationship Between Initial Speed and Speed Inside a Highway Work Zone

RAHIM F. BENEKOHAL AND LI WANG

Drivers' speeds in the advance warning area may influence the speeds that they maintain throughout the work zone. The correlations between the speed at the end of the advance warning area and the speed in the work activity area are examined. Vehicles are grouped into six driver categories or subcategories on the basis of their speed profiles. Vehicles were also divided into Fast-, Faster-, Very Fast-, and Fastest-moving groups on the basis of their initial speeds. Vehicles with higher initial speeds, in general, reduced their speeds more than the vehicles with lower initial speeds; however, vehicles in the higher initial speed groups kept higher speeds in the work zone than the vehicles in the lower initial speed groups, even though the former group had larger speed reductions than the latter group. Drivers were classified into the combinations of speed groups and driver categories. The drivers in a category were not distributed uniformly among the four speed groups. Similarly the drivers in a speed group were not distributed evenly among the driver categories. The data showed that the great majority of the drivers who did not reduce their speeds in work zones belonged to the Faster and Very Fast groups. About one-third of the drivers who were "extremely" speeding (Very Fast and Fastest groups) reduced their speeds and kept reducing them as they traveled in the work activity area. However about one-third of those who were "excessively" speeding (Fast and Faster groups) reduced their speeds initially, but increased them in the work activity area and then reduced them when they reached the work space.

Drivers travel at different speeds in the advance warning area, and their speeds may influence the speeds that they maintain throughout the work zone. This paper discusses how the speed of a vehicle at the end of the advance warning area may influence its speed in the work activity area. It examines the correlation between the speed at the beginning of the lane closure taper with the speed inside the one-lane section of the work zone.

The speeds at the beginning of the taper are called initial speeds and are used to group the vehicles into four speed groups: Fast-, Faster-, Very Fast-, and Fastest-moving groups. The initial speeds for Fast, Faster, Very Fast, and Fastest groups are 74 to 87 km/hr (46 to 54 mph), 86 to 103 km/hr (55 to 64 mph), 105 to 111 km/hr (65 to 69 mph), and over 113 km/hr (70 mph), respectively. Depending on the correlation between speeds in the work activity area and the initial speed, one needs to deal with different types of speeding drivers in each speed group.

Furthermore drivers are grouped into four driver categories on the basis of their speed profiles. The four categories include a total of six driver categories and subcategories. Then the distribution of drivers in the speed groups and driver categories are examined. If speed reduction depends on the initial speed, do drivers in one speed group mainly belong to one driver category? The answer to this question would help to determine where the focus of work zone speed control efforts should be.

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The results obtained from the present study will help in the design of more efficient and effective traffic control plans for construction zones. The terminology suggested by Lewis (*1*) is used whenever possible to identify different locations in a traffic control zone (work zone). According to the terminology a traffic control zone (work zone) is divided into four areas: the advance warning, transition, activity, and termination areas. The activity area is further divided into two spaces: buffer space and work space. It should be noted that the work space is only one small part of a work zone.

It should be noted that initial speed may prove to be an important parameter, but the authors do not believe that speed in the work zone can fully be explained by initial speed alone. Other factors such as type and level of work activities, geometric characteristics, and law enforcement level would influence work zone speed.

STUDY APPROACH

The study approach is based on finding the speed profiles of vehicles in a construction zone and relating them to the speeds at the beginning of the transition area (*2,3*). The speed of a vehicle was monitored from the time that it entered the study section until the time that it exited it. Two video cameras were used to collect data as the vehicles traveled in the traffic control zone.

Study Site Description

The construction zone was located on a rural section of Interstate 57 near Mattoon, Ill. The highway has two lanes in each direction, but one lane in each direction was closed because of the construction. The traffic control zone was about 5.6 km (3.5 mi) long. The construction crew was repairing the bridge decks over State Route 16 and another bridge about 4.0 km (2.5 mi) to the south of Route 16. The crew was also doing patching, overlay, and shoulder reconstruction work on the ramps of Route 16 and I-57.

The speed limit inside the construction zone was 72 km/hr (45 mph) for all vehicles. However outside the work zone it was 105 km/hr (65 mph) for cars and 89 km/hr (55 mph) for heavy trucks. The traffic control plan (TCP) used in the work zone was one of the Illinois Department of Transportation's standard TCPs which is prepared according to the guidelines given in the *Manual on Uniform Traffic Control Devices* (*4*). Figure 1 schematically shows the signs used for traffic control in this work zone.

The study section contained one noticeable crest vertical curve that was approximately 854 m (2,800 ft) long. It started 122 m (400 ft) before the DeWitt Road overpass and ended 61 m (200

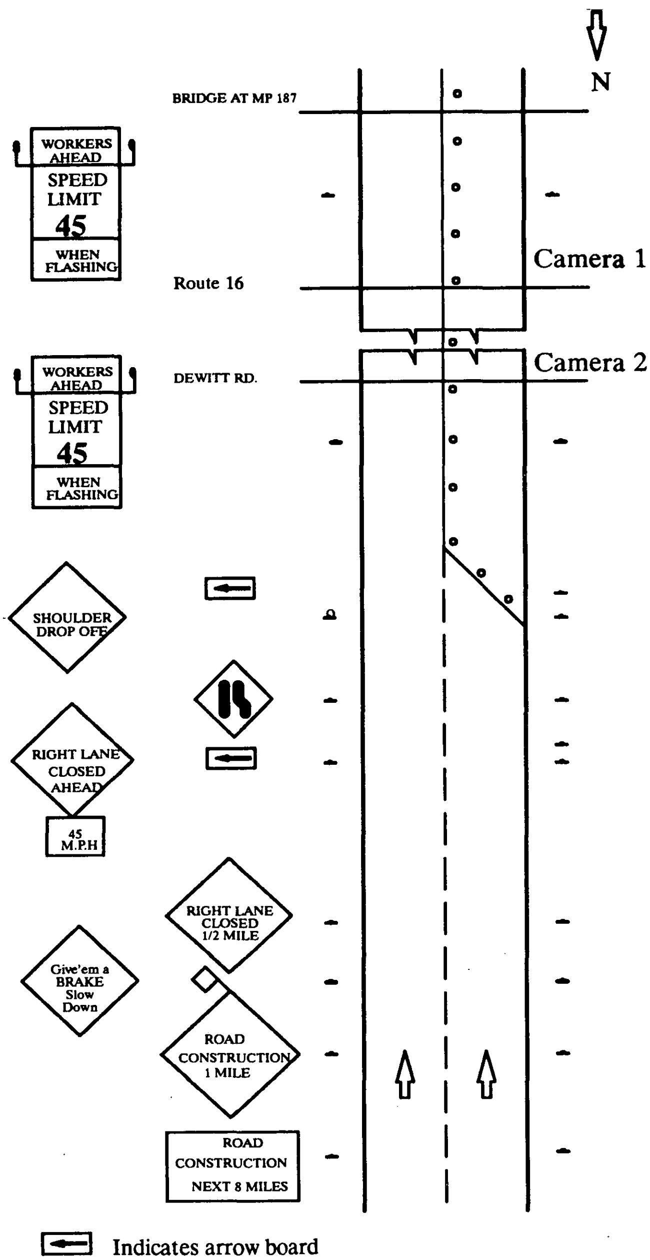


FIGURE 1 Signs on I-57 during speed profile study.

ft) before Route 16. Before the DeWitt Road overpass there is a very short section with a 3 percent upgrade slope. The speed reduction owing to the uphill section, if any, would be noticeable for trucks but not cars (*5*).

Data Collection

The study section was about 2.4 km (1.5 mi) long and was divided into smaller intervals of 122 to 214 m (400 to 700 ft) in length. The speed in each interval is represented by a speed station that is located approximately at the midpoint of the interval. There were 17 different speed stations. Data were collected during weekdays under normal weather conditions and when the construction crew was working. Vehicles that were in the free flow of traffic at the beginning of the study section were videotaped. Two video cameras followed each vehicle from the time that it entered the study section until the time that it exited it. A total of 208 vehicles were videotaped during the 3 days of data collection. The average daily traffic on this section of the freeway was about 12,000 vehicles, with approximately 22 percent being heavy commercial vehicles (*6*).

The speed of a vehicle in each interval was computed on the basis of distance and time information. The actual distance that a vehicle traveled between the two markers was computed by using data on the divergence angle and the locations of the markers from a camera location. The time that a vehicle took to travel this distance was obtained from the videotapes. Then the speed for that interval was computed as the ratio of the distance to time. For more detail on speed calculation, refer to Benekohal et al. (*2,3,7*).

Data Reduction

A vehicle was labeled as *influenced* if it was slowed down by another vehicle in front of it or exited from the ramp; otherwise it was labeled as *uninfluenced*. The uninfluenced vehicles were in free-flow traffic traveling at their desired speeds in the traffic control zone. Of 208 vehicles, 57 vehicles were tagged as influenced vehicles. The remaining 151 vehicles were labeled as uninfluenced. The findings of the present study are based on the speed characteristics of the uninfluenced vehicles. The uninfluenced vehicles were divided into two vehicle groups: the Autos group and the Trucks group.

The Autos group had 102 vehicles, which included 74 passenger cars and 28 vans and pickup trucks. The speed characteristics of the cars group were compared with those of the vans group, and it was determined that there were no significant differences. Thus the cars and the vans were combined into the Autos group. The Truck group had 49 tractor semitrailer trucks.

For each vehicle several sources of errors were identified and their effects on speed were calculated. In general the computed speed could be influenced by 1.6 km/hr (1 mph) or less as a result of these errors. For additional details on data collection and data reduction, refer to Benekohal et al. (*2,3,7*).

Influence Points

Throughout the construction zone there were traffic control signs and roadway features that may influence the speed of a vehicle. An influence point (IP) is defined as a location within the construction zone that may have such a sign or roadway feature. Thirteen IPs, labeled a through m, were identified in the study. The IPs and their distances from the beginning of the study section are listed in Table 1. The layout and traffic control plans in this work zone were typical of those for a one-lane closure on interstate highways. The speed of a vehicle at these IPs was determined by using the speed profiles.

Examination of the speed profiles indicated that the upgrade section did not significantly reduce the speeds of the cars. After the construction work was completed, adjustment data were collected to determine the speed reduction effects of the upgrade section (*2*). The mean speed reduction was 1.6 km/hr (1 mph) for

TABLE 1 IPs and Their Distances from Beginning of Study Section

INFLUENCE POINTS	LOCATION IN WORK ZONE	DISTANCE(ft)[a]
a	Beginning of the taper	600
b	End of the taper	1600
c	Before 1st speed limit signs	2100
d	At 1st speed limit signs	2600
e	After 1st speed limit signs	3100
f	Near the end of upgrade section	4300
g	1200 feet before Rt. 16 bridge	4800
h	600 feet before Rt. 16 bridge	5400
i	At Rt. 16 bridge (work space)	6000
j	500 feet after Rt. 16 bridge	6500
k	1000 feet after Rt. 16 bridge	7000
l	400 feet before 2nd speed limit signs	7900
m	Second speed limit signs and end of the study section	8300

[a] 1 foot = 0.305 meter (m)

cars and 8 km/hr (5 mph) for trucks. The speed changes were concentrated between −1.6 and +3.2 km/hr (−1 and +2 mph) for the Autos group and between −4.8 and −9.7 km/hr (−3 and −6 mph) for the Trucks group.

SPEED PROFILE PATTERNS

Certain speed reduction patterns are repeated by many drivers. The drivers were grouped into four general categories on the basis of their speed profile patterns. Figure 2 shows the typical speed profiles for the Autos group. The criteria for the classification were the visual examination of the speed change patterns and a quasi-quantitative measure of the speed change. Vehicles that showed similar speed profiles were grouped into one category. If the speed change over a significant portion of the study section was less than 8 km/hr (5 mph) for the Autos group and less than 6 km/hr

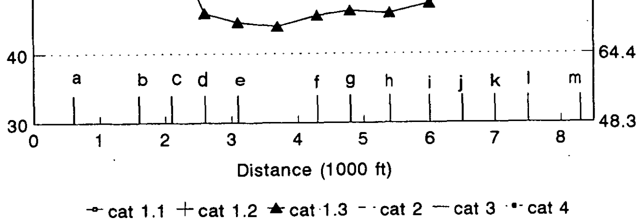


FIGURE 2 Typical speed profiles for Autos group, Categories 1 through 4.

(4 mph) for the Trucks group, it was attributed to expected speed fluctuation and was not used as a criterion in the classification. Brief descriptions of these categories are given below.

Driver Category 1

Category 1 represents the drivers who reduced their speeds at around the first set of speed limit signs. About 58 percent of Autos group and 68 percent of Trucks group drivers belonged to this category. Some drivers in this category reduced their speeds further at the work space (over the Route 16 bridge). The drivers in Category 1 are further divided into three subcategories.

Driver Category 1.1

Category 1.1 represents the drivers who decreased their speeds at around the first work zone speed limit signs and reduced their speeds further at the bridge. Usually the latter speed reduction was greater than the former one. Approximately 24 percent of Auto group drivers and 25 percent of Truck group drivers belonged to this category.

Driver Category 1.2

Category 1.2 represents the drivers who slowed down (sharply) at both the first speed limit signs and the bridge, but who increased their speeds between these two points. Their speed profiles resembled a W shape. About 23 percent of Auto group drivers and 20 percent of Truck group drivers were placed in this category.

Driver Category 1.3

Category 1.3 represents the drivers who reduced their speeds at around the first speed limit signs and kept traveling at the reduced speed until they passed the bridge. After passing the bridge some drivers in this group increased their speeds. About 11 percent of Auto group drivers and 23 percent of Truck group drivers belonged to this category.

Driver Category 2

The criteria for Category 2 were that the drivers traveled faster than the speed limit and had speed reductions of less than 8 km/hr for the Autos group and 6 km/hr for the Trucks group before IP e [152 m (500 ft) after the first speed limit signs]. This means that they basically ignored the first speed limit signs but began to slow down when they arrived at the bridge. Nearly 17 percent of Auto group drivers and 8 percent of Truck group drivers were placed in this category.

Driver Category 3

Category 3 includes those drivers who ignored both the first speed limit signs and the construction activities over the bridge. They drove through the work zone at almost a constant speed that was greater than the 72-km/hr (45-mph) speed limit. The Auto group drivers maintained a speed of approximately 97 km/hr (60 mph) or greater, and Truck group drivers maintained speeds of between 81 km/hr (50 mph) and 97 km/hr (60 mph). The speed fluctuation for this group was very small, 8 km/hr (5 mph) or less. About 11 percent of Auto group drivers and 10 percent of Truck group drivers were grouped in this category.

Driver Category 4

The fourth category is called *others*, which includes those drivers who could not be classified into Categories 1 to 3. Some of the drivers in Category 4 reduced their speeds at the first speed limit signs but did not slow down at the bridge. Some of them even increased their speeds while passing the work space. About 14 percent of Auto group drivers and 14 percent of Truck group drivers belonged to Category 4.

EFFECTS OF INITIAL SPEED ON SPEED REDUCTION

Concept and Motivation

Drivers travel at different speeds in advance warning areas, and their speeds there may influence the speeds that they maintain throughout the work zone. This paper discusses how the speed of a vehicle at the end of the advance warning area influences its speed in the work activity area. It also examines the correlation between the speed at the beginning of the lane closure taper and the speed inside the one-lane section of the work zone.

The speeds at the beginning of the taper are called *initial speeds* and are used to group the vehicles into four speed groups: Fast-, Faster-, Very Fast-, and Fastest-moving groups. The numbers of Auto group drivers in the Fast, Faster, Very Fast, and Fastest groups were 16, 33, 37, and 16, respectively. There were 12, 34, and 3 Truck group drivers in the Fast, Faster, and Very Fast groups, respectively. The Very Fast group had only three trucks; thus the characteristics of this group are not discussed in detail. The drivers in the Fast group had speeds of between 74 and 87 km/hr (46 to 54 mph) when they passed IP a (beginning of the taper). The drivers in the Fast group drove 2 to 14 km/hr (1 to 9 mph) over the speed limit of 72 km/hr (45 mph).

The drivers in the Faster group had initial speeds of 86 to 103 km/hr (55 to 64 mph); thus their speeds were 16 to 31 km/hr (10 to 19 mph) faster than the speed limit. The Very Fast group had initial speeds of 105 to 111 km/hr (65 to 69 mph). The Fastest group includes all the drivers who had an initial speed of 113 km/hr (70 mph) or greater at the beginning of the taper (IP a).

If speed reduction depends on the initial speed, do drivers in one speed group exhibit similar speed reduction profiles? If so then one needs to deal with one type of speeding driver in each speed group. Consequently the speed reduction efforts will mainly be targeted to these four types of drivers. However if drivers in one speed group belong to different driver categories, then it is more difficult to deal with this situation because 24 possible types of speeding drivers are involved (combinations of four speed groups and six driver categories and subcategories).

The drivers in one driver category were not distributed uniformly in these four speed groups. Moreover the drivers in one speed group were also not evenly distributed in the driver categories. This means that to some degree the driver categories are related to the speed groups. These issues are examined for the Autos and the Trucks groups in the following sections.

Speed Characteristics in Speed Groups

In general all speed groups exceeded the 72-km/hr (45-mph) speed limit, and different speed groups showed similar speed reduction trends. Figures 3 and 4 show the average values of the excessive speeds for the four speed groups for the Autos and Trucks groups, respectively. It is clear that the average speeds gradually decreased before the bridge (IP i) and increased after that for all speed groups. The trend for all speed groups shows that vehicles with higher initial speeds had greater speed reductions at the bridge.

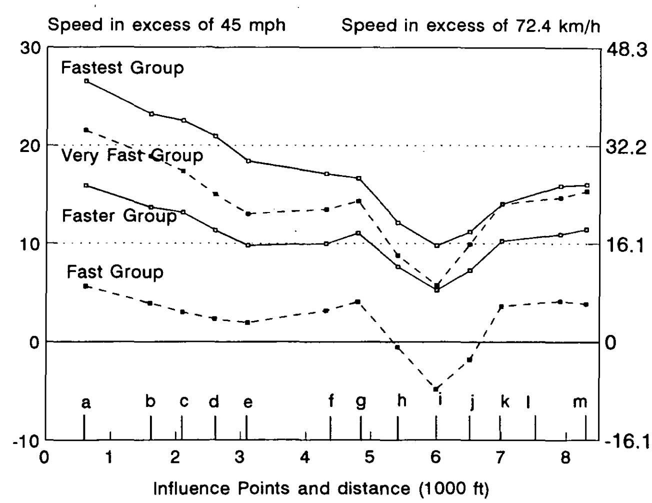
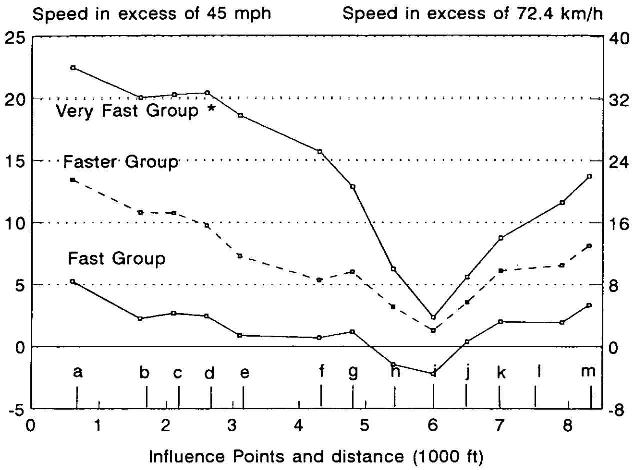


FIGURE 3 Average speed profiles for Autos group (1 ft = 0.305 m).



* Very Fast group had only 3 vehicles

FIGURE 4 Average speed profiles for Trucks group.

However throughout the work zone they still traveled at higher speeds than the vehicles with lower initial speeds.

Excessive Speeds and Ranges

The ranges for 90 percent of the excessive speeds for each speed group are shown in Figures 5 and 6 for the Autos and Trucks groups, respectively. For each speed group these boundaries were obtained by discarding the top and bottom 5 percent of the speed reductions at each IP. These ranges show that the upper bound of a range for a higher initial speed group is always greater than the upper bound of a range for a lower initial speed group. For example the upper bound of the Fastest group is always higher than the upper bound of the Very Fast group, the upper bound of the Very Fast group is always higher than the upper bound of the Faster group, and so on.

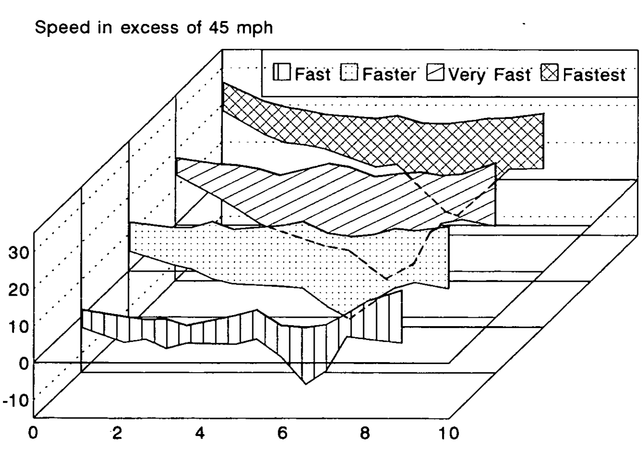
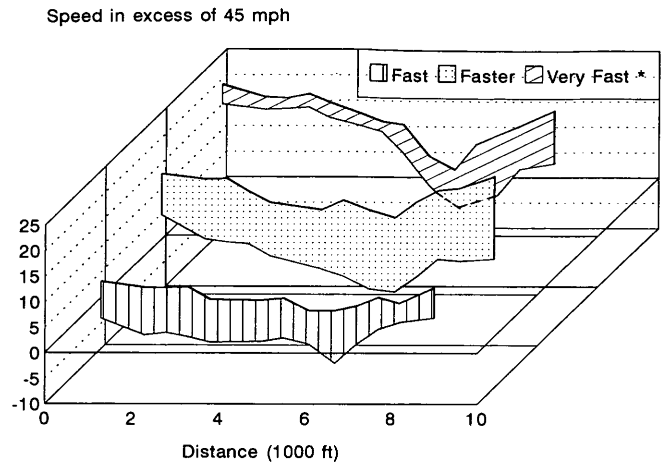


FIGURE 5 Speed bounds for 90 percent of observations in each speed group for Autos group (1 mph = 1.61 km/hr).



* Very Fast group had only 3 vehicles

FIGURE 6 Speed bounds for 90 percent of observations in each speed group for Trucks group.

The minimum speed reductions for each speed group decreased as the vehicles neared the work space. The minimum reductions for the Fast group were less than those for the Faster group. The reductions for the Faster and Very Fast groups were, for the most part, very similar, but they were less than the minimum reductions for the Fastest speed group.

This indicates that in general the vehicles in the higher-speed group would travel faster throughout the work zone than those in the lower-speed group. The ranges became wider as vehicles in all four speed groups got closer to the work space (the bridge). After passing the bridge the ranges were shrunk to almost the same level as those before the bridge.

Maximum Speed Reductions in Speed Groups

For each speed group the maximum speed reduction was computed by subtracting the lowest mean speed from the initial mean speed. For example drivers in the Fastest Auto group exceeded the speed limit by 42.7 km/hr (26.5 mph) at the beginning of the taper (IP a) and by 15.8 km/hr (9.8 mph) at the bridge, so the maximum speed reduction for this group was 26.9 km/hr (16.7 mph). The maximum speed reductions for the Fast-, Faster-, Very Fast-, and Fastest-moving Auto groups were 16.9, 17.1, 25.4, and 26.9 km/hr (10.5, 10.6, 15.8, and 16.7 mph), respectively. The maximum speed reduction increased as the initial mean speeds increased. This trend indicates that on average the vehicles with higher speeds at IP a reduced their speeds more than the vehicles with a lower initial speed.

The maximum speed reductions for the Fast- and Faster-moving Truck groups were 11.98 km/hr (7.44 mph) and 19.61 km/hr (12.18 mph), respectively. Drivers in the Faster Truck group exceeded the speed limit by 1.6 to 21 km/hr (1 to 13 mph), although the trucks in that group had a maximum speed reduction of 19.61 km/hr (12.18 mph). In comparison drivers in the Fast Truck group exceeded the speed limit by only 8 km/hr (5 mph) at the beginning and traveled 3 km/hr (2 mph) below the speed limit at the bridge, but had a maximum speed reduction of 11.98 km/hr (7.44 mph).

The drivers in the higher initial speed groups kept higher speeds in the work zone than the drivers in the lower initial speed groups, even though the former group had greater speed reductions than the latter group. The drivers in the Very Fast Auto group exceeded the speed limit by 10 to 35 km/hr (6 to 22 mph), although they had maximum speed reductions of 25.4 km/hr (15.8 mph). In comparison drivers in the Fast Auto group exceeded the speed limit by only 8 km/hr (5 mph) and traveled 8 km/hr (5 mph) below the speed limit at the bridge, but had a maximum speed reduction of 16.9 km/hr (10.5 mph), which is less than 25.4 km/hr (15.8 mph) reduction for the Very Fast group.

Driver Categories and Speed Groups

Drivers were classified into the combinations of speed groups and driver categories. Figure 7 shows the percentage of Auto group drivers in a given driver category who belonged to each of the speed groups. Figure 8 shows the percentage of Auto group drivers in each speed group who belonged to each driver category. The drivers in one driver category were not evenly distributed among the four speed groups. Similarly the drivers in one speed group were not evenly distributed among the driver categories. This seems to imply that some drivers are over- or underrepresented in certain speed groups or driver categories. These characteristics are examined in the following section.

Speed Group Distribution in Driver Categories

For each driver category the percentages of drivers who belonged to a speed group are shown in Figure 7. The sum of the percentages for each category in Figure 7 should be 100 percent. The great majority of the Auto group drivers who did not reduce their speeds in work zones belonged to the Faster and Very Fast groups. About 91 percent of the drivers in Category 3 belonged to the Faster and Very Fast groups, where the speed was as high as 111 km/hr (69 mph) and as low as 89 km/hr (55 mph), and most of them were driving faster than 97 km/hr (60 mph). The remaining 9 percent of drivers were all in the Fastest group, who drove faster than 113 km/hr (70 mph). Since these drivers traveled at almost constant speeds throughout the work zone, their speeds would need to be reduced before they enter the transition area. Thus to reduce the speed of this group the speed reduction stimulus, whatever it might be, should be located before the transition area.

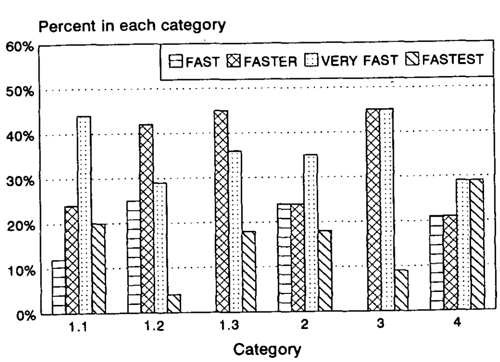


FIGURE 7 Proportions of speed groups in each driver category for Autos group.

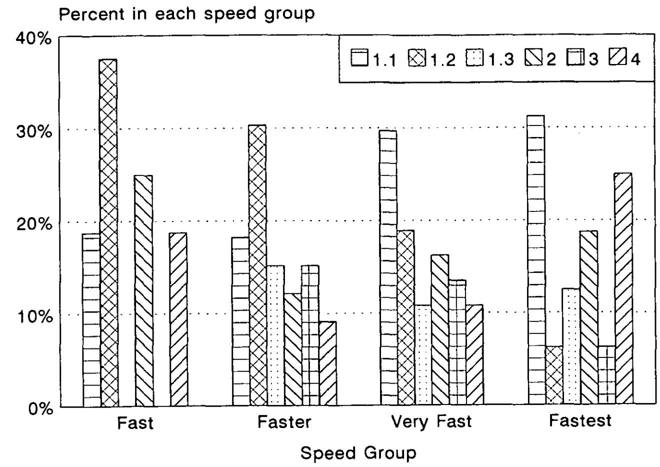


FIGURE 8 Proportions of driver categories in each speed group for Autos group.

The Faster and Very Fast groups dominated Driver Categories 1.2 and 1.3. Two observations can be made about them. First the speeds were high at the beginning of the transition zone even for a large portion of drivers in Categories 1.2 and 1.3. Second these drivers reduced their speeds in the work activity area because of some stimulus, mainly a roadway feature and traffic control devices. Thus if their speeds before entering the work activity area can be reduced, the additional speed reduction patterns that they have exhibited may bring their speeds to reasonable levels. Therefore these groups should be encouraged to reduce their speeds further before they get into the work activity area.

It should be noted that all speed groups are represented almost evenly in Category 4. This implies that those drivers who did not follow a particular speed reduction pattern did not come from a particular speed group.

Driver Distribution in Speed Groups

For each speed group the percentages of drivers in different driver categories are shown in Figure 8 (the sum of the percentages for each speed group is 100). In the Fast and Faster groups about one-third of Auto group drivers belonged to Category 1.2. However, in the Very Fast and Fastest groups about one-third of the Auto group drivers belonged to Category 1.1. This indicates that about one-third of the drivers who were extremely speeding (Very Fast and Fastest groups) reduced their speeds and kept reducing them as they traveled in the work activity area. However about one-third of those who were excessively speeding (Fast and Faster groups) reduced their speeds initially but increased them in the work activity area and then reduced them when they reached the work space.

It should be noted that the proportion of drivers in Category 1.1 increased as the initial speeds of the vehicles increased. Thus with a larger data set one can correlate the likelihood of having a given speed reduction profile to the initial speed of that vehicle. Also about one-fourth of the drivers in the Fastest group belonged to Category 4. This indicates that for one-fourth of the Fastest-moving vehicles the speed reduction pattern was unknown. They may be the main source of complaints by construction crews about "flying vehicles" in work zones.

Unlike the Autos group drivers, Category 2 Truck drivers almost did not exist in the Fast and Faster groups (there was only one driver in Faster Truck group). About 10 percent of Trucks in both speed groups were in Category 3 and basically traveled at constant speeds. A higher proportion of trucks in the Faster group than in the Fast group belonged to Category 1.1. This also supports the discussion presented earlier that the speed of vehicles should be reduced before they get in the work activity area.

Statistical Analyses

This section discusses the adequacy of sample size for studying the mean speeds and the results from analysis of variance and Tukey's multiple comparison tests. The sample size used in the present study provides enough accuracy for statistical analysis of the mean speeds. By using the standard deviations of the initial speeds [10.9 km/hr (6.8 mph) for the Autos group and 8.2 km/hr (5.1 mph) for the Trucks group], it was determined that the sample sizes used in the study would allow the mean speed of Autos and Trucks to be estimated to an accuracy of 2.30 km/hr (1.33 mph) and 2.14 km/hr (1.43 mph), respectively. This accuracy is adequate for the purposes of the present study, although a large sample size would increase the accuracy of finding the mean speeds. If more accuracy is needed data from a large sample should be collected.

Multiple comparison tests were run for all IPs. At each IP pairwise comparisons of average speeds were made. The multiple comparison tests were run to determine whether the average speeds were significantly different for a pair. Tukey's test was used because it is less conservative than Scheffe's multiple comparison tests and the experimentwise error rate is controlled (*8*). The results of Tukey's multiple comparison tests, with a confidence level of 90 percent, are presented in Table 2. Table 2 shows that speeds for most of the pairs were different except for three pairs around IP i (bridge). For those pairs similar speeds were expected to be seen near the bridge. The test results indicated that at most of the locations the speeds were different for various speed groups.

An attempt to correlate the driver categories to speed groups was made. This cross classification resulted in 24 combinations of driver categories and speed groups. A chi-square test was run to examine the distributions of drivers on these cells. However the observations in some of the cells were not adequate for such a detailed level of analysis. It was then decided to combine the Fast group with the Faster group (calling them *excessively speeding*) and the Very Fast group with the Fastest group (calling them *extremely speeding*). A chi-square test was run to compare the distributions of the two speeding groups into six driver categories. The results indicated that there is a correlation between driver category and the two speed groups. However the distributions of the drivers in the two speed groups and six driver categories were not statistically different than the expected values (chi-square, 5.0). This indicated that when drivers are grouped into 12 combinations there was not an under- or an overrepresentation of a speed group in a driver category.

TABLE 2 Multiple Comparison Test Results (Tukey's) for Mean Speeds of Different Speed Groups

IP	Speed Groups Compared						
	Autos						Trucks
	Fast-Faster	Fast-Very Fast	Fast-Fastest	Faster-Very Fast	Faster-Fastest	Very Fast-Fastest	Fast-Faster
a	No	No	No	No	No	No	No
b	No	No	No	No	No	No	No
c	No	No	No	No	No	No	No
d	No	No	No	No	No	No	No
e	No	No	No	No	No	No	No
f	No	No	No	No	No	No	No
g	No	No	No	Yes	No	Yes	No
h	No	No	No	Yes	Yes	Yes	No
i	No	No	No	Yes	Yes	Yes	No
j	No	No	No	Yes	Yes	Yes	No
k	No	No	No	No	No	Yes	No
l	No	No	No	No	No	Yes	No
m	No	No	No	No	No	Yes	No

Yes: Indicates average speeds are similar
No: Indicates average speeds are different

CONCLUSIONS

Vehicles were divided into Fast-, Faster-, Very Fast-, and Fastest-moving groups on the basis of their speeds at the beginning of the transition area. For all speed groups the average speeds gradually decreased before the work space and increased after passing it. The speed of a vehicle in a work activity area seems to be related to its initial speed. Vehicles with higher initial speeds in general reduced their speeds more than the vehicles with lower initial speeds; however, vehicles in the higher initial speed groups kept higher speeds in the work zone than the vehicles in the lower initial speed groups, even though the former group had larger speed reductions than the latter group.

Drivers were classified into the combinations of speed groups and driver categories. The drivers in a category were not distributed uniformly among the four speed groups. Similarly the drivers in a speed group were not distributed evenly among the driver categories. The data showed that the great majority of the drivers who did not reduce their speeds in work zones belonged to the Faster and the Very Fast groups. These drivers should be encouraged to further reduce their speeds before they get into the transition area.

About one-third of the drivers who were extremely speeding (Very Fast and Fastest groups) reduced their speeds and kept reducing them as they traveled in the work activity area. However about one-third of those who were excessively speeding (Fast and Faster groups) reduced their speeds initially, but increased them in the work activity area and then reduced them when they reached the work space.

RECOMMENDATIONS

An attempt was made to use a chi-square test to determine whether one driver category is under- or overrepresented in a speed group. However this could not be done statistically because of the large number of combinations of driver categories and speed groups. A larger sample size is needed for such a statistical analysis. However for certain combinations of driver categories and speed groups the data showed that a correlation exists between the initial speed of a vehicle and its speed profile. A much larger data set is needed to study this correlation for other combinations. Such a study is needed, and it will provide very useful information on traffic control in work zones. It is recommended that a large-scale study be conducted for this purpose.

ACKNOWLEDGMENTS

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Updated Capacity Values for Short-Term Freeway Work Zone Lane Closures

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Recommendations on estimating the capacities of short-term freeway work zone lane closures are presented. The recommendations are based on 45 hr of capacity counts at 33 work zones in Texas between 1987 and 1991. These new data indicate average capacities for short-term freeway work zone lane closures from three lanes to one lane and from two lanes to one lane that are significantly higher than older values reported in the 1985 *Highway Capacity Manual*. A base capacity value of 1,600 passenger cars per hour per lane is recommended for all short-term freeway work zone lane closure configurations. Adjustments are recommended for the effects of the intensity of work activity, the percentage of heavy vehicles in the traffic stream, and the presence of entrance ramps near the beginning of the lane closure. It is recommended that the new base capacity value and adjustments be used in lieu of the current procedures in the 1985 *Highway Capacity Manual*.

This paper presents new capacity values for short-term freeway work zone lane closures. The values are based on data collected in Texas between 1987 and 1991. It is recommended that the new capacity values presented herein be used in lieu of the older values in Chapter 6 of the 1985 *Highway Capacity Manual* (*1*), which were based on data collected in Texas during the late 1970s and early 1980s (*2,3*). The new values are higher than the older values, which has important implications for planning and scheduling work zone lane closures.

This introductory section includes a discussion of the uses of work zone capacity values and a review of previous research on work zone capacity. Then the new capacity values are presented and compared with the older values. Finally a recommended procedure for estimating work zone capacity is outlined.

USES OF WORK ZONE CAPACITY VALUES

Maintenance and construction projects should be conducted in a manner and at a time that minimize the total cost of the project. The two principal components of the total cost of a project are: (a) the costs of administering and performing the required work and (b) the increased road user costs associated with decreased levels of service through the work zone. In the past the first component was of primary concern. More recently, however, minimizing the adverse traffic impacts on motorists has become an important goal.

The traffic-handling capacity of a work zone is the principal determinant of the magnitude of the traffic impacts of a work zone on a given section of freeway during a given time period and given prevailing traffic demands. If the capacity exceeds the prevailing demand, then delays are likely to be minimal. When demand exceeds capacity, however, queues form and delays may be significant.

Demand-capacity analysis is an important step in planning and scheduling freeway work zone lane closures. The analysis may be performed manually or by computer. QUEWZ-92 is one computer program that performs such an analysis. The new capacity values presented herein have been incorporated into QUEWZ-92 (*4*).

PREVIOUS RESEARCH ON WORK ZONE CAPACITY

Chapter 6 of the 1985 *Highway Capacity Manual* (*1*) presents the best available work zone capacity values and outlines manual procedures for demand-capacity analysis of work zones. Most of the values presented are drawn from capacity studies conducted in Texas during the late 1970s and early 1980s (*2,3*). Capacity data are presented for both short-term maintenance work zones and long-term construction sites.

Short-Term Maintenance Work Zones

The previous capacity studies in Texas suggest that the capacity of a short-term maintenance work zone with a work crew at the site is most significantly influenced by the lane closure configuration. The configuration is designated [*A,B*], where *A* represents the normal number of lanes in one direction and *B* represents the number of lanes open through the work zone.

Figure 1 illustrates the observed work zone capacities for each lane closure configuration. Considerable variability is observed in the capacities for each lane closure configuration. This variability may be explained by differences in the type and intensity of work activity, the proximity of the work activity to traffic, traffic composition (percentage of heavy vehicles), the cross section of the traveled way (lane width and lateral clearance to obstructions), and the alignment (percent grade and degree of horizontal curvature). Insufficient data were available to quantify the effects of these factors, and therefore the capacity values presented in Figure 1 represent averages for a given configuration over the range of typical work, traffic, and geometric conditions.

Table 1 summarizes the average capacities in vehicles per hour per lane (vphpl) observed in Texas during the late 1970s and early 1980s on the basis of the datum points in Figure 1 (*1–3*). The capacities represent full-hour volume counts in a work zone lane closure while traffic was queued upstream of the lane closure.

Table 1 also includes the average percentage of heavy vehicles (i.e., trucks, buses, and recreational vehicles) and the calculated

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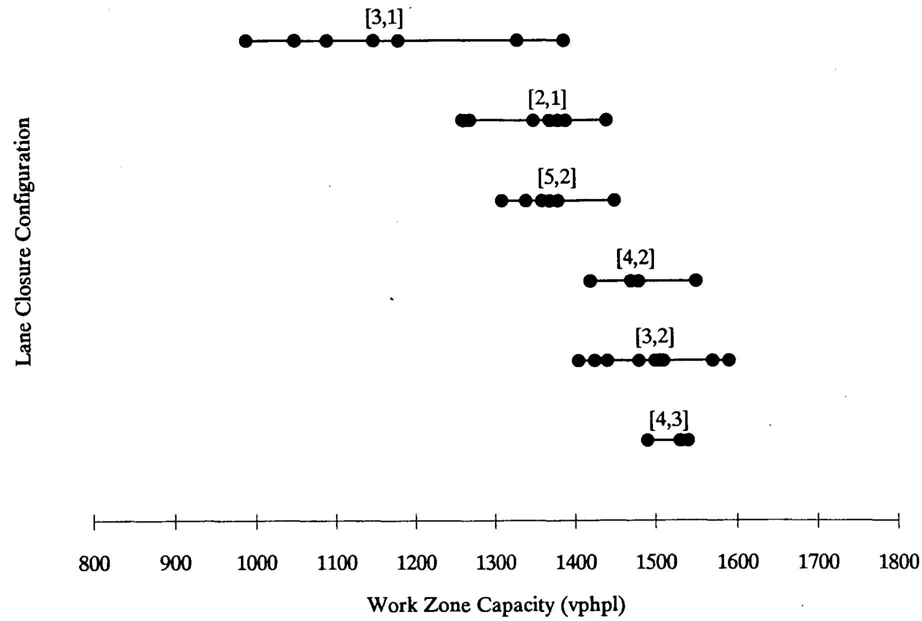

FIGURE 1 Previous data on short-term freeway work zone lane closure capacity.

average capacity in passenger cars per hour (pcph) per lane (pcphpl) for those configurations for which traffic composition data were available. The average capacities in pcphpl were computed by using a passenger car equivalent of 1.7 passenger cars per heavy vehicle, as recommended in the 1985 *Highway Capacity Manual* for trucks on freeway segments in level terrain. Adjustment for the percentage of heavy vehicles narrows the range of average capacities among the lane closure configurations.

Table 2, which is derived from Table 6-3 in the 1985 *Highway Capacity Manual* (*1*), combines California data collected during the late 1960s with the Texas data collected during the late 1970s and early 1980s (in parentheses). The California data were stratified by lane closure configuration and type of work. The California data "are only a guide since they were determined from a very limited amount of data . . . by taking several 3-minute counts after lanes are closed (under congested conditions)" (*5*). These data suggest, however, that the type of work affects the work zone capacity, but they are not sufficient to quantify the relationship between the intensity of work activity and the work zone capacity. In relative terms the reported capacities are lower for pavement marker placement and striping, which occur close to the open travel lanes, and for resurfacing and bridge repair, which typically involve more equipment and workers, whereas the capacities are higher for median barrier/guardrail installation/repair, which occur farther from the open travel lanes, and for pavement repairs, which typically involve less equipment.

A study of traffic characteristics at four [2,1] lane closures in Illinois also evaluated the effect of the intensity and location of work activity on mean speeds through a work zone (*6*). The results suggest that mean speeds through a work zone decrease as the intensity of work activity increases. Work intensity was quantified by using an index based on the number of workers, size of equipment, presence of flaggers, and noise and dust levels at the site. Mean speeds also decreased as the work activity moved closer to the travel lanes. A 3-km/hr (2-mph) drop in mean speeds was observed for every 0.9-m (3-ft) shift of work activity closer to the travel lanes.

TABLE 1 Previous Data on Short-Term Freeway Work Zone Capacity by Lane Closure Configuration (*1–3*)

Lane Closure Configuration [Normal, Open]	Number of Studies	Average Capacity (vphpl)	Average Percentage of Heavy Vehicles	Average Capacity (pcphpl)
[3,1]	7	1170	18.7	1320
[2,1]	8	1340	7.8	1410
[5,2]	8	1370	7.8	1450
[4,2]	4	1480	--	--
[3,2]	9	1490	6.6	1560
[4,3]	4	1520	--	--

TABLE 2 Previous Data on Short-Term Freeway Work Zone Capacity (vph) by Lane Closure Configuration and Type of Work (*1*)

Type of Work	Lane Closure Configuration (Normal, Open)				
	[3,1]	[2,1]	[5,2]	[4 or 3,2]	[4,3]
Median Barrier/Guardrail Installation/Repair	--	1500[1]	--	1600 (1470)[2]	1600 (1523)
Pavement Repair	1050	1400	--	1500 (1450)	1500
Resurfacing, Asphalt Removal	1050	1200 (1300)	-- (1375)	1300 (1450)	1333
Striping, Slide Removal	--	1200	--	1300	1333
Pavement Markers	--	1100	--	1200	1200
Bridge Repair	-- (1350)	-- (1350)	--	1100	1133

[1] California data reported by Kermode and Myyra (*5*)
[2] Texas data reported by Dudek and Richards (*2*)

Further evidence of the effect of the intensity of work activity is the reported capacity data in the 1985 *Highway Capacity Manual* (*1*) for one [4,2] work zone in Texas at which no work was under way in the lane adjacent to the open travel lanes, providing a buffer lane between the work activity and traffic. The capacity of this work zone was estimated to be 1,800 vphpl, which is considerably larger than the average value for [4,2] lane closures.

Other previously published data on short-term work zone lane closure capacity are limited. The only other known data are from an FHWA study for which capacities were measured at two [2,1] lane closures (*7*). Capacities of 1,060 and 950 vph were observed; these values correspond to approximately 1,160 and 1,060 pcph (using a passenger car equivalent for trucks of 1.7). These values were deemed unusually low because of "the very unusual equipment and construction operation in combination with the narrow travelway" at the site.

Long-Term Construction Zones

The 1985 *Highway Capacity Manual* (*1*) includes data on 10 long-term construction zones at which the work activity area is separated from traffic by portable concrete barriers. Table 3 summarizes those data. As with the short-term work zone capacities reported in the 1985 *Highway Capacity Manual*, the data for long-term construction zones were collected in Texas. Average capacities are reported in vphpl; the percentage of trucks was not reported, and therefore capacities could not be computed in pcphpl.

Capacity data were also reported in two FHWA studies for a limited number of long-term construction zones in which a crossover configuration was employed. One study reported capacities for two work zones at which the capacities were 1,450 and 1,550 vph in the crossover direction and 1,720 and 1,800 vph in the opposite direction (*8*). The other study reported capacities for five work zones ranging from 1,030 to 1,600 pcph in the crossover direction and 1,520 to 1,910 pcph in the opposite direction (*7*).

NEW WORK ZONE CAPACITY VALUES

This section summarizes the new data on short-term freeway work zone lane closure capacity that were collected in Texas from 1987 through 1991. First the data collection methodology is described. Next the new data are presented and are then compared with the older values reported in the 1985 *Highway Capacity Manual* (*1*). Finally a study of the effect of lane closure placement relative to entrance ramps is discussed.

Data Collection Methodology

The data reported herein represent more than 45 hr of capacity counts at 33 different freeway work zones with short-term lane closures. Data were collected for five different lane closure configurations: [3,1], [2,1], [4,2], [5,3], and [4,3]. More than 15 hr of data collected at eight additional sites were excluded from the analysis because they violated the requirements described below; they provided insights, however, into the capacity-reducing effects of nonideal conditions.

All sites at which data were collected were short-term lane closures. Most were maintenance work zones, although several were short-term, off-peak lane closures at long-term reconstruction projects. The most common types of maintenance activities at the work zones observed included pavement repairs, seal coating, and the placement or repair of concrete median barriers. All of the work zones were in general compliance with the *Manual on Uniform Traffic Control Devices* (*9*). Standard channelizing devices were used at the lane closures (i.e., traffic cones, drums, or vertical panels).

All capacity counts were taken as the vehicles entered the activity area through the transition area of the work zone by using the standard terminology recommended by Lewis (*10*). The count location is illustrated in Figure 2. Data were used only for time periods during which traffic was queued in all lanes upstream of the activity area. Therefore the capacity counts represent the rate at which vehicles discharge from the upstream queue, merge into the reduced number of lanes through the transition area, and enter the activity area. Sites at which ramps were located within the transition area or the upstream end of the activity area were not analyzed.

In previous work zone capacity studies, some capacity data were collected at points within the activity area (i.e., other than at the downstream end of the transition area and the upstream end of the activity area) where the traffic flow appeared to be the most constrained. At some such sites there were intervening ramps between the upstream end of the activity area and the capacity count location; in these cases the counts within the work zone would differ from the queue discharge rate entering the activity area by the volume of traffic entering or exiting at the intervening ramps.

In the present study, however, it was determined that capacity counts should be taken only at the upstream end of the activity area for the following reasons:

1. To achieve consistency in measurement among work zones,
2. To be consistent with the current general consensus on the definition and measurement of freeway capacity, and
3. To be consistent with the analysis assumptions of demand-capacity analysis.

Counting of the capacity only at the upstream end of the activity area eliminates the variability among sites because of differences

TABLE 3 Previous Capacity Data for Long-Term Construction Zones with Portable Concrete Barriers (*1,3*)

Lane Closure Configuration	Number of Studies	Average Capacity (vphpl)
[3,2]	7	1,860
[2,1]	3	1,550

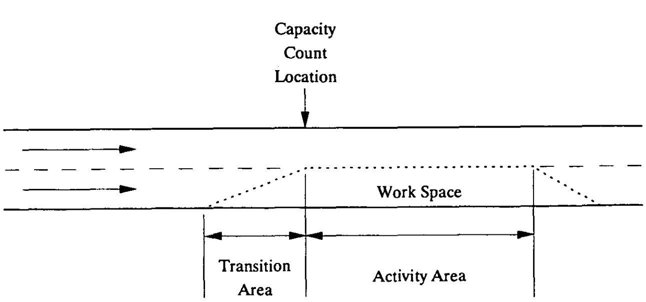

FIGURE 2 Work zone capacity count location.

in the number and traffic volumes of ramps within the work zone. Although sufficient data are not available to test for statistical significance, it appears that entrance ramps within the transition area or the upstream end of the activity area reduce both the queue discharge rate entering the activity area from upstream and the merging capacity of the entrance ramp. In this paper it is therefore recommended that the base capacity value represent conditions in which the impacts of ramps are negligible and that the effect of ramps be treated separately.

Debate on the definition, measurement, and value of freeway capacity has heightened in recent years as work progresses toward a new edition of the *Highway Capacity Manual*. Currently the general consensus appears to be that freeway capacity should be defined and measured as the mean queue discharge rate entering a freeway bottleneck (*11*). The mean queue discharge rate over an extended continuous time period (e.g., 1 hr or more) or for multiple time intervals over several days is recommended. This definition can be applied directly to freeway work zone lane closure capacity and was adopted for the present study. A similar definition was used in the previous capacity studies in Texas (*2,3*), in that full-hour volumes were used as capacity values. The principal difference between the new data and those from previous studies in this regard is the treatment of sites where several hours of capacity counts were taken. In the previous studies each hourly volume was considered a separate capacity study or observation, whereas the new capacity values were taken as the average flow rate during the entire period (i.e., several hours of capacity counts at a site were averaged and considered as one capacity study or observation). This approach is more consistent with the current definition of capacity and more appropriate for statistical analysis purposes than the previous approach.

A principal use of the capacity values recommended herein is as an input to demand-capacity analysis. Therefore it is imperative that the values be consistent with the assumptions of that analysis. A work zone lane closure would typically be modeled as a simple bottleneck with all traffic entering at the upstream end and exiting at the downstream end. The demand would be the traffic flow rates approaching the bottleneck from upstream, and the capacity would be the rate at which that demand can enter the upstream end of the bottleneck. Therefore the capacity used in the analysis should be the rate at which vehicles can enter the upstream end of the activity area.

Observed Capacities

The new capacity data for short-term freeway work zone lane closures are presented in Table 4. A comparison of Table 4 with the corresponding older values in Table 1 indicates that for the [3,1] and [2,1] lane closure configurations the averages (in both vphpl and pcphpl) for the new data are significantly higher than those for the old data (on the basis of a t-test at a .05 significance level). For the other configurations the averages of the old and new data are not significantly different.

The average capacities for the five lane closure configurations for which new data are available range only from 1,588 to 1,629 pcphpl—a difference of only 41 pcphpl. When the statistical procedure analysis of variance was performed on the data summarized in Table 4, the results indicated no statistically significant differences among the average capacities in pcphpl for the five lane closure configurations (at a .05 significance level).

The overall average capacity (for all lane closure configurations combined) is approximately 1,600 pcphpl. This value compares logically to the capacities of 2,200 pcphpl for freeways and multilane highways and of 1,900 pcphgpl (passenger car per hour per green per lane) for signalized intersections, which represent the queue discharge rate and saturation flow rate under ideal conditions for the corresponding facility type.

The peak hour factor is the ratio of the hourly capacity divided by the highest 15-min flow rate. The relatively high average peak hour factors (ranging from 0.92 to 0.96) suggest that although some variability exists at a site over time, the average capacities are reasonably stable.

Figure 3 illustrates the range among the capacities observed at individual work zones. Across all lane closure configurations, capacities ranged between 1,414 and 1,741 pcphpl (except for one value of 1,913 pcphpl). These data, together with observations from previous studies, suggest that factors contributing to below-average capacities include unusual or unusually intense work activities and the presence of ramps within the taper area or immediately downstream of the beginning of the lane closure. These factors distract the driver and complicate the driving task more than the "average" work zone and, as a result, reduce the efficiency of traffic flow. Unfortunately the available data are not sufficient to quantify the magnitudes of these factors' capacity-reducing effects.

Effect of Lane Closure Placement Relative to Entrance Ramps

The capacity of a work zone is typically limited by the efficiency with which vehicles can discharge from the upstream queue,

TABLE 4 New Data on Short-Term Freeway Work Zone Lane Closure Capacity

Lane Closure Configuration (Normal, Open)	Number of Studies	Average Capacity (vphpl)	Average Percentage of Heavy Vehicles	Average Capacity (pcphpl)[1]	Average Peak Hour Factor
[3,1]	11	1460	12.6	1588	0.92
[2,1]	11	1575	4.9	1629	0.94
[4,2]	5	1515	9.8	1616	0.92
[5,3]	2	1580	2.0	1601	0.93
[4,3]	4	1552	4.3	1597	0.96
All	33	1536	8.0	1606	0.93

[1] Calculated using a passenger car equivalent for heavy vehicles of 1.7.

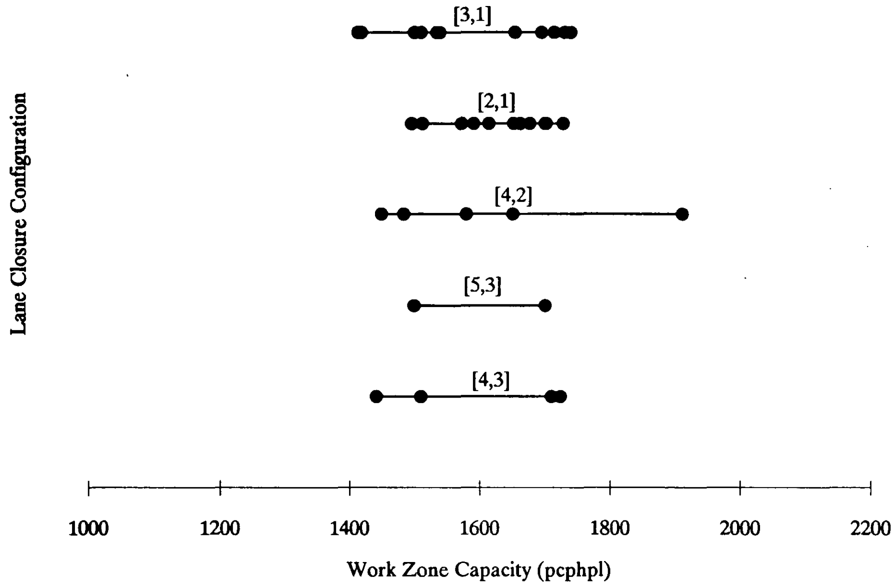

FIGURE 3 New data on short-term freeway work zone lane closure capacity.

merge into the reduced number of travel lanes through the transition area, and enter the activity area. Ramps within a work zone, especially entrance ramps in the transition area and the upstream end of the activity area, create additional turbulence that further reduces the efficiency with which the traffic stream can enter the work zone.

In some cases the most constrained point within the work zone with respect to traffic throughput may be some distance downstream of the beginning of the activity area. Examples include a high-volume entrance ramp or an unusual or unusually intense work area within the lane closure that causes queuing upstream, beyond the beginning of the lane closure. These cases are not treated here.

Traffic delays through work zones can be minimized by maximizing the total vehicular throughput of the work zone. Vehicles may enter the work zone either from the upstream end or through entrance ramps within the work zone.

Although sufficient data are not available to test for statistical significance, three general observations can be made on the basis of the new data and those from previous studies about the effects of lane closure placement relative to entrance ramps on the total vehicular throughput of a work zone:

1. Even though vehicles may be entering the beginning of a lane closure at its capacity, as vehicles release from the queue and accelerate at varying rates through the lane closure, gaps develop in the traffic stream. These gaps are large enough for additional vehicles to enter the lane closure from entrance ramps downstream from the beginning of the lane closure.
2. Entrance ramps either within the transition area or a short distance into the activity area appear to reduce the queue discharge rate entering from upstream (because of the turbulence created by ramp vehicles forcing their way into inadequate gaps in the main lane traffic stream), whereas entrance ramps farther downstream within the activity area appear to have less of an impact.
3. Fewer vehicles can enter the work zone from an entrance ramp near the upstream end of the activity area without disrupting the main lane traffic stream (because of the uniformity of the headways in the traffic stream near the queue discharge point) than from a ramp farther downstream (where the traffic stream is more dispersed).

These observations suggest that the location of the channelizing taper and beginning of the lane closure relative to entrance ramps influences the total throughput of the work zone. In some situations there is flexibility to adjust the location of the beginning of the lane closure in a manner that can increase total vehicular throughput. Therefore a detailed study was undertaken at one work zone to analyze the effects of the placement of freeway work zone lane closures relative to entrance ramps on the basis of ramp merging capacity.

The basic approach was to study the distribution of headways at the beginning of the lane closure and at various points within the work zone [152, 305, and 457 m (500, 1,000, and 1,500 ft) downstream from the beginning of the lane closure]. Gap acceptance procedures developed by Drew (*12*) were applied to estimate the entrance ramp merging capacity on the basis of the observed headway distributions. This section summarizes the study results. Lopez (*13*) provides complete documentation of the study.

Although the mean time headway (the inverse of the flow rate) within a lane closure remains constant, unless or until the flow rate changes as a result of entering or exiting traffic, the distribution of headways changes as vehicles in the traffic stream release from the queue and accelerate at varying rates. At the work zone studied the headways at the beginning of the lane closure and 152 m (500 ft) downstream from the beginning were relatively uniform (as evidenced by a large K parameter for the best-fitting Erlang distribution), whereas by a point 305 m (1,000 ft) downstream headways approached a random distribution (as evidenced by a K parameter approaching 1 for the best-fitting Erlang distribution). For a given mean time headway a more random distribution has more individual headways large enough for ramp vehicle mergers into the main lane traffic stream without disrupting it. Conversely a more uniform headway distribution has fewer such individual headways.

These findings regarding the change in headway distributions within a work zone support the three observations stated earlier. Near the beginning of a lane closure there are fewer headways large enough for ramp vehicle mergers without disrupting the main lane traffic stream, whereas farther downstream there are more headways adequate for use by entrance ramp vehicles. More entrance ramp vehicles can enter a work zone with less disruption to the main lane traffic stream (and therefore with less of an effect on the queue discharge rate entering from upstream) at an entrance ramp farther downstream of rather than closer to the beginning of the closure.

Therefore if conditions permit it is desirable to locate the lane closure such that any entrance ramps within the activity area are as far as possible [preferably at least 152 m (500 ft)] downstream from the end of the transition area while at the same time avoiding ramps within the transition area.

RECOMMENDED CAPACITY ANALYSIS PROCEDURE

This section recommends a procedure for estimating the capacities of short-term freeway work zone lane closures. The procedure includes a base capacity value and a series of adjustments to that base value.

Recommended Base Work Zone Capacity Value

The new capacity data suggest that it would be appropriate to use the overall average capacity of 1,600 pcphpl as the base capacity value for short-term freeway work zone lane closures, regardless of the lane closure configurations. This value is based on work zones whose traffic control is in compliance with the *Manual on Uniform Traffic Control Devices* (*9*).

The recommendation of a single base capacity value departs from previous procedures that recommended a different base value in vph for each lane closure configuration. The new data, however, indicate that after adjusting for the percentage of heavy vehicles there were no statistically significant differences among the average capacities of the five lane closure configurations observed. The use of a single base value is also consistent with the other procedures in the 1985 *Highway Capacity Manual* (*1*). Furthermore the value of 1,600 pcphpl relates logically to the base capacity values used in those procedures.

Adjustments to Base Work Zone Capacity Value

The recommended base value of 1,600 pcphpl represents the average of all recently observed work zone capacities. Figure 3 illustrates the fact that the capacities of individual work zones fell within a range of approximately ±10 percent of 1,600 pcphpl. Therefore when certain conditions are present the base capacity value should be adjusted for better predictions. Recommendations are made on adjustments for the intensity of work activity, the effects of heavy vehicles, and the presence of entrance ramps.

Adjustment for Intensity of Work Activity

The new data suggest that work zone capacity decreases as the intensity of work activity increases. Work zone capacity also may be decreased when the type of work activity is unusual and causes more rubbernecking than a more common activity. The intensity of work activity increases with the number and size of work vehicles, the number of workers, the magnitude of noise and dust, and the proximity of work to the open travel lanes. In Table 2, for example, capacities were lower than average for work that occurs close to the open travel lanes and that involves more and larger equipment and workers, whereas capacities were higher than average for work that occurs farther from the open travel lanes and that requires less and smaller equipment.

Unfortunately the available data are not sufficient to quantify the relationship between the intensity of the work activity and the adjustment to the base capacity value. Therefore the only guidance that can be provided is to adjust the base capacity value up or down within the ±10 percent (160 pcphpl) range for work activities that are significantly more minor or more intense than average.

Adjustment for Effects of Heavy Vehicles

It is recommended that the heavy vehicle adjustment factors in the 1985 *Highway Capacity Manual* (*1*) be used to account for the effect of heavy vehicles. The heavy vehicle adjustment factor H is calculated as follows:

$$H = \frac{100}{[100 + P \times (E - 1)]}$$

where

H = heavy vehicle adjustment factor (vehicle/passenger car),
P = percentage of heavy vehicles, and
E = passenger car equivalent (passenger cars/heavy vehicle).

A passenger car equivalent of 1.7, which is recommended in the 1985 *Highway Capacity Manual* (*1*) for trucks on freeway segments in level terrain, was used to convert the observed capacity counts and percentage of heavy vehicles to capacities in pcphpl. Reference should be made to the *Highway Capacity Manual* (*1*) for passenger car equivalent values for rolling or mountainous terrain and for extended individual grades.

To test the appropriateness of the 1.7 value for work zones, passenger car equivalents were computed by using capacity count and time headway data for two [2,1] lane closures. The passenger car equivalent was computed as the ratio of the mean time headway for trucks divided by the mean time headway for passenger cars entering the activity area (*14*). At one work zone, which had a full-hour capacity count of 1,570 vehicles (1,490 passenger cars and 70 trucks), the passenger car equivalent value was 2.1. At the other work zone the full-hour capacity count was 1,657 vehicles (1,585 passenger vehicles and 72 trucks), and the passenger car equivalent was 1.8. These results, although based on a limited amount of data for only two work zones, suggest that the value of 1.7 is low but not unreasonable.

Adjustment for Presence of Ramps

In demand-capacity analysis care must be taken to adjust appropriately either demand or capacity for the presence of ramps. The upstream end of the channelizing taper should be used as the reference point for estimating both demand and capacity. That is, the demand used for analysis purposes should be the hourly volume of vehicles that attempt to enter at the beginning of the lane closure, and capacity is the hourly rate at which vehicles can actually enter.

Typically historical main lane volume data are used to estimate the approach demand volume. If there are ramps between the main lane count location and the beginning of the lane closure, the main lane counts should be adjusted by exit and entrance ramp volumes to estimate the main lane volume at the beginning of the lane closure.

Another issue that must be addressed in estimating demand is the percentage of normal traffic volumes that divert from the freeway in response to work zone-induced delays. QUEWZ-92, for example, has an algorithm for estimating diversion and adjusting demand accordingly. If the analysis is performed manually, demand volumes should be adjusted on the basis of local experience.

The work zone capacity (i.e., the rate at which the main lane queue upstream of the lane closure discharges into the work zone) appears to be affected by entrance ramps within the taper area or immediately downstream of the beginning of the full lane closure. It has been observed that headways near the beginning of the closure are fairly uniform. Therefore vehicles on entrance ramps near the beginning of the closure must force their way into the traffic stream, reducing the upstream main lane queue discharge rate into the work zone. Merging opportunities for entrance ramp traffic within the work zone increase, because the queue disperses and traffic flow becomes more random with increasing distance downstream of the beginning of the closure.

The available data are not sufficient to quantify precisely the magnitude of the effect on capacity as a function of ramp location and volume. As a conservative approximation, however, when entrance ramps are located within the taper area or within 152 m (500 ft) downstream of the beginning of the full lane closure, it is recommended that the work zone capacity be reduced by the average entrance ramp volume during the lane closure period, but no more than one-half of the capacity of one lane open through the work zone. This approximation assumes that each entrance ramp vehicle entering the work zone prevents one vehicle in the upstream main lane queue from entering the work zone. At high-volume entrance ramps one would expect main lane and ramp vehicles to alternate; therefore, the maximum adjustment for the presence of ramps would be one-half of the capacity of one lane open through the work zone.

If possible the work zone should be set up to avoid entrance ramps within the taper area or near the beginning of the lane closure and thereby avoid the capacity-reducing effects of those ramps. Data from one work zone suggest that traffic flows became nearly random within 457 m (1,500 ft) downstream of the beginning of the full closure. Adjusting the location of the beginning of a work zone lane closure such that the first entrance ramp is at least 457 m (1,500 ft) downstream from the beginning of the full closure should maximize the total work zone throughput (i.e., the sum of volumes that can enter the work zone from upstream and from entrance ramps within the work zone).

Calculation of Estimated Work Zone Capacity

The following equation, which combines the base capacity value and the recommended adjustments, can be used to estimate work zone capacity:

$$c = (1{,}600 \text{ pcphpl} + I - R) \times H \times N$$

where

c = estimated work zone capacity (vph),
I = adjustment for type and intensity of work activity (pcphpl),
R = adjustment for presence of ramps (pcphpl),
H = heavy vehicle adjustment factor (vehicles/passenger car), and
N = number of lanes open through work zone.

In review, the recommended values for the base capacity and the various adjustments are as follows:

I = range (−160 to +160 pcphpl), depending on type, intensity, and location of work activity;
R = minimum of average entrance ramp volume in pcphpl during lane closure period for ramps located within channelizing taper or within 152 m (500 ft) downstream of the beginning of full lane closure, or one-half of capacity of one lane open through work zone (i.e., 1,600 pcphpl/2N); and
H = given in *Highway Capacity Manual* (*1*) for various percentages of heavy vehicles and passenger car equivalents.

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Comparative Study of Glass Bead Usage in Pavement Marking Reflectorization

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Prior research and field experience with a variety of pavement marking materials and glass beads for marking reflectorization are examined. The use of large glass beads to enhance the wet-night reflectivities of pavement markings was investigated, together with binder characteristics, to provide effective service life estimates, optimum bead-binder combinations, and cost-effectiveness comparisons. A literature review and a state-of-the-art survey conducted in southern states revealed that the cost different between large and standard glass beads was low relative to the amount of increased wet-night visibility offered by large glass beads. However it is not known with certainty how the high pavement temperatures and climatic conditions indigenous to the South will affect the service lives of markings with large glass beads. Field testing of large glass beads on roads with a relatively large number of wet-night accidents could enable determination of the effective service lives of bead-binder-pavement type combinations as well as expected accident reduction benefits.

The longevity of pavement marking material has a direct effect on the cost of pavement marking maintenance and user safety. The estimated cost of marking streets and highways in the United States each year is approximately $475 million. This cost consists of about $380 million for materials and the remaining $95 million for their application. Dale (*1*) estimated that the quantities of marking materials used annually in the United States consists of the following:

- 37 million gal of traffic paint,
- 130,000 tons of glass traffic beads,
- 55,000 tons of thermoplastic marking materials, and
- $55 million worth of other materials such as preformed tapes, raised pavement markers, polyesters, epoxies, and adhesives.

This is a significant amount of material representing not only a large monetary effort but also an extensive allocation of human energy and application equipment. Adequate pavement markings are, however, one of the highest-payoff, lowest-cost operational improvements that can be made to streets and highways. The FHWA Pavement Marking Demonstration Program of the 1970s demonstrated that improved transverse and longline pavement markings were effective in improving motorist safety. The improvement in safety was determined to be especially prevalent during nighttime and low-visibility conditions when the pavement markings serve to delineate required vehicle paths. The desirable delineation effect of pavement markings is accomplished by the principles of retroreflectivity.

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Glass beads have been used to make pavement markings reflective for approximately 50 years. If properly embedded in a striping material, glass beads have the ability to collect incident light and reflect part of that light back to its source. It is this ability that makes spherical glass particles ideally suited to making pavement markings visible at night. The performance of the glass beads is dependent on their embedment depth and size and environmental conditions. Research conducted during the late 1960s indicated that the optimum embedment of the glass beads is 60 percent of the bead diameter. It was also noted that during periods of adverse weather small glass beads often become submerged under a film of water. Light from vehicle headlights bounces off the water surface and is lost. It was concluded that the retroreflective capabilities of beads were significantly reduced during rainy or foggy conditions. In addition the different types of marking materials have different effective service lives with regard to their abilities to hold the beads in place and retain their retroreflectivity.

Many changes have occurred in the pavement marking industry in the past 20 years, especially in the increased availability of polymeric nonshrink binders as durable marking materials. In addition to the advent of nonshrink binders, good water-based paints have been developed; these paints have a higher solid content than typical alkyd paints. Alkyd paints have also changed to comply with the requirement for short no-track time. The extended durabilities of these markings, their greater thicknesses, and recent enhancements to the glass bead surface to improve adhesion have resulted in the ability to use larger glass beads than was possible in the past. The advantage of large glass beads is that they break through the water film on the road surface during adverse weather conditions, thereby reflecting light from vehicle headlights and making road markings more visible to the driver. In addition the use of large glass beads in conjunction with the enhanced binding properties and increased thicknesses of the new marking materials has the potential to provide an increased effective service life.

OBJECTIVES

The advent of new binding materials and the recognition of nighttime wet pavement retroreflectivity problems have resulted in efforts by researchers, materials manufacturers, and governmental agencies to identify the most advantageous pavement marking types. This paper summarizes the results of a literature review and state-of-the-practice survey conducted for a study whose primary objectives were to determine which combination of pavement marking and glass bead types (a) provides the longest effective service life and (b) is effective in increasing wet-night retroreflectivity.

PAVEMENT MARKING CHARACTERISTICS

Prior studies have demonstrated that in addition to the durability of the binder itself its effective service life is determined by the ability of the binder to retain the retroreflective material. Determination of the most appropriate binder therefore requires consideration of the type of application (i.e., longline or transverse), glass bead retention ability, traffic volumes, pavement surface type, and total cost over its service life. This section summarizes the durabilities, reflectivities, and advantages and disadvantages of the predominant binder types.

Traffic Paint

Traffic paints have been the most widely used pavement marking material since their introduction in the early 1920s. Usually classified by drying time—that is, instant dry (less than 30 sec), quick-dry (30 to 120 sec), fast-dry (2 to 7 min), and conventional (more than 7 min)—traffic paints comprise paint vehicle (alkyd, modified alkyd, chlorinated rubber, or water base), a solvent, a pigment, and glass beads. The drying time is determined by the specific ingredients used.

Durability is dependent on material composition, weather, application purpose, traffic density, and the type and condition of the application surface. The major problems encountered with traffic paints relate to bonding, reapplication over existing markings, and softening of the pavement surface, resulting in "bleeding" or discoloration of the paint. Bonding problems common to both concrete and asphalt pavements may be caused by surface contamination and the moisture content of the substrate. The use of water-soluble solvents and high temperatures can sometimes counteract such problems. Bleeding may be reduced by using fast-drying paints on asphalt surfaces, but adhesion may be adversely affected if solvents evaporate too rapidly and trap air between the paint and the pavement, a problem known as "bridging."

The relatively low initial cost, well-established technology, ease of installation, and readily available application equipment ensure the continued widespread use of traffic paints. They provide good dry-night visibility and a choice of drying times and are relatively safe to handle. The reduced drying time reduces labor costs and decreases traffic delays and potential accidents related to installation.

Paints have the shortest lives of all marking materials and offer poor wet-night visibility. Year-round delineation with one annual application is difficult to achieve in regions with severe winter climates, particularly on high-volume roadways. Paints with accelerated drying times also require more expensive striping equipment and cleaner pavement surfaces for successful adhesion and durability than those required for their longer-drying counterparts.

Thermoplastics

Thermoplastics are thick pavement marking materials consisting of a resin binder, coloring agents, an inorganic filler, and reflective glass beads. Proportioned and mixed in a factory, thermoplastics can be transported to job sites as solid slabs or as granular powder. Most commercial thermoplastics today use a blend of synthetic hydrocarbon resins, although the use of alkyd-based resin may become more widespread as its price decreases.

Southern states report an average thermoplastic service life of 10 years. Some thermoplastic markings were observed to last the life of the pavement, whereas other applications did not last for 1 year under heavy traffic conditions in northern climates. Thermoplastic durability is considerably better on asphalt than on concrete pavements. The most common problems encountered in northern climates are abrasion, shaving, and bond failure. Abrasion and shaving are principally caused by snow removal equipment. Since material failure is not involved, the solution is to change plowing procedures or use thinner marking materials. Bond failure results from improper installation (inadequate heating, dirty or oily pavement) and is common on concrete. On asphalt abrasion is related to the volume of traffic and the incidence of abrasive materials and studded tires. The poor performance of longitudinal markings is usually attributed to the presence of dirty or deteriorating pavement at the time of marking application.

Thermoplastic forms a relatively durable reflectorized marking. Its initial appearance is generally excellent, and reflectivity is sustained throughout its service life. Thermoplastic's dry reflectivity is generally equivalent to that of beaded paint, but its reflectivity is comparatively better under heavy rain. The crucial factor that affects its effectiveness is the bead application rate, although reflectivity can be diminished by dirt and plow damage. In a study conducted by the Texas Transportation Institute, reflectivity (on skip lines with or without primer) was excellent on asphalt after 2 years but was negligible on concrete because of bond failure.

Thermoplastic has an advantage over paint when year-round painting is not possible and when wet-night visibility is important. However it is a poor choice for transverse lines in areas with high traffic volumes and for longitudinal lines when turning traffic is common. Because of their thicknesses, thermoplastic markings are not suitable for use in regions with severe winter conditions because of their susceptibilities to snowplow damage.

Preformed Materials

Preformed markings are composed of high-quality plastics, pigments, and glass beads. These materials are applied to the pavement surface either by pressure or by heat. The beads are uniformly distributed throughout the film and form a firmly bonded layer on the surface.

The durability of a preformed marking depends primarily on pavement conditions and the number of pieces used. Durability is poor on old and deteriorating pavements, and especially on concrete. Furthermore since these materials can slide or shift, enhanced performance may be obtained by minimizing the number of pieces of preformed tape. One study in Kentucky reported a 4-year average useful life for preformed materials, although manufacturers guaranteed only 2 years for inlaid markings and 1 year for overlaid markings in snowbelt regions (*2*). After 2 years preformed markings in Virginia retained a satisfactory appearance and served adequately at (a) several urban intersections, (b) a section of an interstate highway where the tapes had been applied during resurfacing, and (c) on a highway with annual average daily traffic (AADT) of 25,000 and a high volume of turning traffic. State officials considered 3 years to be a conservative estimate of the life of preformed materials and predicted an 8-year life for at least one area.

The following are advantages of preformed marking materials:

- Their durability eliminates the need for annual or semiannual maintenance even in snowbelt regions.
- Installation is simple, safe, and clean.
- Their appearance and initial retroreflectivity are rated five or six times better than those of paint.
- Preformed tapes meet all of the requirements for color and conform to the *Manual on Uniform Traffic Control Devices.*
- Preformed tapes adhere well, especially when they are laid on new pavements.
- They are easy to repair because pieces adhere well to each other.
- They eliminate the need for major traffic disruption during installation.

Epoxy

Epoxy is a solid, two-component, chemically reacted system. It is safe to both handle and apply since it has no solvents that can evaporate and requires low heat. Epoxy is abrasion resistant and durable and adheres well on both asphalt and concrete.

Epoxy's durability has proven to be good to excellent in several tests in Minnesota. In one test epoxy lasted for over a year on roads with high AADT, in comparison with 3 months or less for traffic paint, and was effective even 5 years later. At present durability of 3 years has been proven on roads with low AADT. Failures occur when pavement surface conditions are poor, when large volumes of crossing or weaving traffic exist, and when application or quality control requirements are not properly followed. Failures associated with epoxy are most likely to be one of the following:

- Chipping caused by surface contamination or poor temperature control, which is apparent within days.
- Color change caused by the lack of pressure control, improper mixing, or improper bead application.
- Wheel tracking resulting from incorrect bead application.

Under low- to medium-AADT conditions epoxy retroreflectivity is excellent when new and is still acceptable after 3 years. Although daytime delineation has been found to drop after 2 years, nighttime delineation was more than adequate even after three northern winters. Epoxy is safe to handle and apply and has good color and bead retention, excellent retroreflectivity, good abrasion resistance, and good adhesion on both asphalt and concrete. Epoxy can be applied on damp pavement and is applied like paint.

The disadvantages associated with epoxy are mostly related to installation procedures and equipment. They include the following:

- The material is unforgiving; that is, there is no room for sloppy workmanship.
- Control of the pumping system is critical; with present formulations temperature control during application is critical.
- Placement in urban, low-speed situations must be protected to prevent tracking.
- Special application equipment is required, although not as much as is required for thermoplastics.
- Placement and drying times are long.

Polyester

Polyester is a two-component thermosetting material consisting of a resin and a catalyst. The resin resembles standard traffic paint, and the catalyst is usually on organic peroxide, methyl ethyl ketone peroxide (MEKP). MEKP must be handled with care because it can cause burns and its fumes are dangerous. Polyester has a long drying time and can be applied over old paint.

One study stated that one application of polyester lines rated equal in color to painted lines after 3 years, although the paint had been applied three times (*2*). In another study the polyester line on a highway with an AADT of 20,000 appeared grayer in color than paint during the daytime, but it was superior to paint at night and remained effective for 8 years. Polyester has been successful when used for center and lane lines and has few reported problems when applied over old paint. Tests show that polyester lines applied at thicknesses of 7.5 and 15 mils are equally durable. On the basis of experience with polyester in Ohio, the following service lives can be expected when polyester is applied on good asphalt pavement:

- 3 to 4 years, centerline, AADT of up to 10,000,
- 1 year, centerline, AADT of up to 10,000 with heavy trucks and curves, and
- 3 to 4 years, lane line, AADT of up to 24,000.

The biggest problem with polyester markings is abrasion. In addition bond failure can occur if polyester is applied over an emulsion seal because of tracking, poor weather, oily asphalt, or poor equipment. The "Swiss cheese" effect, which occurs on oily asphalt if polyester material is applied too soon after paving, can be avoided either by waiting for 2 weeks after paving is completed or by first striping with fast-dry paints.

The use of 15-mil wet thickness and 16 to 20 lb of standard drop-on beads per gal (1.9 to 2.4 kg/L) of polyester material provided good reflectivity in one 3-year study and superior reflectivity for 8 years in another study. In addition to performing consistently well for more than 3 years, polyester material does not require more care than standard traffic paint or a minimum pavement temperature for application. It is low in cost and can be applied over old paint.

Polyester has several disadvantages: poor performance on concrete, bond failure because of abrasion, and long drying time. Also since the resin-catalyst mixture does not stabilize immediately, some time must be allowed before striping begins. The application equipment can be troublesome, and one must handle the catalyst cautiously and use protective goggles and gloves. For the best results the air temperature must be 50°F and application must wait for 2 weeks after paving unless a primer is first applied.

Epoflex

Epoflex is an epoxy thermoplastic material consisting of a binder (60 percent solids, 40 percent liquid resin), pigment, a calcium carbonate filler, and premixed glass beads to provide continuous retroreflectivity as the material wears.

Epoflex has provided satisfactory durability in field trials on both asphalt and concrete pavements in California, Colorado, Minnesota, and Texas (2). In most cases it is many times more durable than paint and is at least twice as good under test conditions in which the AADT was 27,000, there was a high volume of trucks, and studded tires were used. Epoflex applied over existing paint endured as well as that applied on bare pavement. The following conclusions about epoflex were based on these tests:

- Its service life is equivalent to that of 10 applications of paint on concrete and asphalt pavements in warm climates (no snowplowing) under both moderate and heavy traffic conditions.
- The service life of epoflex on asphalt is twice its service life on concrete in cold climates and with moderate traffic conditions.

On a commuter route with an AADT of 42,000 epoflex demonstrated excellent bead retention and no discernible wear after 2 years, whereas similar applications of traffic paint showed clear signs of deterioration. After a year in Minnesota, which included a severe winter, epoflex still provided satisfactory day and night delineation. Glass beads are dropped on during application to provide the initial retroreflectivity, whereas premixed beads ensure the continuance of retroreflectivity. Specially treated chemical-resistant beads provide better bead retention.

The major advantages of epoflex are its lack of a track time (less than 5 sec), lack of volatile components, low cost, and simplicity of formulation. It provides an extended service life and good reflectivity on both asphalt and concrete and can be applied at temperatures down to freezing. Epoflex has three major disadvantages: the high installation temperature required, its incompatibility with existing striping equipment, and the precise timing required for drop-on bead application.

LITERATURE REVIEW

A literature review was conducted to trace the development of bead-binder matching and bead gradation selection and to identify previous studies of the nighttime wet surface effectiveness of various pavement marking materials. The studies reviewed here represent prior research and experience with various binders, bead types, and bead sizes.

Pocock and Rhodes (3) investigated the principles of glass bead reflectorization and studied the effects of application procedures, bead gradations, and the refractive index of the beads on retroreflectivity. That study indicated that an advantage of using small glass beads is that they provide greater reflective area per pound than larger beads. The study did not, however, consider the lower wet-night reflectivity that results from small glass beads submerged under a film of water.

Dale (4) investigated the effectiveness of a silicone-based bead surface layer in improving roadway delineation under both dry and wet conditions. The silicone treatment resulted in reduced clogging of bead-dispensing equipment as well as reduced bead overembedment into the pavement marking material. That study also determined that the optimum depth of bead embedment for retroreflection ranged from 55 to 60 percent of the bead diameter. Lower embedment depths resulted in the premature loss of adhesion between bead and binder, whereas greater depths resulted in the loss of retroreflective efficiency. Glass beads larger than those currently used were suggested as a way of overcoming the problem of the loss of retroreflectivity during wet conditions, and it was noted that the thickness of the paint film would need to be increased to provide the necessary binding depth for the beads.

In subsequent work Dale (5) questioned the appropriateness of the bead gradations used for drop-on application. He concluded that the use of various bead sizes in a constant thickness of paint results in the provision of efficient retroreflectivity by only a small percentage of the beads. The rationale behind the use of a gradation of mixed sizes is that pavement marking paint gradually fails by abrasion. As the paint wears away smaller beads are continuously exposed, thus providing sustained retroreflectivity. However it was noted that paint often does not fail by abrasion but instead chips away. In that case a smaller quantity of large beads could provide higher retroreflectivity at considerable cost savings over that provided by larger quantities of different-sized beads.

An NCHRP Synthesis (6) evaluated various pavement marking materials and how the method of application affected their serviceability. The authors discussed the difficulty of deciding the optimum bead gradation because of wide variations in environmental conditions, application methods, and control of materials. The survey results reported herein indicated that two states preferred the use of premixed beads in paint because of its convenience of use and the uniform distribution of beads in the paint film. However 80 percent of state agencies used drop-on beads because of lower nozzle wear, faster drying time, and the decreased need for paint agitation. The survey determined that the predominant bead gradation used by states was U.S. sieves No. 30 to No. 80.

A research report by the Organization for Economic Cooperation and Development (OECD) (7) indicated the conditions and factors that should be considered in the selection of appropriate materials for various circumstances. An embedment of approximately 60 percent of the bead diameter for optimum retention and reflectivity with a bead gradation of between U.S. sieves 40 and 80 (0.42 to 0.177 mm) was suggested.

Gillis (8) reported on a study that evaluated epoxy, polyester, and thermoplastic resins as pavement marking materials. The study demonstrated that epoxy markings provided adequate delineation for both day and night conditions more than 2 years following installation. It was also found that epoxy provided better retroreflectivity than both paint and thermoplastic markings.

McGrath (2) summarized evaluations of six durable pavement marking materials. Bead gradations of U.S. sieves 20 to 80 were used in thermoplastic markings, which provided excellent reflectivity on asphalt after 2 years but resulted in poor reflectivity on concrete because of bond failure. Preformed materials performed satisfactorily for 4 years, epoxy performed satisfactorily for 2 years, polyester performed satisfactorily for up to 8 years, and epoflex performed exceptionally well even with high traffic volumes and warm climates.

A 1988 NCHRP Synthesis (1) summarized pavement marking needs, different types of pavement marking materials, and methods of preparing the pavement surface before marking. That study investigated two types of paint (solvent based and water based), thermoplastics, thermosets (polyester and epoxy), and marking tapes. The cost-effectiveness of the different pavement marking types was discussed, and life expectancy curves were provided for markings on both asphalt and concrete pavements.

O'Brien (*9*) performed a laboratory investigation of the embedment characteristics of drop-on moisture-proofed and uncoated glass beads and their associated retroreflectivities in combination with various types of hot-applied thermoplastic markings. The study concluded that moisture-proofed drop-on beads give excellent retroreflectivity and that the retroreflectivity of the standard bead gradation was enhanced by increasing the proportion of larger beads.

Kalchbrenner (*10*) described tests of large beads (VISIBEADs) in pavement markings conducted by Potters Industries. Demonstration projects were set up in 25 states encompassing seven geographical areas to investigate a variety of binders, road types, pavement types, and application methods. Laboratory tests concluded that the optimum bead size for good wet-night reflectivity ranged from U.S. sieves 10 to 20, depending on the binder. The field test and demonstration projects showed that large glass beads provided significantly superior retroreflectivities compared with standard bead sizes for a variety of thin and thick binders. That study also suggested bead gradation specifications for single- and dual-drop marking systems.

Mendola (*11*) conducted a study to evaluate the retroreflectivity performance of VISIBEADs in epoxy paint at a 20-mil thickness. That study used photometric and visual tests to compare large glass beads in epoxy paint and standard beads in traffic paint. The tests concluded that large beads produced significantly better retroreflectivity than standard beads and that the cost difference between large and standard beads is relatively low when compared with the increase in wet-night visibility.

King and Graham (*12*) evaluated the visibilities of eight pavement marking materials for the state of North Carolina. The study also evaluated epoxy markings reflectorized with large glass beads and concluded that the larger beads improved the wet-night visibilities of the markings by a factor of two, or more than that for adjacent standard bead lines.

Griffin (*13*) conducted a study for the Colorado Department of Highways to review the performances of pavement marking materials. From that study Griffin concluded that the VISIBEADs met the specifications of the department but were susceptible to loss because of snowplowing operations.

A 1989 *Better Roads* survey (*14*) reported on the pavement marking types preferred by highway departments and some of the problems experienced with them. That survey reported favorably on the performance of VISIBEADs during wet weather.

The Ohio Turnpike substituted VISIBEADs for standard beads in a large glass bead testing project over a wide range of traffic volumes, roadway geometrics, and pavement conditions (*15*). Preliminary results indicated that average reflectivities were well above the minimum acceptable levels. Joint sealing and snowplowing were identified as two major causes of reflectivity loss. It was estimated that 25 to 30 percent of the pavement markings were damaged by snowplowing and another 5 to 10 percent were damaged by joint sealing.

STATE-OF-THE-ART SUMMARY

Considerable effort has been expended in determining which type of pavement marking material has the most cost-effective service life. As expected the service life is dependent on the application procedures, material components, pavement surface, traffic intensity, and environmental conditions. Although some studies have addressed larger bead sizes for increasing wet-night retroreflectivity, specific information regarding the performance of particular bead size and pavement marking material combinations is scarce.

A survey was conducted to determine the experiences of southern states with different glass bead sizes and various pavement marking materials. The survey was mailed to southern state highway agencies, agencies conducting research on large beads, and paint and bead manufacturers. The results of the survey are summarized below.

Pavement Marking Materials Used

Thirty percent of the agencies that used paint used both water-based and alkyd-based paints; 30 percent applied alkyd-based paints only, whereas 40 percent did not specify the type of paint used. Eighty percent of the responding agencies employed paints for both longline and transverse markings; 10 percent used paint only for transverse markings, whereas another 10 percent used paints only for longlines on both asphalt and concrete pavements. Eighty percent of the agencies used thermoplastics at least on an experimental basis, 40 percent used thermoplastics for both longline and transverse markings, 30 percent used thermoplastics on asphalt pavements alone, and 10 percent used thermoplastics on an experimental basis alone.

Eighty percent of the responding agencies used preformed material on both asphalt and concrete pavements. Seventy percent used preformed material for both longline and transverse markings, and 10 percent used preformed material only for transverse markings. Ninety percent of the responding agencies employed epoxy markings on their roadways. Seventy percent used epoxy for longline and transverse markings, 10 percent used epoxy on concrete pavements alone, and 10 percent used epoxy for longlines on asphalt pavements alone.

Forty percent of the responding agencies used polyester for pavement markings. Ten percent used it for longline and transverse markings on both asphalt as well as concrete, 10 percent used it only for longlines on both asphalt as well as concrete, and the remaining 20 percent used polyester for longlines on asphalt alone. Fifty percent of the responses indicated the use of epoflex. Ten percent of the agencies applied epoflex for longline and transverse markings on both asphalt and concrete, 20 percent used it for longlines on both asphalt and concrete, and the remaining 10 percent used it for longline and transverse markings on asphalt alone.

Reapplication Schedule

All agencies reported using paint as a marking material. Of these 20 percent used water-based paints, with a reapplication schedule ranging from 6 to 12 months. Seventy percent did not specify the paint base used. These agencies had a reapplication every 12 months for longlines on asphalt, 6 months to 4 years for transverse markings on asphalt, 6 to 12 months for longlines on concrete, and 6 months to 4 years for transverse markings on concrete. Ten percent of the agencies used alkyd-based paint with a reapplication schedule of 6 months for longline and transverse markings on both asphalt and concrete pavements.

Thermoplastics were used by 10 percent of the agencies for longline and transverse markings on both asphalt and concrete, with a reapplication every 3 years. An additional 10 percent used thermoplastics for longlines on both asphalt and concrete, with reapplication once a year, whereas 10 percent used thermoplastics for longlines on asphalt, with reapplication every 5 years, and for transverse markings on asphalt, with reapplication every 3 to 4 years. Twenty percent of the agencies reported the use of thermoplastics, 10 percent without specifying a reapplication schedule and 10 percent having let out the maintenance to contract.

Thirty percent of the agencies responding used preformed materials for longlines and transverse markings on both asphalt and concrete pavements, with reapplication every 2 to 5 years; 10 percent of the agencies used preformed materials for transverse markings on both asphalt and concrete, with reapplication every 3 years; and 20 percent used preformed markings for longlines on asphalt and concrete, with reapplication every 2 to 8 years on asphalt and 1 to 5 years on concrete. An additional 10 percent used preformed materials for longline and transverse markings on concrete but let out the maintenance to contract.

Fifty percent of the agencies responding used epoxy for longline and transverse markings on both asphalt and concrete, with reapplication every 2 to 5 years; 20 percent used epoxy for longlines on asphalt and concrete, with reapplication every 2 to 4 years; and 10 percent used epoxy for longlines on asphalt, reapplying it every 2.5 years. Ten percent of the agencies employed epoxy on new construction projects.

Ten percent of the agencies used polyester for longlines on both asphalt and concrete, with a reapplication every 2 years. The remaining 30 percent applied polyester for longlines on asphalt, with reapplication every 6 months to 2 years. Of the agencies that used epoflex, 30 percent used epoflex for longlines on asphalt, with the prevalent conclusion that the material was unsuitable for pavement marking. The remaining 20 percent either did not indicate the type or area of use or reported that the maintenance of the markings was let out to contract.

Special Problems with Marking Materials

Seventy percent of the agencies reported encountering problems with paint. Thirty percent noted that paints had a poor service life, 10 percent reported the unavailability of large quantities of chlorinated rubber, 10 percent said that the slow drying times of paints led to damage claims, whereas 10 percent indicated that the paint markings faded when used for longlines on asphalt pavements. An additional 10 percent indicated that paints performed as expected, with only localized problems occurring.

Fifty percent of the responding agencies reported problems with thermoplastic markings. Ten percent reported adhesion problems with longline and transverse markings on both asphalt and concrete. This problem was attributed to the pavement condition and the method of application. Snowplow damage to longline and transverse markings on asphalt as well as poor adhesion for longlines on concrete pavements were reported as problems by 10 percent of the agencies. Ten percent reported bond failure of longlines on asphalt because of snowplowing, whereas another 10 percent reported cracks in longlines on asphalt and bond failure of transverse markings on concrete because of snowplowing. A final 10 percent indicated that thermoplastics had poor service lives on concrete pavements.

A large portion, 70 percent, of the responding agencies encountered problems with the use of preformed materials as pavement markings. Ten percent reported problems with adhesion and the shifting of longlines and transverse markings on both asphalt and concrete pavements; 10 percent reported the shifting of longlines and another 10 percent reported the shifting of transverse markings on asphalt. Ten percent of the agencies indicated that they encountered retention problems and snowplow damage to markings placed on concrete. Another 20 percent reported the loss of retroreflectivity on both asphalt and concrete, with a final 10 percent reported peeling and adhesion problems at crosswalks.

Seventy percent of the responding agencies reported encountering problems with the use of epoxy as a pavement marking material. Most of these were concerned with discoloration, yellowing or graying, of the white epoxy markings. Thirty percent reported a yellowing or graying of longlines on asphalt, 10 percent noted graying of longlines and transverse markings on concrete, whereas another 10 percent reported fading and discoloration of longlines on concrete. Dead spots (sunken beads) were reported by 10 percent of the agencies. This problem was remedied by using larger beads, but this in turn led to mixing problems. A final 10 percent of the agencies reported some early adhesion problems on both asphalt as well as concrete, but were generally satisfied with the performance of epoxy as a marking material.

Half of the agencies that employed polyester reported problems with its use. The long drying times of longlines on asphalt and concrete and the short service lives of longlines on asphalt were the two principal problems encountered with the use of polyester. Forty percent of the agencies reported adhesion problems and a short service life of epoflex. Of these, 10 percent reported adhesion inadequacies of longline and transverse epoflex markings on both asphalt and concrete, and another 10 percent reported the total failure within 1 year of epoflex longlines on asphalt and concrete. A final 20 percent indicated adhesion problems and a poor service life of epoflex longlines on asphalt.

Marking and Bead Application

The predominant application rate of drop-on, standard-sized beads reported was 6 lb/gal of paint and 20 to 25 lb/gal of epoxy. Ten percent of the agencies reported a drop-on bead application rate for VISIBEADs of 12 lb/gal of paint and a mix of 12 lb of standard beads and 15 lb of VISIBEADs per gal of epoxy by using drop-on application.

Bead Surface Treatments

Specific bead treatments for moisture proofing and adhesion were reported by 40 percent of the agencies. Twenty percent employed moisture proofing treatments; 10 percent used adhesion coatings for beads in epoxy and flotation beads in thermoplastics. A final 10 percent applied a silane coating for paint beads.

Use of Large Beads

All of the responding agencies had experience with the use of large glass beads, at least on an experimental basis. Ten percent used glass beads of sizes 16 to 50 at an application rate of 12 lb/100 ft^2 of 120-mil-thick thermoplastic. The remaining agencies employed large glass beads of sizes 12 to 50 on epoxy at a 15- to

20-mil thickness. Ninety percent of the agencies found the retroreflectivity level of the large glass beads to be satisfactory, and the remaining were still evaluating it.

Effective Bead-Marking Combinations

Eighty percent of the responding agencies provided information on marking material and glass bead combination. Epoxy markings were used by 40 percent for longlines and by 30 percent for transverse markings in urban and rural areas. Thermoplastics were used by 30 percent for longlines and transverse markings and by 20 percent in urban areas. The glass beads mainly employed for epoxy were VISIBEADs of sizes 14 to 20, a mixture of standard and large beads for thermoplastics, whereas standard-sized beads were used with paints.

COMPARATIVE APPLICATION COSTS

The cost estimates contained in this section were obtained from McGrath (*2*) unless otherwise identified. Paint costs approximately $5 to $10/gal ($1.31 to $2.63/L), and installation costs range from $0.03 to $0.05/ft ($0.09 to $0.18/m) for a 15-mil (4-in.; 10.2-cm) line. Although cost per installed foot for paint is reasonable, the apparent cost savings are often lost because of the required frequency of application. The cost of thermoplastics ranges from $0.30 to $0.40/ft ($0.92 to $1.22/m) for a 4-in. (10.2-cm) line and is 5 to 10 times higher than that of paint because of material and installation costs.

The use of preformed tapes for lane lines is too new to have produced good benefit-cost statistics, with costs ranging from $0.56 to $0.90/ft ($1.71 to $2.75/m). Virginia paid $0.60/ft ($1.83/m) for a 4-in. (10.2-cm) line and expected the price to decrease. Only on the basis of the costs of materials and installation, state officials estimated that preformed tape placed on a section of Interstate highway must last approximately 8 years to offset the cost of traffic paint. Although high cost has been reported as a disadvantage for preformed markings, it is quick and easy to install, and application equipment is readily available.

Epoxy has an initial cost six times higher than that of paint, although its extended service life makes its cost comparable to that of paint. On the basis of a 4-year projected cost study under various traffic volume situations, the Minnesota Department of Transportation found that epoxy outlasted paint, thermoplastic, and polyester materials by factors of two to eight. New epoxy formulations and redesign of equipment are expected to reduce costs (*2*). Polyester compares with other durable materials, as shown in Table 1.

Epoflex costs approximately $25/gal, compared with $4/gal for traffic paint. A 4-in. line of epoflex at a thickness of 15 mil costs $0.21/ft, which is $0.16/ft more than paint applied at 10 mil dry. Since epoflex has twice the service life of paint in cold climates, however, it actually costs less in the long run. The cost of producing epoflex from raw materials is expected to decrease because of the simplicity of its manufacture. Although the cost of converting stripers designed for paint can be significant, epoflex's service life under almost any condition is expected to rapidly amortize this cost over a short time.

TABLE 1 Comparative Costs of Durable Pavement Markings

	Marking Cost ($ per foot)		
	Polyester	Fast-dry Paints	Epoxy
Edge line	0.12	0.03	NA
Lane line	0.15	0.04	0.40
Center line	0.20	0.05	0.50

SUMMARY

Potters Industries conducted the initial research that has resulted in the modern large glass bead technology. This technology tailors the bead coating and size to the type and thickness of the pavement marking material. Experience with large beads indicates that they become an integral part of the wearing surface and actually extend pavement marking life in high-traffic areas. Since the effective service life of pavement markings is, however, dependent on more variables than traffic wear, the planned lives of materials with large beads should be considered equal to those with the same marking materials with standard beads. Large beads, even when worn by traffic and abraded by dirt, provide greater retroreflectivity values than new standard beads.

Advances in two pavement marking areas have resulted in the ability to use large bead sizes. The first is in new pavement marking materials, applications systems, and enhancements to existing marking materials that have resulted in the ability to apply thicker layers of material. The high quantity of solids results in a greater dry thickness with acceptable drying times. The second area of advancement is in bead manufacture. Potters Industries has devised a system of large bead size and bead coatings that is optimal for each type of binding material. The result is beads manufactured for a particular type of pavement marking material.

Since the size and bead coating varies in accordance with the type of pavement marking material used, there exists variation in applied cost. Accompanying this variation in cost is the variation in service life because of durability of the pavement marking material. Although there have been claims that the larger bead sizes become part of the wearing surface and thus increase pavement marking life, it is generally found that pavement markings with larger beads will not last longer than pavement markings with regular bead sizes. The higher cost of larger beads, coupled with the limited life of paint, results in paint not being a cost-effective medium for such beads. A possible exception to this would be in areas of high precipitation that are experiencing a relatively large number of accidents attributable to poor nighttime delineation. Even in this instance, however, the agency would experience increased cost-efficiency by using more durable binders.

Much of the current knowledge on the effectiveness and service lives of large beads is based on the experience of states with climatic conditions that are drastically different from those of the southern United States. The study described here revealed that the large beads are effective in increasing the retroreflectivities of pavement markings in wet-night conditions. However it is not known with certainty how the high pavement temperatures and other climatic conditions indigenous to the South will affect the effective service lives of markings. It would be advantageous to test large beads on rural roads that experience a relatively large number of wet-night accidents. The proper selection of test segments will enable determination of accident reduction benefits in addition to marking visibility enhancement. No studies that have attempted to quantify accident reductions because of large bead application have been identified.

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Fly Ash as Mineral Filler in Traffic Marking Paint: A Feasibility Study

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A feasibility study was a cooperative effort between the Nebraska Department of Roads (NDOR), American Electric Power Company, and FHWA. The goal of the study was to give the highway community an indication of the feasibility of using fly ash in traffic marking paint. The study was an initial field test to verify the possibilities of using fly ash as an abrasion-resistant mineral filler pigment in traffic paint to improve the paint's durability and performance. Five types of paint were compared in the study: the standard NDOR yellow traffic paint and four other oil-alkyd yellow traffic paints specially formulated for the test by two different companies. Two of the paints contained fly ash. All of the paints performed satisfactorily in the laboratory tests in comparison with NDOR standard traffic paint specifications. The five traffic paints were applied with glass beads to Portland Cement Concrete pavement with a hand-pushed mechanical paint sprayer on October 6, 1992. The reflectivity readings of the paint stripes were taken and the paints were visually monitored for film failure for 1 year. In most cases the paint containing fly ash performed better than the other paints. The study was a small feasibility study to evaluate fly ash as a mineral filler pigment in traffic marking paint. The satisfactory performance of the paint containing fly ash indicates that fly ash could be used in traffic paint to improve its durability and performance. Further field testing of traffic marking paint containing fly ash is justified.

The feasibility study described here was a cooperative effort between the Nebraska Department of Roads (NDOR), the American Electric Power Company (AEP), and FHWA. The goal of the test program was to give the highway community an indication of the feasibility of using fly ash in traffic marking paint. It was believed that paint traffic formulated with fly ash as a mineral filler pigment would be more wear resistant and would therefore last longer on pavements than the typical sprayed-applied paints currently being used. A more durable traffic paint will allow motorists to see the stripes longer and will require less repainting, hence less risk for highway maintenance workers. This will increase the safety on the nation's highways and streets and decrease costs.

Fly ash's ceramic composition of silica, alumina, and other metal oxides makes it one of the hardest and most abrasion-resistant mineral fillers available. In mineralogy the hardness of a mineral is generally defined as its resistance to scratching. The Mohs Hardness Scale categorizes materials from 1 to 10, rating talc, a soft mineral, as 1 and diamond as 10. Fly ash particles are judged to have ratings of between 7 and 8. Fly ash is an excellent paint extender (filler) material that helps to provide body, mechanical strength, and abrasion resistance, and it assists with opacity. Fly ash also fits other filler pigment requirements. It exhibits low resin and oil absorption properties (which permits high levels of loading without inordinate thickening), has a fine particle size, provides insolubility, is easy to disperse, and is chemically inert.

Calcium carbonate ($CaCO_3$) is one of the most widely used mineral filler pigments in traffic marking paints. Calcium carbonate is a soft, chalklike material, and fly ash is a much harder material. In theory by replacing part of the calcium carbonate with fly ash in traffic paint the paint would be more durable.

Fly ash originates from the residual inorganic matter contained in coal. Coal is burned in steam generators (boilers). Fly ash-laden flue gas is a by-product of coal combustion. As flue gas cools and flows through the steam generator, ash forms into spherical ceramic particles in the range of 1 to 20 μm. The particles are collected in electrostatic precipitators.

The use of Class F fly ash as an abrasion-resistant filler in a variety of protective coatings, including alkyds, acrylics, epoxies, asphalts, vinyls, and polyurethane vehicle systems, has been extensively evaluated by AEP. Ash-filled paint is currently the standard paint used on AEP's transmission towers, railcars, barges, structural steel, tanks, piping, equipment, and architectural structures. AEP also uses fly ash in traffic marking paints on the roads and parking lots at their facilities. Preliminary indications led AEP to believe that it is more durable than the traffic paint that they previously used. The feasibility study described here was the first time that field testing and comparative evaluations of traffic paint containing fly ash were initiated.

One of the shortcomings of fly ash is that it is not white in color. Its color can vary greatly, typically from light to dark grey (depending on the carbon content of the fly ash) or brown. This limits its use in paint when the final color is important, especially if the desired color is a light shade. There are also two types of fly ash, Class C and Class F. The two types have different properties, and each type can vary from source to source. The formulation of paint containing fly ash needs to take into account the type being used, Class C or Class F, and which power plant the fly ash comes from.

There have been many discussions in the past concerning hazardous waste and fly ash and whether it was appropriate to put fly ash in paints. A recent Final Regulatory Determination published in the *Federal Register* (*1*, p. 42466) stated:

> Environmental Protection Agency (EPA) has concluded that regulation under Subtitle C of the Resource Conservation and Recovery Act (RCRA) is inappropriate for the four large-volume fossil-fuel combustion waste streams—fly ash, bottom ash, boiler slag, and flue gas emission control waste—because of the limited risks posed by them and the existence of generally adequate State and Federal regulatory programs. The EPA also believes that the potential for damage

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from these wastes is most often determined by site- or region-specific factors and that the current State approach to regulation is thus appropriate. Therefore, the EPA will continue to exempt these wastes from regulation as hazardous wastes under RCRA Subtitle C.

This determination became effective September 2, 1993. EPA does not consider fly ash to be a hazardous waste, and the authors believe there is very low risk of damage to the environment by putting fly ash in traffic marking paint.

With the national consciousness focused on recycling and utilization of domestic resources, the utilization of fly ash, an abundant U.S. mineral resource, appears to be warranted. Most other mineral fillers in paints (i.e., talc, calcium carbonate, silica, feldspar, and clay) are mined, crushed, and further processed before they can be used. Many of them are also imported. Approximately 506 million L (50 million gal) of paint is used each year to mark roads, highways, streets, and parking lots in the United States. Since traffic marking paints typically contain mineral filler at levels of 0.6 to 0.7 kg/L (5 to 6 lb/gal), fly ash-based traffic paint may pave the way to an environmentally favorable use of this abundant domestic mineral resource.

DISCUSSION OF RESULTS

Five types of paint were compared in the study: the standard NDOR yellow traffic paint and four other oil and alkyd yellow traffic paints specially formulated for the test by two different companies (referred to here as P-S and Y-M). The paint companies each formulated two identical paints that met the NDOR traffic paint specifications, except that one paint had Class F fly ash replacing a percentage of the calcium carbonate as a mineral filler pigment and one did not. The five paints were designated as follows: NDOR standard, P-S with fly ash, P-S without fly ash, Y-M with fly ash, and Y-M without fly ash. The chemical and physical properties of the fly ash used are given in Table 1. The NDOR traffic paint and the Y-M paints contained a lead pigment, chrome yellow. The P-S paints were made with an organic pigment, arylide yellow. The pigment compositions of the paints are presented in Tables 2 through 6. Pigment percentages of total weight for Tables 2 through 6 are 59.6, 55.37, 55.68, 65.03, and

TABLE 1 Chemical and Physical Properties of Fly Ash

Chemical	% of Total Weight
Silicon Dioxide (SiO_2)	59.6
Aluminum Oxide (Al_2O_3)	29.9
Iron Oxide (Fe_2O_3)	4.2
Titanium Dioxide (TiO_2)	1.6
Calcium Oxide (CaO)	0.8
Magnesium Oxide (MgO)	6.9
Sodium Oxide (NaO_2)	0.3
Potassium Oxide (K_2O)	2.4
Sulphur Trioxide (SO_3)	0.2
Phosphorus Pentoxide (P_2O_5)	0.1
Other	0.6
Total (rounded)	100 %

pH = 6.1
Specific Gravity = 2.17
% Retained on #325 Sieve = 16.13%

TABLE 2 Pigment Composition of NDOR Standard

Pigment	Percent of pigment by weight
Medium Chrome Yellow	8.7
Titanium Dioxide	2.7
Yellow Iron Oxide	0.4
Magnesium Silicate	13.4
Aluminum Silicate	26.8
Calcium Carbonate	46.9
Anti Settling Agent	1.1

TABLE 3 Pigment Composition of P-S with Fly Ash

Pigment	Percent of pigment by weight
Calcium Carbonate	71.18
Treated Fly Ash	10.95
Arylide Yellow Pigment	9.39
Titanium Dioxide	5.99
HDPE	2.04
Thixotrope	0.45

TABLE 4 Pigment Composition of P-S without Fly Ash

Pigment	Percent of pigment by weight
Calcium Carbonate	82.12
Arylide Yellow Pigment	9.39
Titanium Dioxide	5.99
HDPE	2.05
Thixotrope	0.45

TABLE 5 Pigment Composition of Y-M with Fly Ash

Pigment	Percent of pigment by weight
Calcium Carbonate	72.98
Chrome Yellow Medium	12.28
Titanium Dioxide	3.04
Fly ash - yellow	11.70

TABLE 6 Pigment Composition of Y-M without Fly Ash

Pigment	Percent of pigment by weight
Calcium Carbonate	84.68
Chrome Yellow Medium	12.28
Titanium Dioxide	3.04

65.03 percent, respectively. The percentage of fly ash placed in the two special paints was determined by using the maximum amount possible while still maintaining the desired yellow color for traffic marking paint.

Traffic paint containing fly ash is a relatively new concept and is still in the test and evaluation stages. The two paint companies that formulated the special test paints made small prototype quantities and supplied the paint for the test at no charge. These companies typically do not make traffic paint, so the cost for them to make the test paint (although not specifically calculated) was relatively expensive. They purchased small quantities of ingredients used in traffic paints that they do not normally use in the paints that they manufacture. To reduce the particle size the fly ash had to be processed in a steel ball mill. Calcium carbonate is also ground before it is sold as a mineral filler. After the grinding process the fly ash was introduced into the manufacturing process for the paint in the same manner as calcium carbonate. It is believed that if manufactured on a larger scale by a traffic marking paint manufacturer, the cost of traffic paint containing fly ash would be comparable to the cost of commercially available traffic paint. It could perhaps be even less costly in areas where fly ash is given away by power plants. The average cost of NDOR traffic paint is \$0.87/L (\$3.30/gal) of paint.

The four special paints were tested at the NDOR laboratory for mixing characteristics, color, finish, consistency, flexibility, bleeding, water resistance, settling properties, dry hiding power, paint composition, pigment composition, X-ray diffractogram of pigment, infrared spectrum of vehicle, nonvolatile content, and luminous reflectance. All of the paints performed satisfactorily in comparison with NDOR standard traffic paint specifications. The laboratory test results of the four special paints are listed in Tables 7 through 10.

The five traffic paints with glass beads were applied to Portland Cement Concrete pavement with a hand-pushed mechanical paint sprayer on October 6, 1992. The bead application rate was 0.72 kg/L (6 lb/gal) of paint. The pavement was dry and the air temperature was 21°C (70°F). Four 10-cm (4-in.)-wide stripes of each paint were applied perpendicularly to the centerline of the road for evaluation. The road is near the NDOR headquarters, which allowed for easy observation of the test paint, and has an average daily traffic of 500 vehicles. Each stripe was 3.35 m (11 ft) in length and extended from the pavement edge to the centerline of the road.

No modifications to the mechanical paint sprayer were required for the application of the two paints containing fly ash. The two paints performed normally during the application process; neither showed signs of clogging or settling in the sprayer or running after application on the road. These two paints also did not require special cleanup procedures.

In the initial observations the four specially formulated paints did not meet the NDOR drying time (no pickup) requirements of

TABLE 7 P-S with Fly Ash Paint, Laboratory Test Results

	Actual Results	Nebraska 1991 Requirements
Appearance and Mixing Characteristics....	satisfactory	Well Ground and Readily Mixed
Color..................................	satisfactory	No. 33538 or Federal 595.a
Finish.................................	satisfactory	Flat or Eggshell
Drying Time, 25°C * (ASTM D711), minutes:		
No Pickup............................	24	15 Max.
Thoroughly Dry & Free From Tackiness.	30	--
Consistency, 25°C *, 1/2" Krebs Unit.......	86	70 to 80
Flexibility, 25°C *, 1/2" Mandrel..........	satisfactory	--
Bleeding (Bituminous Surface)...........	no bleeding	--
Water Resistance.......................	9	--
Settling Properties, 2 weeks, (ASTM D 1390)	8	6 Min.
Dry Hiding Power.......................	complete hiding	Complete Hiding
Paint Composition:		
Pigment, percent by weight............	53.10	55 Min.
Vehicle, percent by weight............	46.90	45 Max.
Weight, kg/L **, 25°C.................	1.39	1.40 Min.
Coarse Particles, Lumps, & Skins (Retained No. 325 sieve), percent by weight.....	0.10	1 Max.
Pigment Composition, percent by weight:		
Chrome Yellow ($PbCrO_4$)................	---	5 to 25
Titanium Dioxide (TiO_2)...............	---	5 Max.
Calcium Carbonate ($CaCO_3$).............	---	10 to 65
Siliceous Inerts (by difference).......	---	10 to 85
X-Ray Diffractogram of Pigment..........	satisfactory	Satisfactory
Infrared Spectrum of Vehicle............	satisfactory	Satisfactory
Non-Volatile, percent by weight of paint.	70.5	73 Min.
Luminous Reflectance	49.2	48 to 52

* 25°C = 77°F
** 1 kg/L = 0.119 lb/gal

TABLE 8 P-S without Fly Ash Paint, Laboratory Test Results

	Actual Results	Nebraska 1991 Requirements
Appearance and Mixing Characteristics....	satisfactory	Well Ground and Readily Mixed
Color..................................	satisfactory	No. 33538 or Federal 595.a
Finish.................................	satisfactory	Flat or Eggshell
Drying Time, 25°C *(ASTM D711), minutes:		
No Pickup............................	23	15 Max.
Thoroughly Dry & Free From Tackiness.	28	--
Consistency, 25°C *, 1/2" Krebs Unit.......	87	70 to 80
Flexibility, 25°C *, 1/2" Mandrel..........	satisfactory	--
Bleeding (Bituminous Surface)............	no bleeding	--
Water Resistance.......................	9	--
Settling Properties, 2 weeks, (ASTM D 1390)	8	6 Min.
Dry Hiding Power.......................	complete hiding	Complete Hiding
Paint Composition:		
Pigment, percent by weight............	54.0	55 Min.
Vehicle, percent by weight............	46.0	45 Max.
Weight, kg/L **, 25°C	1.39	1.40 Min.
Coarse Particles, Lumps, & Skins (Retained No. 325 sieve), percent by weight.....	0.10	1 Max.
Pigment Composition, percent by weight:		
Chrome Yellow ($PbCrO_4$)................	---	5 to 25
Titanium Dioxide (TiO_2)...............	---	5 Max.
Calcium Carbonate ($CaCO_3$).............	---	10 to 65
Siliceous Inerts (by difference).......	---	10 to 85
X-Ray Diffractogram of Pigment..........	satisfactory	Satisfactory
Infrared Spectrum of Vehicle............	satisfactory	Satisfactory
Non-Volatile, percent by weight of paint.	70.9	73 Min.
Luminous Reflectance	55.0	48 to 52

* 25°C = 77°F
** 1 kg/L = 0.119 lb/gal

TABLE 9 Y-M with Fly Ash Paint, Laboratory Test Results

	Actual Results	Nebraska 1991 Requirements
Appearance and Mixing Characteristics....	satisfactory	Well Ground and Readily Mixed
Color..................................	satisfactory	No. 33538 or Federal 595.a
Finish.................................	satisfactory	Flat or Eggshell
Drying Time, 25°C * (ASTM D711), minutes:		
No Pickup............................	27	15 Max.
Thoroughly Dry & Free From Tackiness.	32	--
Consistency, 25°C * 1/2" Krebs Unit......	77	70 to 80
Flexibility, 25°C * 1/2" Mandrel..........	satisfactory	--
Bleeding (Bituminous Surface)............	no bleeding	--
Water Resistance.......................	7-8	--
Settling Properties, 2 weeks, (ASTM D 1390)	8	6 Min.
Dry Hiding Power.......................	complete hiding	Complete Hiding
Paint Composition:		
Pigment, percent by weight............	65.3	55 Min.
Vehicle, percent by weight............	34.7	45 Max.
Weight, kg/L **, 25°C.................	1.56	1.40 Min.
Coarse Particles, Lumps, & Skins (Retained No. 325 sieve), percent by weight.....	0.5	1 Max.
Pigment Composition, percent by weight:		
Chrome Yellow ($PbCrO_4$)................	---	5 to 25
Titanium Dioxide (TiO_2)...............	---	5 Max.
Calcium Carbonate ($CaCO_3$).............	---	10 to 65
Siliceous Inerts (by difference).......	---	10 to 85
X-Ray Diffractogram of Pigment..........	satisfactory	Satisfactory
Infrared Spectrum of Vehicle............	satisfactory	Satisfactory
Non-Volatile, percent by weight of paint.	77.0	73 Min.
Luminous Reflectance	50.0	48 to 52

* 25°C = 77°F
** 1 kg/L = 0.119 lb/gal

TABLE 10 Y-M without Fly Ash Paint, Laboratory Test Results

	Actual Results	Nebraska 1991 Requirements
Appearance and Mixing Characteristics....	satisfactory	Well Ground and Readily Mixed
Color..................................	satisfactory	No. 33538 or Federal 595.a
Finish.................................	satisfactory	Flat or Eggshell
Drying Time, 25°C * (ASTM D711), minutes:		
No Pickup............................	18	15 Max.
Thoroughly Dry & Free From Tackiness.	24	--
Consistency, 25°C *, 1/2" Krebs Unit......	80	70 to 80
Flexibility, 25°C *, 1/2" Mandrel.........	satisfactory	--
Bleeding (Bituminous Surface)...........	no bleeding	--
Water Resistance.......................	9	--
Settling Properties, 2 weeks, (ASTM D 1390)	7	6 Min.
Dry Hiding Power.......................	complete hiding	Complete Hiding
Paint Composition:		
Pigment, percent by weight.............	60.1	55 Min.
Vehicle, percent by weight............	39.9	45 Max.
Weight, kg/L **, 25°..................	1.49	1.40 Min.
Coarse Particles, Lumps, & Skins (Retained No. 325 sieve), percent by weight.....	0.10	1 Max.
Pigment Composition, percent by weight:		
Chrome Yellow ($PbCrO_4$)................	---	5 to 25
Titanium Dioxide (TiO_2)...............	---	5 Max.
Calcium Carbonate ($CaCO_3$).............	---	10 to 65
Siliceous Inerts (by difference).......	---	10 to 85
X-Ray Diffractogram of Pigment..........	satisfactory	Satisfactory
Infrared Spectrum of Vehicle............	satisfactory	Satisfactory
Non-Volatile, percent by weight of paint.	76.0	73 Min.
Luminous Reflectance	54.5	48 to 52

* 25°C = 77°F
** 1 kg/L = 0.119 lb/gal

15 min maximum. Their drying times ranged from 18 to 27 min. The P-S with fly ash paint's drying time was 1 min longer than that of the P-S without fly ash paint. The fly ash did not significantly affect the drying time of the P-S paints. The Y-M with fly ash paint's drying time was 9 min longer than that of the Y-M without fly ash paint. In the case of the Y-M paints fly ash did significantly affect the drying times of the paints. It is unclear at this time why the fly ash affected the drying time in one manufacturer's paint and not the other manufacturer's paint. The paint manufacturers believe that the drying time for all four paints could be reduced to below the NDOR requirement of 15 min with minor paint formulation adjustments.

The paints went through a typical Nebraska winter. The average temperature and precipitation for the months of October 1992 to February 1993 are given in Table 11. The paints were visually monitored for film failure for 1 year. Film failure is a visual determination of the percentage of paint in each stripe that is no longer adhered to the road. This is an indication of the paint's durability. Film failure of 0 percent means that no paint has worn off the pavement. Film failure of 100 percent means that all of the paint has worn off the pavement. The actual film failure readings for all four stripes of each type of paint and the averages are given in Table 12. The average film failure values for each type of paint are graphically compared in Figure 1. After 1 year the

TABLE 11 Average Monthly Temperatures and Precipitation

	Temperature		Precipitation	
Month	°F	°C	Inches	Millimeter
Oct 1992	53.7	12.1	1.70	42.5
Nov 1992	33.9	1.1	1.44	36.0
Dec 1992	26.4	-3.1	0.87	21.8
Jan 1993	17.3	-8.3	1.34	33.5
Feb 1993	20.7	-6.3	0.67	16.8

Y-M with fly ash paint, at an average of 40 percent film failure, performed better than the Y-M without fly ash paint, at 50 percent. However, the P-S with fly ash, at an average of 39 percent film failure, performed worse than the P-S without fly ash, at 31 percent. All of the special paints performed better than the NDOR standard paint, which had an average of 78 percent film failure 1 year after application.

The reflectivity readings of the paints were taken monthly for a period of 1 year with a retroreflectometer (Mirolux 12) in the inside wheeltrack, the middle, and the outside wheeltrack of the stripes. Reflectivity is an indication of the paint's bead retention capability. The higher the reading the better the reflectivity. This

TABLE 12 Actual and Average Film Failure Readings of Test Paints

FILM FAILURE: APPROX. 11 FEET STRIPE LENGTH THEREFORE
1 FOOT = 9% FILM FAILURE (VISUALLY DETERMINED)

		10/7/92	10/20/92	11/6/92	12/6/92	1/10/93	3/6/93	4/6/93	7/6/93	8/11/93	10/21/93
PRUETT SCHAFFER WITH FLYASH											
NORTH	B1	0	2	2	4	6	20	22	25	25	30
	B2	0	2	2	4	6	15	17	20	20	28
SOUTH	B1	0	2	2	3	5	15	22	40	45	48
	B2	0	2	2	3	5	15	22	40	40	48
	AVG	0.0	2.0	2.0	3.5	5.5	16.3	20.8	31.3	32.5	38.5
PRUETT SCHAFFER WITHOUT FLYASH											
NORTH	C1	0	3	3	4	6	20	22	25	25	30
	C2	0	3	3	4	6	20	22	25	25	38
SOUTH	C1	0	2	2	3	5	12	16	20	20	28
	C2	0	2	2	3	5	12	16	20	20	28
	AVG	0.0	2.5	2.5	3.5	5.5	16.0	19.0	22.5	22.5	31.0
NDOR STANDARD PAINT FROM CENTERLINE INDUSTRIES											
NORTH	D1	0	1	1	5	10	45	50	60	65	75
	D2	0	1	1	3	6	45	50	60	65	75
SOUTH	D1	0	1	1	2	7	50	55	70	75	80
	D2	0	1	1	2	8	50	55	70	75	80
	AVG	0.0	1.0	1.0	3.0	7.8	47.5	52.5	65.0	70.0	77.5
YENKIN MAJESTIC WITH FLYASH											
NORTH	E1	0	2	2	3	6	30	35	40	45	50
	E2	0	2	2	3	6	25	30	40	40	50
SOUTH	E1	0	1	1	2	4	12	15	22	25	30
	E2	0	2	2	3	5	12	15	22	25	30
	AVG	0.0	1.8	1.8	2.8	5.3	19.8	23.8	31.0	33.8	40.0
YENKIN MAJESTIC WITHOUT FLYASH											
NORTH	F1	0	2	2	5	10	30	40	45	45	50
	F2	0	2	2	5	10	25	32	45	45	50
SOUTH	F1	0	2	2	5	10	20	25	35	40	50
	F2	0	2	2	5	10	20	25	35	40	50
	AVG	0.0	2.0	2.0	5.0	10.0	23.8	30.5	40.0	42.5	50.0

shows how well the paint stripes can be seen at night by a driver with the vehicle's headlights shining on the stripe. The luminous reflectance unit of measure for reflectivity is millicandelas per square meter. The actual reflectivity readings for all four stripes of each type of paint and the averages are listed in Table 13. The average test reflectivity readings for the inside wheeltrack, the

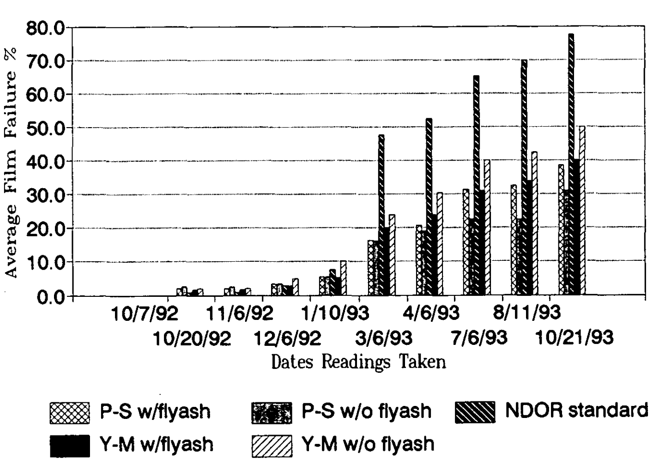

FIGURE 1 Film failure of traffic paint determined visually.

middle, and the outside wheeltrack of each type of paint are compared in Figures 2 through 4, respectively.

At the end of 1 year the following is a comparison of average reflectivity readings for the P-S paints. In the inside wheeltrack, P-S with fly ash (average reflectivity = 50.5) performed better than P-S without fly ash (average reflectivity = 46.3); in the middle of the stripe, P-S with fly ash (average reflectivity = 86.5) performed better than P-S without fly ash (average reflectivity = 77.5); in the outside wheeltrack, P-S with fly ash's average reflectivity at 43.8 was less than P-S without fly ash's average reflectivity at 58.8.

For the Y-M paints, in the inside wheeltrack, Y-M with fly ash (average reflectivity = 55.8) performed better than the Y-M without fly ash (average reflectivity = 24.3); in the middle of the stripe, Y-M with fly ash (average reflectivity = 112.0) performed better than the Y-M without fly ash (average reflectivity = 69.5); in the outside wheel track, Y-M with fly ash (average reflectivity = 61.5) also performed better than Y-M without fly ash (average reflectivity = 24.8).

For the NDOR standard paint, in the inside wheeltrack, all special paints except the Y-M without fly ash performed better than the NDOR standard; in the middle of the stripe, only YM with fly ash performed better than the NDOR standard; in the outside wheeltrack, all of the other paints performed better than the NDOR standard.

Summarizing the reflectivity readings of the four special test paints, in five of six cases the special paints containing fly ash had higher 1-year average reflectivity readings than the special

TABLE 13 Actual and Average Reflectivity Readings of Test Paints

REFLECTIVITY		10/6/92	11/6/92	12/11/92	1/19/93	3/9/93	4/6/93	7/6/93	8/11/93	10/21/93
PRUETT SCHAFFER PAINT WITH FLYASH										
INSIDE WHEELTRK										
PS N	B1	259	163	139	70	60	81	68	61	45
	B2	240	169	138	73	71	102	102	80	71
PS S	B1	298	185	145	86	66	66	64	57	38
	B2	279	160	136	80	62	72	50	53	48
	AVG	269.0	169.3	139.5	77.3	64.8	80.3	71.0	62.8	50.5
MIDDLE										
PS N	B1	232	193	167	101	89	129	124	90	106
	B2	238	180	117	101	101	140	129	105	103
PS S	B1	294	197	175	102	98	125	81	99	62
	B2	278	214	120	102	85	102	88	83	75
		260.5	196.0	144.8	101.5	93.3	124.0	105.5	94.3	86.5
OUTSIDE WHEELTRK										
PS N	B1	276	174	117	72	56	79	53	72	40
	B2	252	148	145	70	73	94	75	90	58
PS S	B1	270	148	146	71	63	84	77	77	33
	B2	226	130	158	74	65	110	57	63	44
	AVG	256.0	150.0	141.5	71.8	64.3	91.8	65.5	75.5	43.8
PRUETT SCHAFFER PAINT WITHOUT FLYASH										
INSIDE WHEELTRK										
PS N	C1	183	122	131	70	81	102	83	62	49
	C2	219	180	138	80	76	74	67	59	45
PS S	C1	230	203	152	76	63	63	56	61	42
	C2	245	172	170	83	74	75	75	75	49
	AVG	219.3	169.3	147.8	77.3	73.5	78.5	70.3	64.3	46.3
MIDDLE										
PS N	C1	204	140	95	98	94	109	114	77	89
	C2	216	175	130	108	102	126	96	80	61
PS S	C1	200	214	180	114	106	137	101	108	78
	C2	225	219	178	97	96	147	143	111	82
	AVG	211.3	187.0	145.8	104.3	99.5	129.8	113.5	94.0	77.5
OUTSIDE WHEELTRK										
PS N	C1	184	126	144	77	79	104	73	76	64
	C2	221	120	147	86	76	95	64	67	59
PS S	C1	210	151	131	64	67	79	81	66	63
	C2	275	121	176	72	70	111	65	69	49
	AVG	222.5	129.5	149.5	74.8	73.0	97.3	70.8	69.5	58.8
NEBRASKA DEPARTMENT OF ROADS STANDARD PAINT BY CENTERLINE INDUSTRIES										
INSIDE WHEELTRK										
NE N	D1	142	90	102	66	75	52	42	47	25
	D2	195	140	152	79	68	70	50	44	31
NE S	D1	262	184	159	85	64	64	48	56	36
	D2	316	209	156	99	51	46	38	49	25
	AVG	228.8	155.8	142.3	82.3	64.5	58.0	44.5	49.0	29.3
MIDDLE										
NE N	D1	123	86	97	88	80	85	63	71	49
	D2	196	163	127	109	115	151	111	118	115
NE S	D1	268	222	179	121	113	160	145	128	97
	D2	353	273	199	102	126	162	132	126	100
	AVG	235.0	186.0	150.5	105.0	108.5	139.5	112.8	110.8	90.3
OUTSIDE WHEELTRK										
NE N	D1	103	59	70	58	57	58	41	45	25
	D2	220	131	132	75	70	82	59	65	43
NE S	D1	283	156	180	73	62	64	47	56	31
	D2	339	193	191	76	56	65	40	47	23
	AVG	236.3	134.8	143.3	70.5	61.3	67.3	46.8	53.3	30.5
YENKIN MAJESTIC PAINT WITH FLYASH										
INSIDE WHEELTRK										
YM N	E1	163	125	153	71	82	75	61	45	26
	E2	256	210	183	89	105	136	73	61	43
YM S	E1	261	247	198	109	98	141	86	97	82
	E2	295	242	204	127	102	124	95	98	72
	AVG	243.8	206.0	184.5	99.0	96.8	119.0	78.8	75.3	55.8
MIDDLE										
YM N	E1	158	146	118	101	94	123	88	95	75
	E2	237	231	140	137	110	141	102	128	117
YM S	E1	274	251	198	140	132	197	151	154	138
	E2	313	270	201	109	119	176	144	139	118
	AVG	245.5	224.5	164.3	121.8	113.8	159.3	121.3	129.0	112.0
OUTSIDE WHEELTRK										
YM N	E1	226	160	136	87	72	82	53	49	37
	E2	307	216	168	93	73	101	59	61	38
YM S	E1	300	210	174	72	113	144	105	103	92
	E2	314	202	188	125	96	201	99	104	79
	AVG	286.8	197.0	166.5	94.3	88.5	132.0	79.0	79.3	61.5

(continued on next page)

TABLE 13 (*continued*)

REFLECTIVITY		10/6/92	11/6/92	12/11/92	1/19/93	3/9/93	4/6/93	7/6/93	8/11/93	10/21/93
YENKIN MAJESTIC PAINT WITHOUT FLYASH										
INSIDE WHEELTRK										
YM N	F1	214	184	148	63	62	52	51	47	21
	F2	257	207	149	81	71	64	48	50	22
YM S	F1	244	183	167	67	56	49	44	54	26
	F2	265	202	171	80	60	49	44	51	28
	AVG	245.0	194.0	158.8	72.8	62.3	53.5	46.8	50.5	24.3
MIDDLE										
YM N	F1	237	194	136	103	96	74	63	83	61
	F2	265	230	183	127	121	146	93	94	69
YM S	F1	253	212	164	91	104	120	81	77	67
	F2	291	252	196	117	99	124	100	109	81
	AVG	261.5	222.0	169.8	109.5	105.0	116.0	84.3	90.8	69.5
OUTSIDE WHEELTRK										
YM N	F1	232	179	140	63	65	51	40	46	23
	F2	255	191	151	68	60	51	40	47	20
YM S	F1	290	141	167	66	69	78	50	54	27
	F2	230	142	119	67	70	68	51	56	29
	AVG	251.8	163.3	144.3	66.0	66.0	62.0	45.3	50.8	24.8

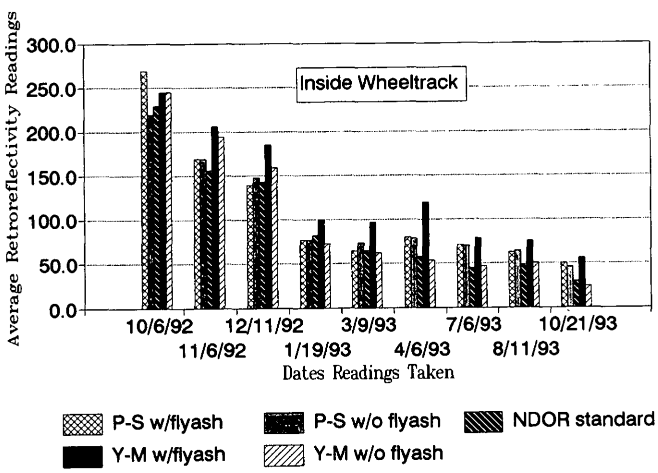

FIGURE 2 Average reflectivity readings inside wheeltrack in daytime using retroreflectometer.

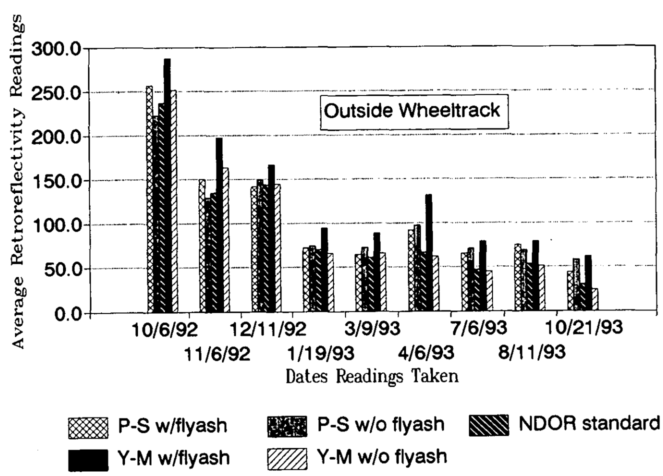

FIGURE 4 Average reflectivity readings outside wheeltrack in daytime using retroreflectometer.

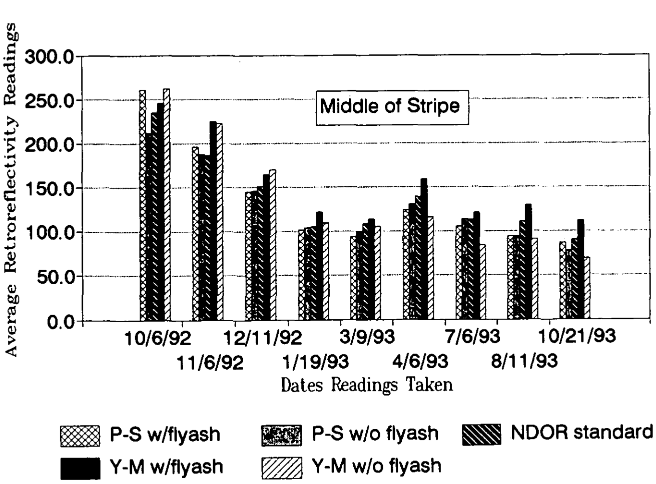

FIGURE 3 Average reflectivity readings in middle of stripe in daytime using retroreflectometer.

paints not containing fly ash. It was only in the outside wheeltrack where P-S without fly ash had a higher average reflectivity reading than P-S with fly ash. Comparing the two paints containing fly ash with the NDOR standard, in five of the six cases the paints containing fly ash had higher 1-year average reflectivity readings than the NDOR standard paint. It was only in the middle of the stripe where the NDOR standard paint had a higher average reflectivity reading than the P-S with fly ash paint.

CONCLUSIONS

1. In five of six cases the two paints containing fly ash had greater 1-year average reflectivity readings than the special paints not containing fly ash. The paints containing fly ash also had greater 1-year average reflectivity readings than the NDOR standard paint in five of six cases. This is a good indication that the addition of fly ash may help increase the reflectivity of traffic paint.

2. After 1 year the two paints containing fly ash had approximately the same percentage of film failure. Y-M with ash performed better than Y-M without fly ash, yet P-S with fly ash performed worse than P-S without fly ash. The fly ash had both a positive and a negative effect on the percentage of film failure of the special paint, depending on the paint's manufacturer. Yet both paints with fly ash had approximately 40 percent less film failure than the NDOR standard. This is inconsistent evidence of the possibility that fly ash increases the durability of traffic marking paint. More field testing on a larger scale would probably lead to more conclusive comparisons.

3. None of the special paints met the NDOR drying time requirement of 15 min. The presence of fly ash did not significantly affect the drying time (no pickup) in the P-S paint, yet fly ash significantly increased the drying time in the Y-M paint.

4. As with film failure the effect of fly ash on the drying time was different for P-S and Y-M paints, depending on the manufacturer. Y-M with fly ash took 9 min longer to dry and also had a lower percentage of film failure than Y-M without fly ash. P-S with fly ash took only 1 min longer to dry but had a higher percentage of film failure than P-S without fly ash. Perhaps the drying time affected the film failure rate of the paint.

5. The paint containing fly ash performed well in the laboratory tests. In general it also showed promising results on the road, especially in the increase in reflectivity readings. This indicates the feasibility of using fly ash as a mineral filler in traffic marking paint.

RECOMMENDATIONS

The creative use of fly ash, an abundant artificial material, in technically sound applications such as traffic marking paints is a prudent endeavor. This is especially true with the pressing national concern for effective resource management. From the indications of the test program described here it is recommended that an extensive field test of traffic paint containing fly ash be conducted. The test should be done with various types of fly ash, both Class C and Class F from different sources, to determine how the different types perform in traffic paint.

ACKNOWLEDGMENTS

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REFERENCE

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PART 3

Structures

Feasibility of Incremental Benefit-Cost Analysis for Optimal Budget Allocation in Bridge Management Systems

FOAD FARID, DAVID W. JOHNSTON, BASHAR S. RIHANI, AND CHWEN-JINQ CHEN

A bridge management system (BMS) is a systematic framework that formalizes the decision-making process for bridge improvements. BMS decisions are analyzed at two levels: (a) at the bridge level, BMS determines the optimal improvement alternative for a bridge, and (b) at the system level, BMS supports decision makers in developing systemwide strategies for optimal use of the limited bridge improvement budgets. A major BMS module is an optimization algorithm for selecting the optimal combination of alternatives to maximize net benefits expected from the budget granted. The feasibility of implementing the Incremental Benefit-Cost (INCBEN) program for optimal allocation of the limited budgets to bridge improvement alternatives at the system level is investigated. Techniques and data exist for forecasting bridge agency costs and user costs, needed as input to INCBEN. Incremental benefits and costs are estimated from a base alternative. INCBEN ranks improvement alternatives in the decreasing order of their incremental benefit-cost ratios. These rankings are superior to those based on sufficiency ratings or level-of-service goals. INCBEN recommends near-optimal sets of bridge improvement alternatives under limited budgets. INCBEN selections under unlimited budgets are optimal and identical to the best alternatives selected by the economic analysis at the bridge level. INCBEN internally adds "do-nothing" alternatives to bridges without considering their consequences. This problem can be circumvented by manipulating the input data to ensure that the least-cost alternatives are funded first.

Because of the insufficient funding of bridge improvements, many bridges have become deficient in the United States (*1*). Budgets granted for bridge improvements are expected to be lower than budgets requested. Thus, a comprehensive bridge management system (BMS) is needed for consistent and efficient management of bridge improvements. BMS is a systematic framework that formalizes the decision-making process for bridge improvements. BMS decisions are analyzed at two levels: (a) at the bridge level, BMS determines the optimal improvement alternative for a bridge, and (b) at the system level, BMS supports decision makers in developing systemwide strategies for optimal use of the limited bridge improvement budgets (*2*).

Many states allocate their limited bridge improvement budgets by using sufficiency ratings or empirical formulas used to priority rank deficient bridges. Usually, a priority ranking formula translates physical conditions and level-of-service deficiencies into a priority index for every bridge. Bridges are then ranked according to their priority indexes for receiving improvement funding. Priority ranking formulas cannot select the optimal improvement alternative for a bridge, nor can they optimize net benefits expected from the bridge improvement budget granted. Thus, a systematic algorithm is needed for efficient allocation of the limited budget to deficient bridges.

Such an optimization algorithm is a major BMS module for selecting the optimal combination of alternatives that maximizes the performance standards and net benefits expected from the budgets granted. The primary objective is to investigate the feasibility of implementing the Incremental Benefit-Cost (INCBEN) program (*3*) for optimal allocation of the limited budgets to bridge improvement alternatives at the system level. More specifically, the objectives are to

1. Evaluate the theoretical framework, limitations, and implications using INCBEN as the optimization algorithm in BMS; and
2. Review techniques available for estimating bridge agency costs and benefits, and user costs and benefits.

ECONOMIC EVALUATION OF BRIDGE IMPROVEMENTS AT BRIDGE LEVEL

Economic analysis has been successfully applied to evaluating highway improvements. Application areas include highway and bus transit improvements (*4*), pavement management systems (*5*), and highway accident countermeasures (*6*). It can be used to evaluate bridge improvements as well. Economic analysis of highway improvements requires identifying all feasible alternatives and evaluating their consequences (*7*). Since any economic analysis deals with estimated future cash flows, it involves uncertainty. Economic analysis reduces the uncertainty surrounding the consequences of decisions. However, it does not dictate a decision; it is merely a management tool. Economic analysis at the bridge level evaluates the agency and user costs of the improvement alternatives to identify the alternative that maximizes net benefits expected, without violating budget constraints.

Improvement Alternatives

Three improvement alternatives are considered for deficient bridges: maintenance, rehabilitation, and replacement. The expected future costs of the "with and without" improvement are

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compared to determine "benefits." However, these benefits cannot justify a bridge at the system level. The need for a bridge is established at the bridge level instead.

The "with and without" concept requires adding the so-called do-nothing alternative to improvement proposals. If deficient bridges are left without improvement, however, they will deteriorate faster than if they were improved. Thus, the do-nothing alternative results in increased future agency and user costs. In short, consequences of the do-nothing alternative should be evaluated if it is considered. In general, only deficient bridges needing immediate improvements are considered at the system level. Thus, the do-nothing alternative is considered unfeasible. As a result, improvement benefits are determined by comparing the expected future costs of improvement alternatives with those of a base alternative.

Forecasting Input Data

Agency costs and benefits as well as user costs and benefits are the required input data to INCBEN. Agency benefits are defined as the present value of future agency cost savings due to the proposed improvements. Agency costs generally include periodic maintenance and rehabilitation costs over, and the replacement cost at the end of, the useful life of the bridge. User benefits are defined as user costs before improvement minus user costs after improvement (*8*).

Agency Costs

Three improvement alternatives are usually available for a bridge that is deficient but needed:

1. *New-bridge alternative* replaces the existing bridge with a new one having desirable levels of service. Regular maintenance needs increase with age to prevent future accelerated deterioration of the bridge. Eventually, a major rehabilitation is probably considered to reduce its level-of-service deficiencies. The cost profile for one replacement cycle of a new bridge is depicted in Figure 1 (*top*). The replacement-cycle cost of a new bridge is

$$\mathrm{RCC(NB)} = \mathrm{ICNB} + \sum_{t=1}^{\mathrm{SL}} \mathrm{ARMC}(t) * (P/F, r, t) + \mathrm{TRHC} * (P/F, r, n) \tag{1}$$

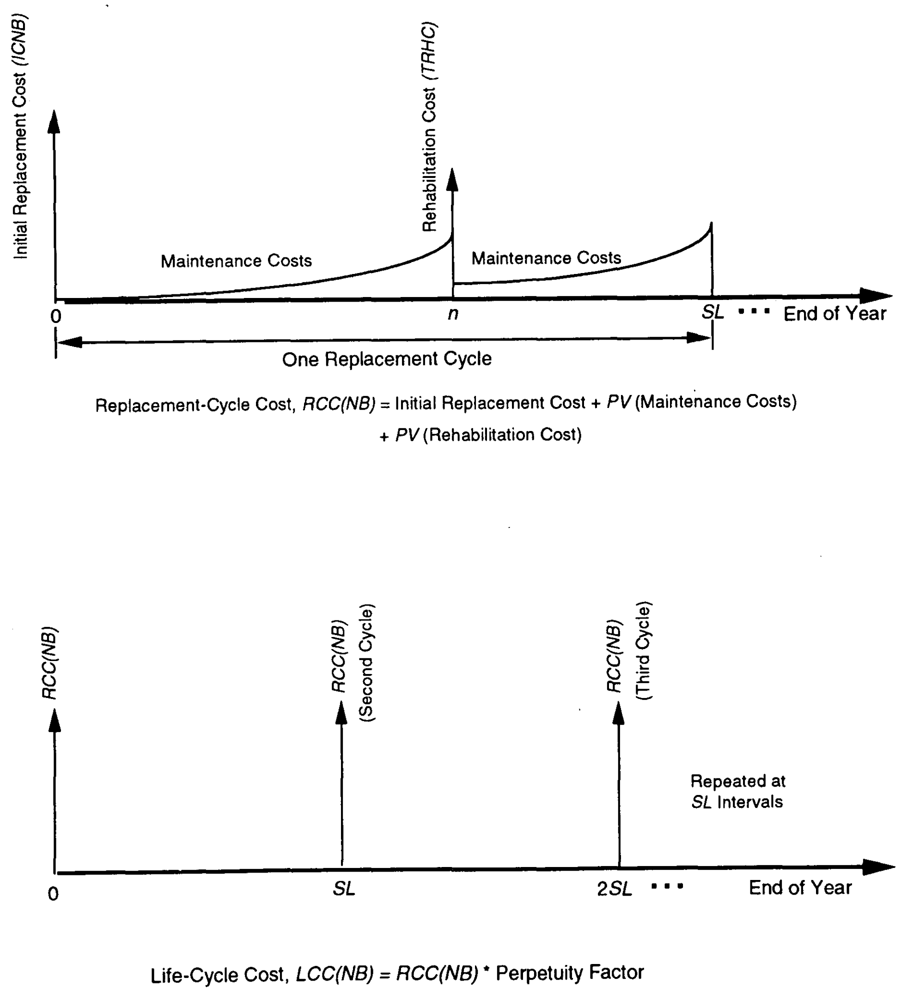

FIGURE 1 Life-cycle cost of replacement (new-bridge) alternative: *top*, cost profile for one replacement cycle; *bottom*, cost profile in perpetuity.

where

RCC(NB) = present value of one replacement-cycle cost of a new bridge;
ICNB = initial cost of a new bridge;
ARMC(t) = annual regular maintenance cost by end of year t;
TRHC = total rehabilitation cost at end of year n;
(P/F, r, t) = single-payment present-value factor;
r = discount rate or required rate of return; and
SL = expected service life of bridge (*9*).

If the first replacement cycle is followed by another cycle, RCC(NB) would be repeated at SL intervals. The cost profile for repeated replacement cycles in perpetuity (i.e., forever) is shown in Figure 1 (*bottom*). Thus, the life-cycle cost of the new-bridge alternative in perpetuity, LCC(NB), is

$$\text{LCC(NB)} = \frac{\text{RCC(NB)}}{1 - (1 + r)^{-\text{SL}}} \tag{2}$$

2. *Rehabilitation alternative* extends the bridge service life by several years before it is replaced, as shown in Figure 2 (*top*). Thus, the perpetual life-cycle cost of the rehabilitation alternative, LCC(RH), is

$$\text{LCC(RH)} = \text{ICRH} + \sum_{t=1}^{e} \text{ARMC}(t) * (P/F, r, t) + \text{LCC(NB)} * (P/F, r, e) \tag{3}$$

where ICRH is the initial cost of the rehabilitation alternative, and e is the extended service life of the bridge after rehabilitation (*9*).

3. *Maintenance alternative* maintains the deficient bridge until the end of its remaining life and then replaces it with a new bridge, as depicted in Figure 2 (*bottom*). The perpetual life-cycle cost of the maintenance alternative, LCC(MT), is

$$\text{LCC(MT)} = \sum_{t=1}^{\text{RL}} \text{ARMC}(t) * (P/F, r, t) + \text{LCC(NB)} * (P/F, r, \text{RL}) \tag{4}$$

where RL is the remaining life of the existing bridge (*9*).

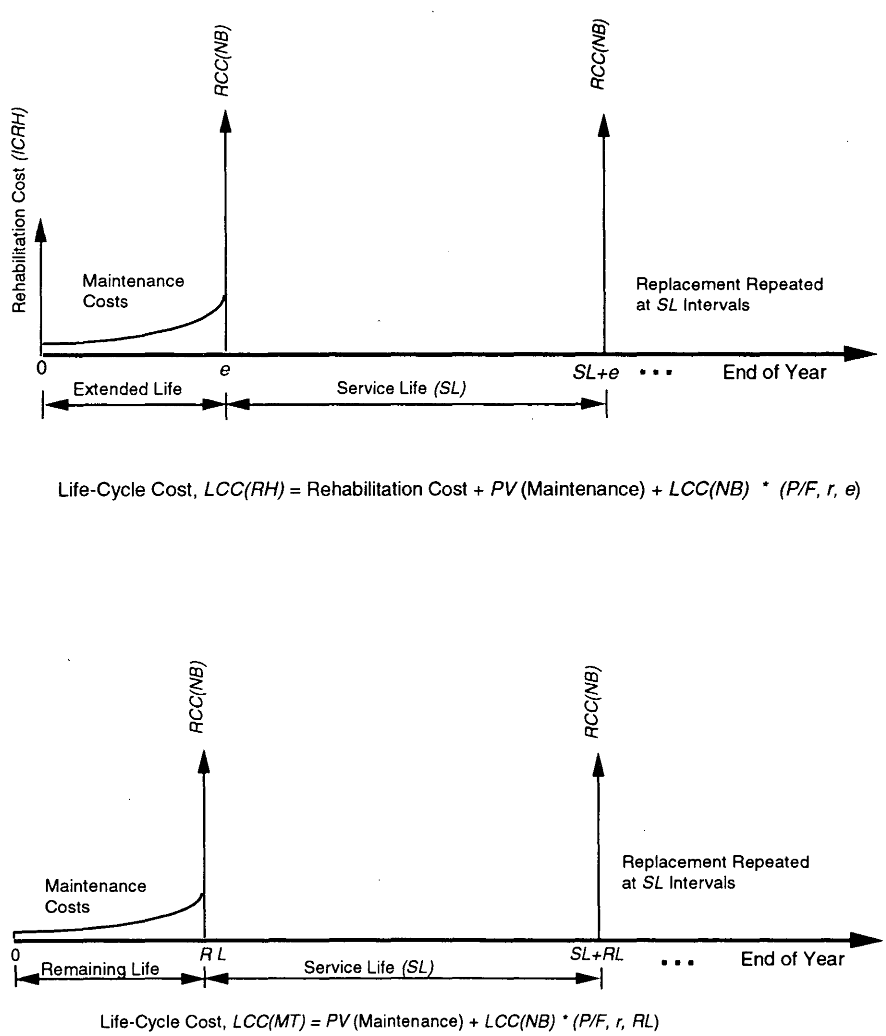

FIGURE 2 Life-cycle costs of rehabilitation (*top*) and maintenance (*bottom*) alternatives.

Agency Benefits

The agency net benefit of a bridge improvement alternative is the difference between the agency life-cycle cost of the base alternative and the agency life-cycle cost of the alternative in question. FHWA selected the new bridge alternative as the base alternative because it usually results in the highest life-cycle cost to the agency (*8*,pp.VI-11–VI-12).

The agency total benefit is "the present worth of future cost savings to the agency because of a bridge expenditure" (*8*,p.VI-11). Thus, for incremental benefit-cost analysis, the agency total benefit is the agency net benefit plus the initial cost of the bridge improvement alternative.

User Costs

User costs of deficient bridges are often due to narrow clear-deck width, low vertical clearance, poor alignment, and low load capacity. Bridges with narrow widths, low vertical clearances, or poor alignments have high accident probabilities. Bridges with low vertical clearances and low load capacities cause additional user costs to detoured vehicles. Chen and Johnston (*10*,p.122) estimated the annual user cost of a deficient bridge as

$$\mathrm{AURC}(t) = 365 * \mathrm{ADT}(t) * [(C_{\mathrm{ADW}} + C_{\mathrm{AAL}} + C_{\mathrm{ACL}}) * U_{\mathrm{AC}} + (C_{\mathrm{DCL}} * U_{\mathrm{DCL}} + C_{\mathrm{DLC}} * U_{\mathrm{DLC}}) * \mathrm{DL}] \quad (5)$$

where

$\mathrm{AURC}(t)$ = annual user cost of existing bridge during year t;
$\mathrm{ADT}(t)$ = average daily traffic over bridge during year t;
C_{ADW} = proportion of vehicles incurring accident costs due to a deck-width deficiency;
C_{AAL} = proportion of vehicles incurring accident costs due to an alignment deficiency;
C_{ACL} = proportion of vehicles incurring accident costs due to a vertical clearance deficiency;
U_{AC} = average unit cost of vehicle accidents on bridges ($/accident);
C_{DCL} = proportion of vehicles detoured due to a vertical clearance deficiency;
U_{DCL} = average unit cost for vehicles detoured due to a vertical clearance deficiency ($/mi);
C_{DLC} = proportion of vehicles detoured due to a load capacity deficiency;
U_{DLC} = average unit cost for vehicles detoured due to a load capacity deficiency ($/mi); and
DL = detour length (mi).

C_{ADW}, C_{AAL}, C_{ACL}, and C_{DCL} generally remain constant during the service life unless bridge deficiencies are corrected. If the load capacity deteriorates, however, the proportion of vehicles detoured (C_{DLC}) will increase with time. For a given level-of-service deficiency, bridges with higher ADTs cause proportionally higher user costs because of higher numbers of detours and accidents. Chen and Johnston (*10*) estimated these proportions as functions of the bridge functional classification and level-of-service deficiencies. Average unit costs of vehicles detoured and those of accidents were also estimated.

User Benefits

User benefits are interpreted as the reduced user costs due to the initial cost of a bridge improvement alternative. More generally, user benefits are the difference between the user life-cycle cost of the base alternative and that of the alternative under consideration.

Economic Decision Criteria

Four decision criteria are used for evaluating highway improvements (*8*,pp.VI-3–VI-30). These criteria can also be used to evaluate bridge improvement alternatives: (a) first-cost analysis, (b) life-cycle cost analysis, (c) simple benefit-cost analysis, and (d) incremental benefit-cost analysis. The selected criteria should provide analysts with correct and consistent results to

1. Determine the economic desirability of proposed improvement alternatives, and
2. Compare merits of these mutually exclusive options to select the most desirable alternative.

Farid et al. (*9*,pp.11–18) stated that incremental benefit-cost analysis satisfied both requirements.

Incremental Benefit-Cost Analysis

The incremental (marginal) benefit-cost ratio is "the extra benefits of advancing from one improvement level to the next divided by the corresponding extra costs" (*8*,p.VI-16). Thus, to have a justifiable investment increment, its incremental benefit-cost ratio must be at least 1. To apply the incremental benefit-cost analysis for selecting an alternative for an independent project under no budget constraints:

1. Sort all mutually exclusive alternatives in increasing order of their initial costs.
2. Tentatively accept the first economical least-cost alternative.
3. Calculate the incremental benefit-cost ratio for the second least-cost alternative. If the ratio equals or exceeds 1, replace the alternative accepted previously with the current alternative. This now becomes the base alternative for comparison with the least-cost alternative.
4. Repeat Step 3 for all alternatives.
5. Select the highest-cost alternative with an incremental benefit-cost ratio of at least 1. The incremental benefit-cost analysis seeks the maximum net benefit by justifying each cost increment.

For independent projects under budget limits, the last step is changed to

5. Select the highest-cost alternative that satisfies budgetary constraints and has an incremental benefit-cost ratio of at least 1 (*4*,p.140;*11*).

Incremental Benefit-Cost Analysis Applied to Hypothetical Bridge

The incremental benefit-cost analysis can select the most desirable improvement alternative for a bridge in tabular form. Table 1

TABLE 1 Incremental Benefit-Cost Ratios for Improvement Alternatives of a Hypothetical Bridge

Alternative[a]	First Cost $1,000	Total Benefit $1,000	ΔC $1,000	ΔB $1,000	$\Delta B/\Delta C$	Net Benefit $1,000
(1)	(2)	(3)	(4)	(5)	(6)=(5)/(4)	(7)=(3)-(2)
M	1	62	-	-	-	61
r	40	206	39	144	3.69	166
R[b]	50	217	10	11	1.10	167
N	80	238	30	21	0.70	158

[a] *M* stands for Maintenance, *r* or *R* for Rehabilitation, *N* for New bridge (replacement), and *C* for Closure

[b] Best Alternative

gives benefits and first costs of four improvement alternatives for a hypothetical bridge. The alternatives are listed in ascending order of their first costs. The incremental benefit-cost ratios are calculated. Step 5 selects Alternative *R* under no budget constraints. *R* is the highest-cost alternative with an incremental benefit-cost ratio of at least 1.

ECONOMIC EVALUATION OF BRIDGE IMPROVEMENTS AT SYSTEM LEVEL

The bridge-level analysis outcome is important as input to economic analysis at the system level. The objective is to select improvement alternatives that yield the highest net benefits expected under the budget granted. Economic analysis at the system level can generate a priority ranking of improvement alternatives. It can also analyze the sensitivity of the results to bridge improvement policies.

Priority Ranking and Budget Allocation

Only bridges needing improvement should be considered for budget allocation. Such screening should considerably reduce the size of the analysis. To set improvement priorities and to optimize the budget allocation, do the following:

1. Establish level-of-service goals or standards for a safe and functional operation.
2. Determine the deficient bridges, with attributes below the level-of-service goals, needing improvement in this period.
3. Determine the improvement alternatives for deficient bridges and their costs, benefits, and the remaining or extended service lives.
4. Allocate part of the budget granted to the least-cost improvement alternatives for all bridges deemed deficient in Step 2 in descending order of their benefit-cost ratios (this approximate ranking formula should be adequate for those rare periods in which not all the least-cost alternatives can be funded).
5. Obtain a priority ranking of the remaining alternatives (those not funded in Step 4) in descending order of their incremental benefit-cost ratios.
6. Allocate the remaining budget, if any, to increase the improvement levels of these bridges on the basis of the priority ranking determined in Step 5.

First costs are used to calculate the incremental benefit-cost ratios necessary for ranking the alternatives. This procedure is appropriate for allocating a one-period budget, usually no longer than 5 years. First costs, however, are not suitable for multiperiod budget allocation (*9*).

INCBEN Algorithm

INCBEN is designed to allocate a granted budget such that net benefits expected from improvement alternatives are maximized (*3*). It generates a decreasing order of incremental benefit-cost ratios as a priority ranking. An initial set of locations, along with the best possible alternative at each location, is then selected. A switching rule is used to induce "marginal" improvements to the initial solution (*3*,p.2). The input data required for applying INCBEN in BMS are as follows:

1. Identification of every improvement alternative and its bridge number;
2. Initial cost of every improvement alternative;
3. Total benefits expected from every improvement alternative; and
4. Granted budget.

INCBEN processes the input data by performing these steps:

1. Alternatives for every bridge are sorted in the increasing order of their first costs.
2. If two or more alternatives for a deficient bridge have the same initial cost, only the alternative with the highest total benefit is retained.

3. The incremental benefit-cost ratios for all bridges are calculated.

4. All alternatives with incremental benefit-cost ratios of 1 or less are discarded.

5. The incremental benefit-cost ratios must be in descending order. If an alternative's ratio exceeds that of the previous, less-expensive alternative for the same bridge, the incremental benefits and costs for the two alternatives are combined. The overall ratio for the more expensive alternative will be in decreasing order.

6. All deficient bridges in the data base are ranked in the descending order of their adjusted incremental benefit-cost ratios.

7. INCBEN selects the highest-ranking alternative and proceeds downward until the granted budget is exhausted. When a bridge alternative is selected, it replaces the less-expensive alternative previously selected for the same bridge. The algorithm may skip an alternative, and consider the next less-costly alternatives, *if* it cannot be funded with the remaining budget.

8. When the selection of an alternative causes the cumulative cost to exceed the budget, INCBEN replaces the last selected alternative with additional increments until the budget is exhausted. The algorithm compares the initial and revised improvement-alternative sets and selects the one with higher "total benefits," although it should select the solution with higher *net benefits.*

INCBEN internally adds do-nothing alternatives to all bridges. It considers this alternative's benefits to be zero, which is not necessarily true for bridges. Hence, INCBEN should be modified to exclude the do-nothing alternative before applying it in BMS.

INCBEN Applied to Four Hypothetical Bridges

Table 2 gives four hypothetical bridges along with their alternatives, first costs, and total benefits. These data are processed in the same order as INCBEN to illustrate its algorithm:

1. Alternatives for each bridge are sorted in the increasing order of their first costs.

2. For Bridge 2, Alternatives *R* and *N* have the same initial cost. Thus, Alternative *R*, with the lower benefit, is eliminated.

3. The incremental benefit-cost ratios for remaining alternatives are calculated. INCBEN considers the simple benefit-cost ratio for every least-cost alternative to be an incremental ratio. This adds a do-nothing alternative with zero first-cost and benefits to every bridge. But such an alternative may have some benefits or be unacceptable.

4. Alternative *N* for Bridge 4 is deleted because its incremental benefit-cost ratio is less than 1.

5. Alternative *N* for Bridge 1 has a 3.44 incremental benefit-cost ratio (R_{1N}), higher than that of Alternative *R* (R_{1R} = 3.41). A combined incremental benefit-cost ratio (R'_{1N}) is calculated to ensure its decreasing order:

$$R'_{1N} = \frac{\Delta B_{1R} + \Delta B_{1N}}{\Delta C_{1R} + \Delta C_{1N}} = \frac{23{,}900 + 8{,}600}{7{,}000 + 2{,}500} = 3.42 \qquad (6)$$

The same applies to Alternatives *M* and *R* for Bridge 3.

6. Alternatives are ranked in decreasing order of their adjusted incremental benefit-cost ratios, as shown in Table 3. 3-*R* now combines the cost and benefit increments for 3-*R* and 3-*M*. Similarly, 1-*N* represents 1-*N* and 1-*R* combined. 3-*M* and 1-*R* are still included, but their incremental costs and benefits are not added again to the cumulative cost or benefit. This is because their costs and benefits are included in the combined entries 3-*R* and 1-*N*. 3-*M* will be considered only if 3-*R* cannot be funded without exceeding the budget. The same applies to 1-*R* and 1-*N*. For example, if the budget is between $13,600 and $23,100, 1-*N* cannot be funded. Instead, 1-*R* should be funded if the budget granted is $20,600 to $23,100.

7. Alternatives are selected by adding the incremental costs until the granted budget is exhausted. For example, 2-*N*, 4-*R*, and 3-*R* are tentatively selected for a $12,000 budget. These replace other alternatives previously selected for the same bridges. The allocated budget is $11,100, with a $900 balance. The total benefits expected are $64,500. No alternative for Bridge 1 is selected. Direct INCBEN application may leave out several bridges.

8. The "switching rule" now becomes active. The last added alternative, 2-*N*, is tentatively replaced by the next alternatives until the budget is exhausted. Only 1-*M* can be added. The total benefit expected from 1-*M*, 4-*R* and 3-*R* is reduced to $52,500, for a cumulative cost of $9,000. Thus, the initial set of 2-*N*, 4-*R*, and 3-*R* with higher total benefits is adopted. If 3-*R*, 4-*R*, and 1-*M* were adopted, Bridge 2 would have received no improvement because the switching rule drops 2-*N* without readopting 2-*M*.

Unlimited Budget Forecasts

Budget requests initially assume an unlimited budget and reflect the actual conditions of bridges. When the granted budget falls below these requests, the bridge program is "underfunded." The budget granted must then be allocated among improvement alternatives. INCBEN can forecast the optimal unlimited budget by assuming a huge budget to ensure all alternatives with incremental benefit-cost ratios of at least 1 are selected. Alternatively, the incremental benefit-cost analysis at the bridge level is applied to every bridge independently, as demonstrated in Table 1. The sum of the initial costs of all "best" alternatives represents the optimal unlimited budget forecast.

EVALUATION OF INCBEN PROGRAM

INCBEN systematically ranks all improvement alternatives in descending order of their incremental benefit-cost ratios. The program then selects a set of bridge improvement alternatives that will nearly maximize the expected net benefits. McFarland and Rollins (*12*) indicated that INCBEN performed satisfactorily.

Theoretical Framework

The incremental benefit-cost analysis at the bridge level produces optimal results, given a discount rate. Difficulties arise when several bridges with multiple improvement alternatives are evaluated under budget constraints. To find an optimal combination of improvement alternatives, all possible alternative combinations for various bridges must be compared. A statewide BMS covering hundreds of bridges, each with several improvement alternatives, requires comparing a large number of combinations. The total number of combinations is $(X_1)(X_2)(X_3) \ldots (X_i) \ldots (X_n)$, where X_i is the number of proposed alternatives for Bridge *i* and *n* is the

TABLE 2 Incremental Benefit-Cost Ratios for Sample Bridges

Bridge i	Alternative[a] j	First Cost \$1,000	Total Benefit \$1,000	ΔC \$1,000	ΔB \$1,000	$R_{ij}=\Delta B/\Delta C$	R'_{ij}
(1)	(2)	(3)	(4)	(5)	(6)	(7)=(6)/(5)	(8)
			(a) Four Hypothetical Bridges				
1	*M*	2.5	10.0	2.5	10.0	4.00[c]	
	R	9.5	33.9	7.0	23.9	3.41	
	N	12.0	42.5	2.5	8.6	3.44	3.42
2	*M*	2.0	11.0	2.0	11.0	5.50[c]	
	R[b]	4.6	20.0	---	----	----	
	N	4.6	22.0	2.6	11.0	4.23	
3	*M*	1.5	9.0	1.5	9.0	6.00[c]	
	R	5.0	33.0	3.5	24.0	6.86	6.60[c]
4	*M*	0.5	4.5	0.5	4.5	9.00[c]	
	R	1.5	9.5	1.0	5.0	5.00	
	N[b]	2.5	10.4	1.0	0.9	0.90	
			(b) Five North Carolina Bridges				
05125	*M*[b]	2	-212	2	-212	-106.00[c]	
	R	40	210	40	210	5.25[c]	
	N	283	647	243	437	1.80	
61010	*M*	3	114	3	114	38.00[c]	
	R	86	278	83	164	1.98	
	N[b]	145	312	59	34	0.58	
73411	M	17	2,304	17	2,304	135.53[c]	
	R	86	2,667	69	363	5.26	
	N[b]	3,600	5,743	3,514	3,076	0.88	
89034	*M*	9	250	9	250	27.78[c]	
	R	72	524	63	274	4.35	
	N[b]	319	645	247	121	0.49	
97060	M	5	365	5	365	73.00[c]	
	R[b]	300	584	295	219	0.74	
	N	560	1,084	555	719	1.30	

[a] *M* stands for Maintenance, *R* or *r* for Rehabilitation, *N* for New bridge (replacement), and *C* for Closure

[b] Alternative Deleted

[c] Simple benefit-cost ratio for this alternative

number of bridges. Apart from using mathematical programming to solve the combinatorial problem, INCBEN can provide near-optimal solutions. INCBEN deletes all bridge improvement alternatives with incremental benefit-cost ratios of 1 or less. All alternatives with incremental ratios of at least 1 are economical. Thus, INCBEN should be modified to delete only alternatives with incremental benefit-cost ratios of less than 1.

INCBEN was developed for allocating safety improvement budgets. It initially assumes no improvement at all "locations." Funds are then allocated to successively higher improvement increments in the decreasing order of their incremental benefit-cost ratios. If the budget is exhausted before any improvement level is funded for a location, the do-nothing alternative is selected. Its benefits and first cost are assumed to be 0. The do-nothing alternative is generally unacceptable in BMS. Even if it were acceptable, its consequences would not necessarily be 0.

Submarginal alternatives can often replace one or more previously selected alternatives to obtain greater "benefits" without exceeding the budget (*6*,pp.301–302). Thus, INCBEN is expected to select near-optimal sets of improvement alternatives. This may occur because the optimal set may contain one or more alternatives with incremental benefit-cost ratios lower than those of the last alternative selected by INCBEN. McFarland et al. (*3*,p.4) subsequently added a switching rule to the INCBEN algorithm. This rule replaces the last accepted cost increment with other increments until the budget is exhausted. The total benefits of the initial and revised solutions are compared, and the solution with the greatest total benefits is selected. Although this rule may improve

TABLE 3 Alternatives Ranked in Decreasing Order of Incremental Benefit-Cost Ratios

Bridge i	Alternative[a] j	First Cost \$1,000	Total Benefit \$1,000	ΔC \$1,000	$\Delta B/\Delta C$	Budget Allocated \$1,000
(1)	(2)	(3)	(4)	(5)	(6)	(7)
		(a) Four Hypothetical Bridges				
4	*M*	0.5	4.5	0.5	9.00[c]	0.5
3	*R*	5.0	33.0	5.0	6.60[c]	5.5
3	*M*[b]	1.5	9.0	1.5[b]	6.00[c]	---
2	*M*	2.0	11.0	2.0	5.50[c]	7.5
4	*R*	1.5	9.5	1.0	5.00	8.5
2	*N*	4.6	22.0	2.6	4.23	11.1
1	*M*	2.5	10.0	2.5	4.00[c]	13.6
1	*N*	12.0	42.5	9.5	3.42	23.1
1	*R*[b]	9.5	33.9	7.0[b]	3.41	---
		(b) Five North Carolina Bridges				
73411	*M*	17	2,304	17	135.53[c]	17
97060	*M*	5	365	5	73.00[c]	22
61010	*M*	3	114	3	38.00[c]	25
89034	*M*	9	250	9	27.78[c]	34
73411	*R*	86	2,667	69	5.26	103
05125	*R*	40	210	40	5.25[c]	143
89034	*R*	72	524	63	4.35	206
61010	*R*	86	278	83	1.98	289
05125	*N*	283	647	243	1.80	532
97060	*N*	560	1,084	555	1.30	1,087

[a] *M* stands for Maintenance, *r* or *R* for Rehabilitation, *N* for New bridge (replacement), and *C* for Closure

[b] Included in entry immediately preceding; not added separately to cumulative costs and benefits

[c] Simple benefit-cost ratio for this alternative

the initial solution, it does not guarantee that the optimal solution under the budget constraint is selected. Further, the switching rule should compare net benefits, not total benefits, as illustrated later.

Limitations of Incremental Benefit-Cost Analysis

Although the algorithm is straightforward, many calculations and checks are required for large bridge systems. INCBEN can process up to 85 bridges, each with up to eight improvement alternatives (*3*). These limits can be increased to fit the available computer hardware. Results should be examined carefully because of theoretical and other limitations of the incremental benefit-cost analysis.

General Limitations

Economic analysis and resource allocation organize information that is useful to decision makers (*13*,p.41), but they cannot account for all available information. Winfrey (*13*,p.42) classified other factors that need be considered as (a) road-user nonpriceable, personal preferences; and (b) nonuser socioeconomic consequences. Thus, any incremental benefit-cost analysis is merely a management tool. Applied in BMS, its underlying assumptions and limitations are as follows:

1. Bridge improvement needs remain constant. Only bridges deemed deficient at the time compete for funds, but deficient bridges and their deficiency types and levels change with time.
2. Statistical techniques are used to forecast the bridge remaining life, extended life, and service life due to improvement alternatives. Future cost profiles and trends are also forecast to model the costs of improvement alternatives under uncertainty.
3. A single risk-adjusted discount rate is used for calculating the agency and user costs (*14*). This fixed rate is assumed to be known and is not expected to change significantly in the future.

These assumptions make the incremental benefit-cost analysis suitable for allocating budgets over a short horizon, perhaps 1 to 5 years (*8*,p.VI-39). The quality of the results can be improved by conducting sensitivity analysis, as an intermediate step between economic analysis based on best estimates and the final decision (*15*,p.236). In another paper in this Record, Farid et al. analyze sensitivity of INCBEN results to the discount rate, remaining life, and service life of bridges.

Limitations of INCBEN Application in BMS

INCBEN has several features that may produce improper results. These features were discovered through experimentation and by examining the algorithm.

First, INCBEN does not necessarily select the "optimal" set of improvement alternatives. The optimal set should maximize net benefits expected from alternatives selected under the limited budget. To illustrate, Table 2 presents initial costs and total benefits expected from improvement alternatives for five North Carolina bridges. The incremental benefit-cost ratios are estimated by INCBEN. Alternatives 05125-*M*, 61010-*N*, 73411-*N*, 89034-*N*, and 97060-*R* with incremental benefit-cost ratios of 1 or less are dropped. Table 3 ranks the remaining alternatives in decreasing order of their incremental benefit-cost ratios.

Under a $205,000 budget, INCBEN selects 61010-*M*, 73411-*R*, 89034-*R*, and 97060-*M*, as listed in Table 4. These are the final INCBEN selections after the switching rule replaces alternatives 05125-*R* and 89034-*M* by 89034-*R*. The budget allocated (cumulative first cost) is $166,000, total benefits expected are $3,670,000, and net benefits expected are $3,504,000.

TABLE 4 Alternatives Selected for Several Levels of Budget Granted

Bridge No. (1)	Budget Granted ($1,000) 205 (2)	750 (3)	1,000 (4)
05125	-	*N*	-
61010	*M*	*R*	*R*
73411	*R*	*R*	*R*
89034	*R*	*R*	*R*
97060	*M*	*M*	*N*
Expected Total Benefits	3,670	4,481	4,553
Budget Allocated	166	532	804
Expected Net Benefits	3,504	3,949	3,749
Excess Budget	39	218	196

M stands for Maintenance, *R* or *r* for Rehabilitation, *N* for New bridge (replacement), and *C* for Closure

By inspection, Alternative 05125-*R* can replace 61010-*M*. This set comprises 05125-*R*, 73411-*R*, 89034-*R*, and 97060-*M*, with $3,766,000 in total benefits and $203,000 in cumulative first costs. The $3,563,000 net benefits expected from this set are greater than the $3,504,000 expected from the INCBEN selections. The revised set leaves out Bridge 61010, which is usually unacceptable. But INCBEN left Bridge 05125 without improvement.

Such complications become particularly troublesome where alternatives, especially those near the budget limit, vary considerably in cost, are quite costly in relation to the budget, and have widely varying incremental benefit-cost ratios. Selecting proper submarginal alternatives in highway safety programs is not critical because most safety-program budgets are large relative to the first cost of any alternative. Thus, INCBEN is expected to provide near-optimal solutions (*6*,p.302).

The bridge improvement budgets are also large in relation to the first costs of improvement alternatives. However, wide variations in costs and incremental benefit-cost ratios of alternatives are expected. Thus, their effects on INCBEN application in BMS may be significant. For example, the $40,000 first cost of 05125-*R* is large in relation to the $205,000 budget because only five bridges are analyzed. This results in $39,000 of excess budget and clearly suboptimal INCBEN selections. Later in this Record, Farid et al. demonstrate that INCBEN produces near-optimal results even for as few as 25 bridges.

Second, the switching rule compares total benefits of the initial and revised solutions (*3*,p.26). But the algorithm's objective is to maximize net benefits under budget constraint (*3*,p.1; *12*,p.9). Moreover, INCBEN's Step 8 (*3*,pp.4–5) contains conflicting statements on the comparison basis of the switching rule.

This problem is best illustrated by reanalyzing bridges listed in Tables 2, 3, and 4 at two more budget levels. At $750,000, INCBEN selects the optimal set 05125-*N*, 61010-*R*, 73411-*R*, 89034-*R*, and 97060-*M* with a $532,000 cumulative first cost. Total benefits of $4,481,000 and net benefits of $3,949,000 are expected. Under $1,000,000, INCBEN selects 61010-*R*, 73411-*R*, 89034-*R*, and 97060-*N*. An $804,000 cumulative first cost and $4,553,000 total benefits are expected. The $3,749,000 net benefits expected under $1,000,000 budget are lower than $3,949,000 under $750,000 budget because of the switching rule. The initial solution under the $1,000,000 budget is the same optimal set as the solution under the $750,000 budget. Since the initial $4,481,000 total benefits are less than the revised $4,553,000, the switching rule selects the revised solution. This decision is not cost-effective, however, because the correct criterion for comparing alternatives is net benefits. This criterion would have properly selected the initial solution. Thus, the switching rule should be modified to compare net benefits.

Finally, INCBEN internally adds a do-nothing alternative, with a zero first cost and zero total benefits, to all "locations" because the simple benefit-cost ratios of the least-cost alternatives are taken as incremental ratios (*3*,p.26). The do-nothing alternative may be acceptable in evaluating highway accident countermeasures, but it is unacceptable in BMS for two reasons. First, INCBEN may leave a deficient, unsafe bridge without improvement. Second, INCBEN assumes that benefits of the do-nothing alternative are 0. This is inconsistent with the economic analysis principles requiring that consequences of alternatives be incorporated (*15*,pp.9–10).

If the do-nothing alternative is acceptable, the remaining lives of the deficient bridges left without improvement should be estimated to forecast the associated benefits. No data exist for estimating the remaining life of a bridge receiving no improvement. Thus, INCBEN should be modified to consider alternatives entered by bridge managers only. The do-nothing problem can be avoided by ensuring that all least-cost alternatives are funded first. The budget balance can then be allocated to further improve these deficient bridges.

Implementation and Advantages of Incremental Benefit-Cost Program

INCBEN application in allocating limited budgets to bridge improvement alternatives may result in considerable savings over priority ranking formulas and simple benefit-cost ratios. INCBEN generates near-optimal solutions and offers these improvements over existing practices:

1. Explicit consideration of the time value of money,
2. Systematic allocation of limited budgets among improvement alternatives,
3. Computerized algorithm capable of evaluating many alternatives,
4. Maximized net benefits expected from alternatives under limited budget, and
5. Incremental benefit-cost ratio as criterion for selecting improvement alternatives under budget constraint.

INCBEN produces a superior priority ranking of the improvement alternatives in the decreasing order of their incremental benefit-cost ratios. The INCBEN ranking is based on economic principles and prescribes specific improvement alternatives for deficient bridges.

Difficulties in implementing INCBEN are in estimating the user costs and the extended lives of bridges due to improvement alternatives. Procedures exist for estimating the remaining, and the extended, lives (*8*,pp.IV-1–VI-7). Improved estimates are expected as bridge data bases are upgraded and states' resources are pooled (*8*,p.VI-41). States may link their data bases to allow automated estimates of the detoured traffic, accident costs, travel time, and vehicle operating costs that account for user costs. Automation may facilitate the preparation of INCBEN input data. Estimating costs and benefits of improvement alternatives requires numerous calculations and checks. This is particularly cumbersome in sensitivity analysis, requiring repeated calculations and checks for ranges of input variables.

Once costs and benefits expected from all improvement alternatives are estimated, running INCBEN, designed for batch input, is straightforward. The INCBEN documentation describes data requirements and program testing procedures (*3*) and the program documents solutions in clear tables. The output includes the input echoprints, incremental benefit-cost ratios, deleted alternatives, alternatives ranked in decreasing order of their incremental benefit-cost ratios, and the improvement alternatives selected under the granted budget. Users may become familiar with the INCBEN algorithm by comparing their manual solutions to small examples with the INCBEN results.

CONCLUSIONS

Implementing a revised INCBEN program for optimal budget allocation in BMS appears feasible. Major conclusions include the following:

1. Techniques and data exist for forecasting bridge agency costs and user costs, which are the required INCBEN input data. Incremental benefits and costs are estimated from a base alternative. Thus, the input data are meaningful only for comparing improvement alternatives.

2. INCBEN ranks improvement alternatives in the decreasing order of their incremental benefit-cost ratios. INCBEN rankings are superior to those based on sufficiency ratings or level-of-service goals. INCBEN generates rankings based on economic principles and recommends specific alternatives for deficient bridges.

3. INCBEN recommends near-optimal sets of bridge improvement alternatives, which nearly maximize net benefits, under limited budgets. INCBEN selections under unlimited budgets are optimal and identical to alternatives selected by bridge-level economic analysis.

4. INCBEN internally adds do-nothing alternatives without considering their consequences. This is inappropriate for BMS applications. The problem can be overcome by manipulating the input data to ensure that the least-cost alternatives are funded first. The budget balance can then be allocated to further improve deficient bridges in the decreasing order of their incremental benefit-cost ratios.

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Application of Incremental Benefit-Cost Analysis for Optimal Budget Allocation to Maintenance, Rehabilitation, and Replacement of Bridges

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Bridge improvement funding in the United States has been insufficient for years. Thus, a systematic algorithm for efficient allocation of limited budgets to deficient bridges is needed, as part of a comprehensive bridge management system. Application of one such algorithm, the Incremental Benefit-Cost (INCBEN) program, for optimal allocation of the limited budgets to bridge improvement alternatives at the system level is investigated. INCBEN is applied to a sample of highway bridges to determine a near-optimal set of improvement alternatives. The sample consists of 25 in-service bridges in North Carolina with varying structural or functional deficiencies. Selection of the near-optimal bridge improvement alternatives under several levels of budget granted; sensitivity of budget-allocation results to the discount rate, remaining life, and service life; and comparison of results with those of the sufficiency rating methods are described.

Due to the insufficient funding of bridge improvements over the years, many bridges have become deficient in the United States (*1*). Budgets granted for bridge improvements are expected to be lower than budgets requested by most agencies. Thus, a comprehensive bridge management system (BMS) is needed for consistent and efficient management of bridge improvement activities. BMS is a systematic framework that formalizes the decision-making process for bridge improvement. BMS decisions are analyzed at two levels: (a) at the bridge level, BMS determines the optimal improvement alternative for a bridge, and (b) at the system level, BMS supports decision makers in developing systemwide strategies for making optimal use of the bridge improvement budgets (*2*).

A major BMS module is an optimization algorithm that maximizes the performance standards and the net benefits expected from the budgets granted. Application of one such algorithm, the Incremental Benefit-Cost (INCBEN) program (*3*), for optimal allocation of the limited budgets to bridge improvement alternatives at the system level is investigated. The input data to INCBEN are random variables because they are forecasts of future events. Thus, sensitivity of the INCBEN results to major input data is analyzed.

INCBEN PROGRAM

INCBEN can allocate a limited budget to maximize the net benefits expected from improvement alternatives. Farid et al. have provided a detailed description of INCBEN (*4*; another paper in this Record), and a complete description of INCBEN has also been presented by McFarland et al. (*3*).

INCBEN is used to rank a sample of 25 North Carolina bridges under budget constraints. The input data required for applying INCBEN in BMS are

1. Identification of every improvement alternative and its bridge number,
2. Initial cost of every alternative,
3. Total benefits expected from every alternative, and
4. Granted budget.

Forecasting Input Data

Farid et al. (*4*; another paper in this Record) described the techniques used for developing the improvement alternatives and their life-cycle costs. Agency and user costs were estimated on the basis of approaches developed by Chen and Johnston (*5*). Replacement is adopted as the base alternative for forecasting agency benefits because it usually results in the highest life-cycle cost to the agency (*6*). Thus, the agency net benefit of a bridge improvement alternative is defined as the difference between the agency life-cycle cost of the base alternative and that of this alternative. The agency total benefits of an improvement alternative is equal to its agency net benefit plus its initial cost. Column 3 of Table 1 presents the agency total benefits of the 25 North Carolina bridges.

User costs are due to level-of-service deficiencies in load capacity, clear deck width, alignment, and vertical clearance. User benefits of a bridge improvement alternative is interpreted as the difference between the user life-cycle cost of the base alternative and that of this alternative. The most cost-effective alternative to the agency, representing the last investment increment with an incremental benefit-cost ratio greater than 1, is taken as the base alternative (*4,6*). For example, the agency incremental benefit-cost ratios for Bridge 05125 are estimated in Table 2. Rehabilitation is the most cost-effective alternative because it is the last increment with an incremental benefit-cost ratio greater than 1. Thus, user benefits will be estimated using rehabilitation as the base alternative—that is, user benefits of an improvement alternative are the difference between the user life-cycle cost of the rehabilitation alternative and that of this alternative. Column 4 of Table 1 gives the agency and user benefits for the 25 bridges.

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INCBEN Analysis

INCBEN cannot be applied directly in BMS because it automatically adds "do-nothing" alternatives. Its algorithm should eventually be modified to exclude the do-nothing alternative (*4*, other paper by Farid et al. in this Record). The INCBEN input data are manipulated so that the existing INCBEN could produce correct results.

Data manipulation essentially ensures that the least-cost bridge improvement alternatives are funded first. The modified marginal input data are compiled by subtracting the cost and benefits of the least-cost alternative for every bridge from the corresponding cost

TABLE 1 Costs and Benefits for All Bridge-Improvement Alternatives ($ thousands)

Bridge[a]	Initial Cost	Agency Benefit	Agen. & User Benefits	Bridge[a]	Initial Cost	Agency Benefit	Agen & User Benefits
00210*M*	165	827	827	59100*r*	163	464	464
R	742	819	2641	*R*	462	450	657
N	895	895	2752	*N*	485	485	692
05125*M*	2	92	-212	58102*r*	173	505	505
R	40	210	210	*R*	496	485	694
N	283	283	647	*N*	521	521	730
08052*M*	4	548	548	58128*r*	202	462	462
N	520	520	905	*R*	297	454	585
				N	471	471	602
10381*M*	429	1703	1703				
R	1550	2512	-3884	61010*M*	3	114	114
N	2650	2650	9268	*R*	86	112	278
				N	145	145	312
17001*M*	70	1600	1600				
R	725	699	724	73411*M*	17	994	2173
N	1400	1400	1560	*R*	86	2667	2667
				N	3600	3600	5743
29058*M*	7	363	363				
R	627	479	931	75171*M*	4	150	150
N	548	548	1000	*R*	116	236	2185
				N	444	444	2549
45009R	6	1747	1747				
M	28	1207	16564	80173*M*	1	80	80
N	3755	3755	19162	*R*	4	78	79
				N	115	115	119
58016*r*	102	280	280				
R	289	270	395	84007*M*	1	53	53
N	304	304	1863	*r*	40	61	177
				R	50	61	188
58030*r*	121	350	350	*N*	80	80	207
R	349	344	611				
N	366	366	633	84133*C*	0	583	583
				R	53	358	80
58032*r*	168	363	363	*N*	516	516	689
R	247	356	388				
N	392	392	424	89034*M*	9	250	250
				R	72	242	524
58033*r*	209	637	637	*N*	319	319	645
R	712	611	698				
N	682	682	1863	91014*M*	1	275	275
				R	455	273	474
58089*r*	129	386	386	*N*	410	410	611
R	370	373	538				
N	388	388	553	97060*M*	5	365	365
				R	300	424	584
58091r	129	371	371	*N*	560	560	1084
R	370	359	526				
N	388	388	555				

[a] *M* stands for Maintenance, *R* or *r* for Rehabilitation, *N* for New bridge, and *C* for Closure

and benefits of every other alternative for the same bridge. These marginal input data are used to allocate the balance of the budget granted; the balance is determined by subtracting the sum of all the least-cost alternatives' initial costs from the budget granted. This marginal budget allocation ensures that if INCBEN selects no alternative for a bridge, its least-cost alternative is automatically recommended for funding.

INCBEN is used to analyze the data for two types of benefits expected from the improvement alternatives: the first analysis considers both agency and user benefits, and the second analysis considers agency benefits only. Alternatives are ranked for several budget levels.

INCBEN Results Considering Agency and User Benefits

Results of the INCBEN analysis are presented in Tables 3 and 4 and in Figure 1. Up to $8,036,000, the higher the budget granted, the higher the net benefits expected. However, for granted budgets of more than $8,036,000, the budget allocated and benefits expected remain constant. The net benefits are expected to be at their highest level of $39,493,000. Thus, $8,036,000 is the optimum budget justified under no budget constraints for improving the 25 bridges.

Figure 1 and Table 3 demonstrate that as the granted budget is increased from $2,120,000 to $2,142,000, sharp increases in total benefits and net benefits are expected. A $2,120,000 budget is just enough to fund all least-cost alternatives for the 25 bridges. These alternatives include 45009*R* which is not cost-effective at all, as indicated in Table 1. If the granted budget is increased by a small $22,000, Table 4 indicates that the maintenance alternative is selected for Bridge 45009 that is significantly more cost-effective than 45009*R*, as shown in Tables 1 and 3 and in Figure 1.

Table 4 verifies that only replacing Bridges 05125, 10381, 29058, 58016, 58033, 75171, and 97060 is cost-effective even under unlimited funding. The other 17 bridges should not be replaced, even under no budget constraints. For granted budgets of $1,000,000 or less, maintenance is the preferred alternative (52 percent), but 28 percent of the bridges will receive no improvement because of insufficient funding. For budgets of $8,036,000 or more, 32 percent of the bridges would be replaced, 44 percent rehabilitated, and 20 percent maintained; 4 percent would remain closed.

Table 4 also shows that Bridges 08052, 17001, 80173, and 91014 should always be maintained as long as the budget granted is at least $250,000. Bridge 84133 must remain closed regardless of the granted budget; the other 24 bridges receive some improvement and granted budgets of at least $2,120,000.

INCBEN Results Considering Agency Benefits Only

Table 3 and Figure 2 indicate that budgets allocated and benefits expected remain constant for granted budgets over $2,227,000. At such levels, net benefits expected are at their highest level of $13,126,000. Thus, $2,227,000 is the optimum justifiable budget under no budget constraints, if user costs are excluded. Figure 2 shows that up to $2,227,000, the higher the budget granted, the higher the net benefits expected. Table 5 indicates that replacement is never cost-effective for these 25 bridges if user benefits are excluded. Maintenance is the alternative selected most when only agency benefits are considered.

Figure 2 and Table 3 also show that total and net benefits expected gradually increase with increasing budgets up to $2,158,000. As the budget granted is further increased to $2,189,000, sharp increases are expected in net benefits and total benefits because of changes in the alternatives selected, as indicated in Table 5. For a $2,158,000 budget, the alternatives selected for Bridges 05125 and 73411 are *R* and *M*, respectively. For a $2,189,000 budget, these selections change to 05125*M* and 73411*R*. As a result, net benefits expected increase by $1,524,000, as shown in Figure 2 and in Tables 1 and 3.

Table 5 shows that Bridges 08052, 17001, 29058, 61010, 75171, 80173, 84007, 89034, 91014, and 97060 should always receive maintenance as long as the budget granted is $250,000 or more. Again, Bridge 84133 must remain closed regardless of the budget granted. For a $2,120,000 budget or more, all the other bridges receive some improvement: 48 percent of the bridges would be maintained, 48 percent would be rehabilitated, and 4 percent would remain closed. For budgets less than $1,000,000, maintenance is the alternative selected most.

SENSITIVITY ANALYSIS OF BRIDGE IMPROVEMENT DECISIONS

The accuracy of the INCBEN results is, of course, a function of the accuracy of the input data. These data are best described as

TABLE 2 Incremental Benefit-Cost Ratios for Bridge 05125, Agency Costs Only

Alternative (1)	Net Benefit (2)	First Cost (3)	Total Benefit (4)	ΔB (5)	ΔC (6)	$\Delta B/\Delta C$ (7)
	Thousands of Dollars					
Maintenance	90	2	92	NA[a]	NA[a]	NA[a]
Rehabilitation	170	40	210	118	38	3.11
New Bridge	0	283	283	73	243	<1

[a] Not Applicable

TABLE 3 INCBEN Results at 6 Percent Discount Rate ($ thousands)

Budget Granted (1)	Budget Allocated (2)	Total Benefits (3)	Net Benefits (4)	Excess Budget (5)
250	249	8,651	8,402	1
500	466	9,633	9,167	34
1,000	972	11,567	10,595	28
2,120	2,120	14,437	12,317	0
2,142	2,142	29,254	27,112	0
2,180	2,180	29,676	27,496	0
2,249	2,249	30,170	27,921	0
3,000	2,991	34,787	31,796	9
4,000	3,903	37,524	33,621	97
5,000	4,979	41,926	36,947	21
6,000	5,946	44,802	38,856	54
7,000	6,922	46,244	39,322	78
8,036	8,036	47,529	39,493	0

TABLE 4 Alternatives Selected for Several Budget Levels, Agency and User Benefits

	Budget Granted ($1,000)									
Bridge No. (1)	250 (2)	500 (3)	1000 (4)	2120 (5)	2142 (6)	2180 (7)	2249 (8)	3000 (9)	5000 (10)	≥8036 (11)
00210	-	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*R*
05125	-	-	-	*M*	*M*	*R*	*R*	*N*	*R*	*N*
08052	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
10381	-	-	*M*	*M*	*M*	*M*	*M*	*M*	*N*	*N*
17001	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
29058	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*N*
45009	*R*	*R*	*R*	*R*	*M*	*M*	*M*	*M*	*M*	*M*
58016	-	-	-	*r*	*r*	*r*	*r*	*N*	*N*	*N*
58030	*r*	-	*r*	*r*	*r*	*r*	*r*	*r*	*r*	*N*
58032	-	-	-	*r*	*r*	*r*	*r*	*r*	*r*	*r*
58033	-	-	-	*r*	*r*	*r*	*r*	*r*	*r*	*N*
58089	-	-	*r*	*r*	*r*	*r*	*r*	*r*	*r*	*r*
58091	-	-	-	*r*	*r*	*r*	*r*	*r*	*r*	*r*
58100	-	-	-	*r*	*r*	*r*	*r*	*r*	*r*	*r*
58102	-	*r*	-	*r*	*r*	*r*	*r*	*r*	*r*	*r*
58128	-	-	-	*r*	*r*	*r*	*r*	*r*	*r*	*R*
61010	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*R*	*R*	*R*
73411	*M*	*M*	*M*	*M*	*M*	*M*	*R*	*R*	*R*	*R*
75171	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*R*	*R*	*N*
80173	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
84007	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*r*	*R*	*R*
84133	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*
89034	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*R*	*R*	*R*
91014	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
97060	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*N*

M stands for Maintenance, *R* or *r* for Rehabilitation, *N* for New bridge (replacement), and *C* for Closure

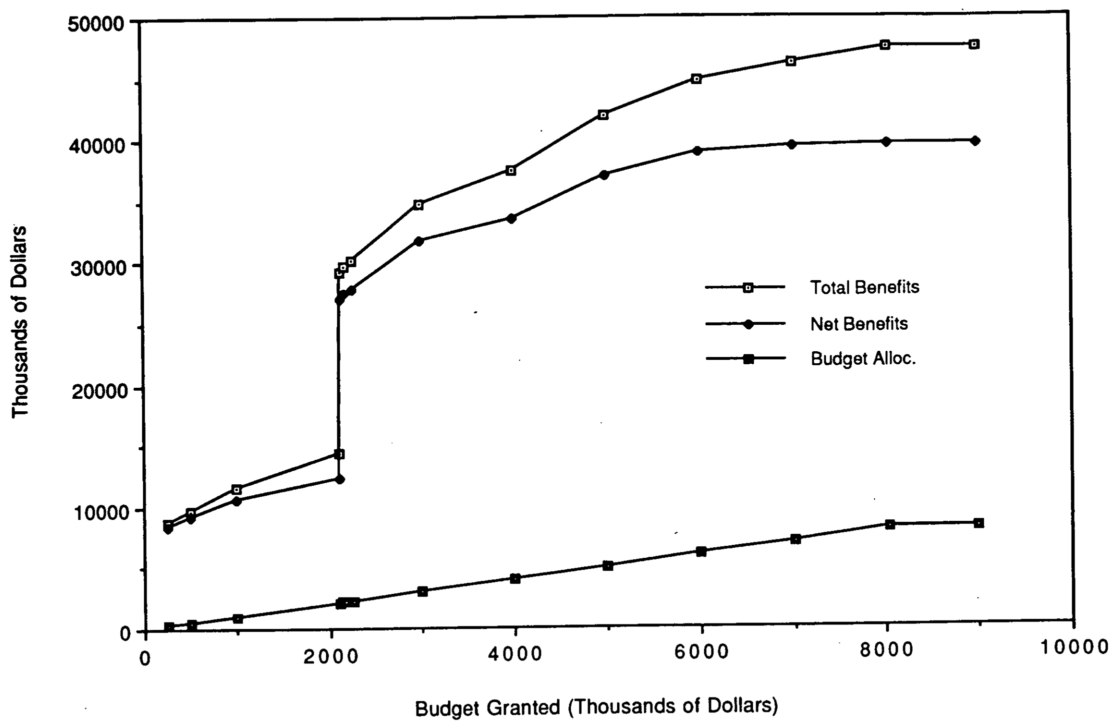

FIGURE 1 **INCBEN results, agency and user benefits at 6 percent discount rate.**

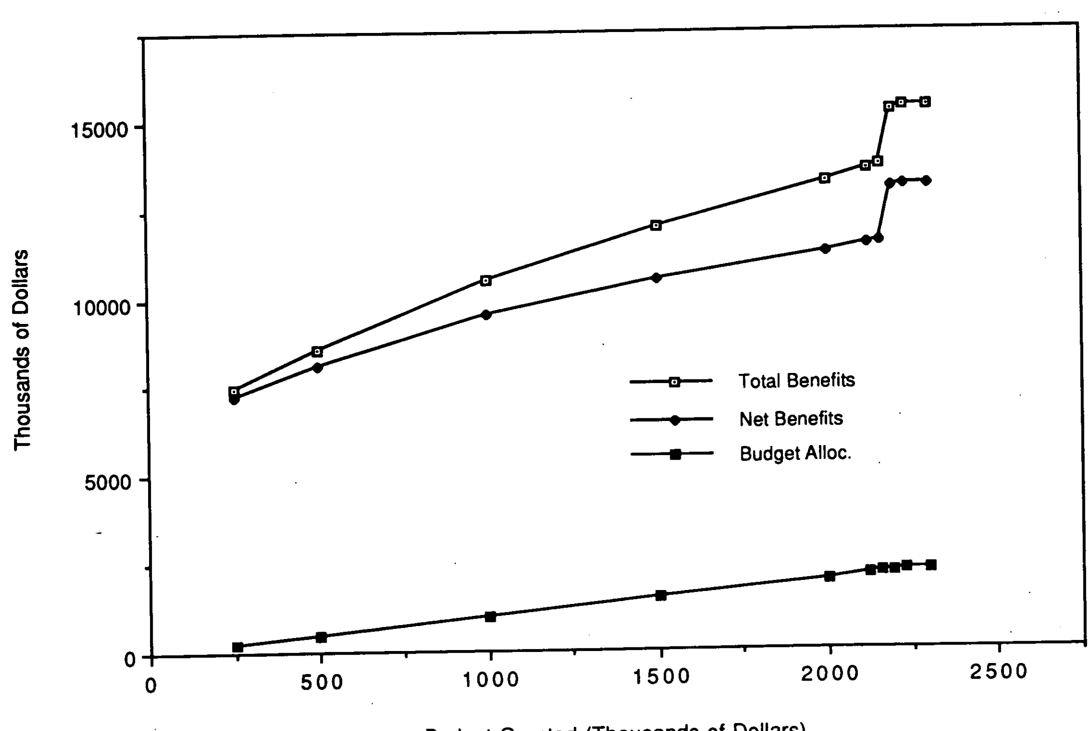

FIGURE 2 **INCBEN results, agency benefits only at 6 percent discount rate.**

random variables because they are only forecasts of future events. Thus, impacts of varying major input data on the INCBEN results should be of interest. Such a "sensitivity analysis" is a "study to see how the economic decision will be altered if certain factors are varied" (*7*). Or, "sensitivity refers to the relative magnitude of the change in one or more elements of an engineering economy problem that will reverse a decision among alternatives" (*8*). To perform a sensitivity analysis, variables that are most likely to affect the results and their probable ranges are determined first. A variable's expected value is often selected as the base. Results obtained by using other values of the variable are compared with those obtained by using the base value. Under a limited budget, sensitivity of the ranking of the bridge improvement alternatives and their expected net benefits to the discount rate, remaining life, and service life are analyzed.

Sensitivity to Discount Rate

The discount rate is used to compute the present value of the future cash-flow stream representing costs and benefits. Selection of an appropriate discount rate is an important step in any discounted cash flow analysis because it can easily affect the results (*9*). A 6 percent discount rate is used as the "base," the recommended are for long-term public projects. Outcomes obtained by using discount rates of 4, 5, 7, and 8 percent are compared with those obtained at the 6 percent base rate.

Figure 3 depicts the sensitivity of net benefits to the discount rate. If both agency and user benefits are considered, the higher the discount rate, the lower the net benefits expected. This is because the present value of user benefits, which usually lag behind costs, decreases more than the present value of costs as the discount rate increases. The results appear consistent with Miller et al.'s position that low discount rates favor projects with high capital costs (*10*). While net benefits vary slightly with the discount rate, the improvement alternatives selected remain essentially unaffected (*4*). Thus, the INCBEN results are not sensitive to discount rate when both agency and user benefits are considered.

Figure 3 also depicts the sensitivity of net benefits to discount rate considering agency benefits only. Net benefits vary slightly: the higher the discount rate, the higher the net benefits expected. Again, the improvement alternatives selected are not sensitive to the discount rate for a $2,200,000 budget. No new-bridge alternative is selected for any of the 25 bridges at any discount rate (*4*). These results signal that replacement is rarely economical if user benefits are excluded.

TABLE 5 Alternatives Selected for Several Budget Levels, Agency Benefits

Bridge No.	Budget Granted ($1,000)							
	250	500	1000	2000	2120	2158	2189	≥2227
00210	-	*M*	*M*	*M*	*M*	*M*	*M*	*M*
05125	*M*	*M*	*M*	*M*	*M*	*R*	*M*	*R*
08052	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
10381	-	-	*M*	*M*	*M*	*M*	*M*	*M*
17001	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
29058	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
45009	*R*	*R*	*R*	*R*	*R*	*R*	*R*	*R*
58016	*r*	-	-	*r*	*r*	*r*	*r*	*r*
58030	-	-	*r*	*r*	*r*	*r*	*r*	*r*
58032	-	-	-	-	*r*	*r*	*r*	*r*
58033	-	-	-	*r*	*r*	*r*	*r*	*r*
58089	-	-	*r*	*r*	*r*	*r*	*r*	*r*
58091	-	-	-	*r*	*r*	*r*	*r*	*r*
58100	-	-	-	*r*	*r*	*r*	*r*	*r*
58102	-	*r*	-	*r*	*r*	*r*	*r*	*r*
58128	-	-	-	*r*	*r*	*r*	*r*	*r*
61010	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
73411	*M*	*M*	*M*	*M*	*M*	*M*	*R*	*R*
75171	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
80173	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
84007	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
84133	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*
89034	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
91014	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*
97060	*M*	*M*	*M*	*M*	*M*	*M*	*M*	*M*

M stands for Maintenance, *R* or *r* for Rehabilitation, *N* for New bridge (replacement), and *C* for Closure

Sensitivity to Remaining Life

A bridge's remaining life is a function of the deterioration rate of its structural elements. Some prediction data are available, but the actual remaining lives can be highly variable. Thus, variations in remaining life can affect the INCBEN results.

Sensitivity analyses are preformed covering a ±30 percent range of the expected remaining lives of the bridges. As depicted in Figure 4, the higher the remaining life, the higher the net benefits expected if both agency and user benefits are considered. But results are more sensitive to the shorter remaining lives. Improvement alternatives selected for the 30 percent shorter remaining lives are different from selections for the other two cases (*4*).

Considering agency benefits only, Figure 4 confirms that the expected benefits are slightly sensitive to the remaining life. Improvement alternatives selected are slightly sensitive to the longer remaining life. For a 30 percent longer remaining life, the maintenance alternative for Bridge 73411 and the rehabilitation alternative for Bridge 05125 are the only changes in the alternatives selected for the other two cases (*4*).

Sensitivity to Service Life

A bridge's service life is the number of years that it can serve the traffic before it becomes structurally unsafe (*6*). Therefore, estimating the service life of a bridge is a function of how its structural conditions will deteriorate because of factors such as the weather and traffic conditions. Statistical techniques can be used to estimate deterioration formulas from which the service life can be approximated. However, estimated the extended service life after rehabilitation or maintenance "is not exact and requires engineering judgment" (*6*). Service life may be reduced by significant increases in the level-of-service needs.

Sensitivity analyses of the INCBEN results to service life considers ±30 percent variation in the 50-year expected service life of the bridges. Figure 5 confirms that longer service lives result in slightly higher net benefits, when both agency and user benefits are considered. Bridge improvement decisions are somewhat sensitive to the service life. Improvement alternatives selected for Bridges 58128 and 61010 are the same for ±30 percent service life cases, but they are different from the expected service life case. Bridge 84007 should also receive a different improvement for the shorter service life case compared with the other cases (*4*).

Figure 5 also confirms that the expected net benefits are not very sensitive to the service life if only agency benefits are considered. Improvement alternatives selected are not sensitive to the service life, either. As a result, the total initial costs of the alternatives selected remain constant for the three cases (*4*).

Analysis of Sensitivity Results

Results of the three sensitivity analyses indicate that variances of net benefits from the base cases are generally less than ±10 percent. One exception is for the remaining life when considering both agency and user benefits, where the variance of net benefits ranges from −22.5 to +16.5 percent (*4*). Since estimates of agency and user costs and benefits are generally no more reliable, these variations are not considered significant.

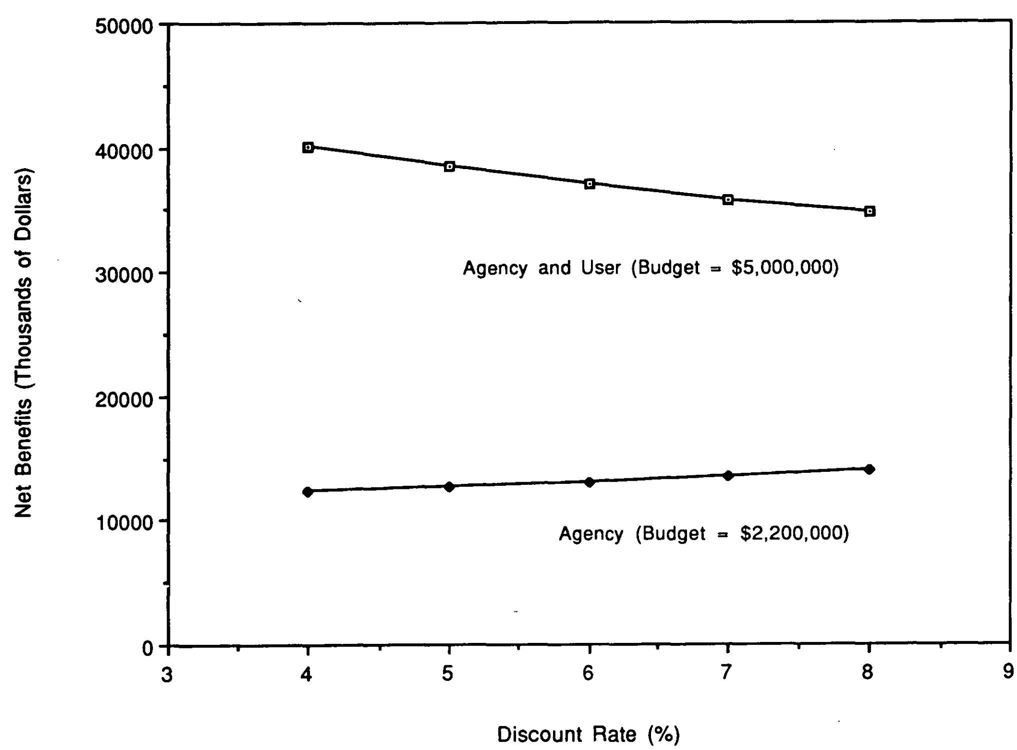

FIGURE 3 Sensitivity of net benefits to discount rate.

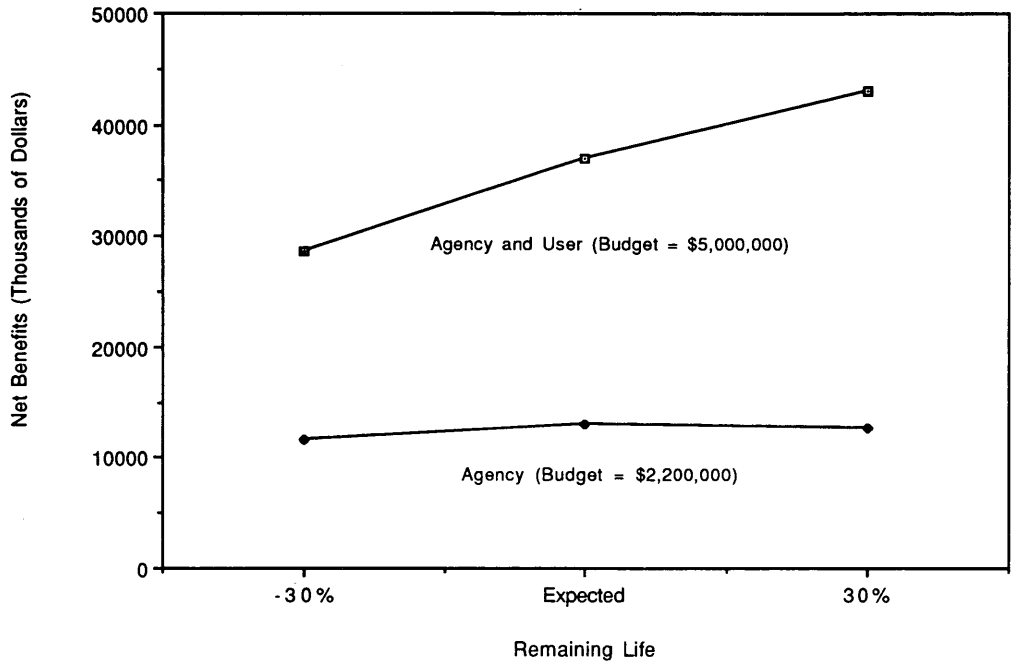

FIGURE 4 **Sensitivity of net benefits to remaining life.**

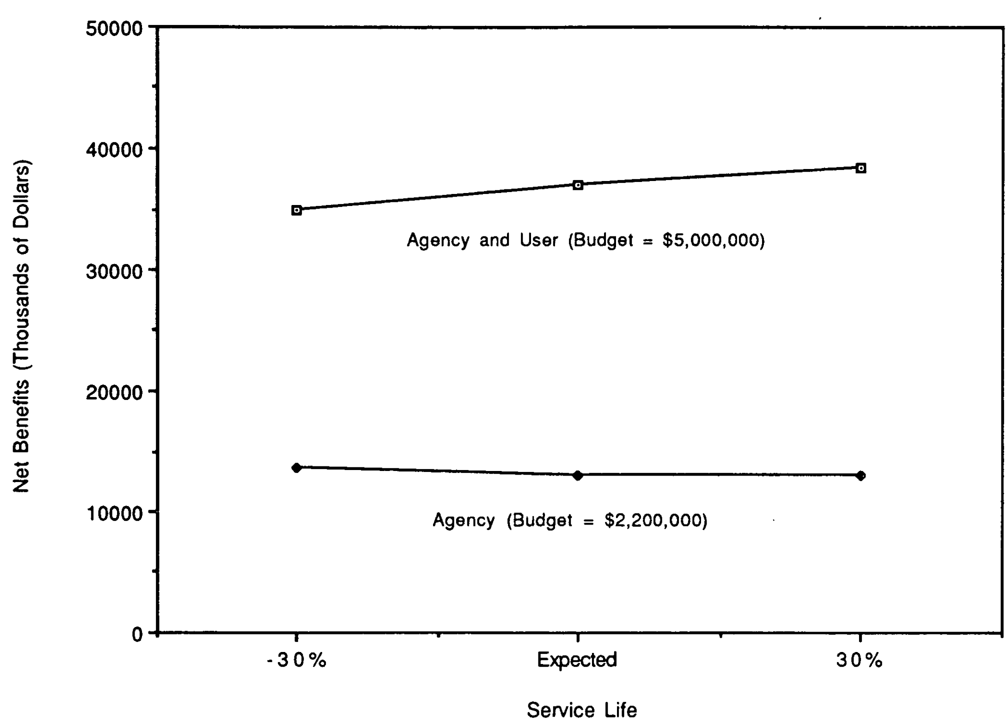

FIGURE 5 **Sensitivity of net benefits to service life.**

Selections of improvement alternatives change only slightly from one end of the probable ranges of the variable to the other. Improvement alternatives always change for fewer than 20 percent, usually for fewer than 10 percent, of the bridges. Alternative changes usually result in small additional first costs (*M* to *R*, *r* to *R*, or *R* to *N*). Large alternative shifts (*M* to *N*) are not encountered (*4*).

Sensitivity of the INCBEN results is somewhat greater when both agency and user benefits are considered as opposed to when only agency benefits are considered. Overall, while improvement alternatives selected and their expected net benefits vary somewhat, the results are relatively insensitive to the discount rate, remaining life, and service life, within reasonable ranges of these variables.

COMPARISON OF INCBEN AND SUFFICIENCY RATING METHODS

Many states have used the sufficiency rating to priority rank bridges for improvement (*4*). The budget granted is sometimes allocated to bridges in ascending order of their sufficiency ratings—that is, a bridge having a lower sufficiency rating receives a higher priority for improvement. Table 6 gives the priority rankings of the 25 sample bridges by sufficiency rating.

After the priority rankings are formulated, a specific improvement alternative should be selected from all the possible alternatives for every bridge. Methods available for selecting improvement alternatives at the bridge level produce varying results. Table 7 gives the results of budget allocations by five sufficiency rating methods that may be used to select improvement alternatives for bridges in the order of their priority rankings. Budget allocations produced by INCBEN are also listed.

To have a compatible comparison with the INCBEN analysis, sufficiency rating methods also assume that an improvement alternative on every bridge is mandatory. Thus, all sufficiency rating methods first fund the least-cost alternatives in the order of the priority rankings. If all the least-cost alternatives are funded, the budget balance is then allocated in the priority-ranking order according to the specific criteria of various methods:

1. "Economic Analysis," shown in Column 4 of Table 7, funds improvement alternatives on the basis of an economic analysis at the bridge level.
2. "All Replacement," shown in Column 5 of Table 7, funds the replacement alternative for each bridge.
3. "<50 Replacement/<80 Rehabilitation," shown in Column 6 of Table 7, funds replacement if the sufficiency rating is lower than 50 or funds rehabilitation if the sufficiency rating is between 50 and 80.
4. "Rehabilitation if $ <50%," shown in Column 7 of Table 7, funds replacement unless the initial rehabilitation cost is less than 50 percent of the replacement cost.
5. "Worst Case," shown in Column 8 of Table 7, funds the least economic alternatives.

None of these five sufficiency-rating methods is advocated; they are presented for comparison only. Budget allocations based on the economic analysis and worst-case criteria theoretically form the two extremes of the budget allocations using priority rankings based on sufficiency ratings.

Table 7 compares budget allocations by INCBEN and five sufficiency rating methods at various levels of budget granted. Based on these results, the incremental benefit-cost analysis consistently produces selections for all budget levels analyzed that are equal to or better than other methods using priority rankings based on sufficiency ratings. At a $2,120,000 budget level, which is the minimum budget required to fund the least-cost alternatives for all bridges, all five sufficiency rating methods produce results identical to those produced by INCBEN. The percentages shown inside the parenthesis in Table 7 indicate variances from net benefits expected from INCBEN selections. A negative sign indicates lower net benefits than those produced by the incremental benefit-cost analysis.

TABLE 6 Priority Rankings of Sample Bridges in Ascending Order of Sufficiency Ratings

Priority Ranking (1)	Sufficiency Rating (2)	Bridge Number (3)	Priority Ranking (4)	Sufficiency Rating (5)	Bridge Number (6)
1[a]	0.0	05125	14	61.4	80173
1[a]	0.0	84133	15	66.8	58091
3	1.0	17001	16	67.0	58089
4	5.0	73411	17	67.8	58030
5	29.2	45009	18	70.8	89034
6	30.2	00210	19	70.9	91014
7	37.1	10381	20	71.9	58128
8	37.3	08052	21	73.6	84007
9	46.1	58100	22	74.8	61010
10	49.8	75171	23	75.1	58102
11	56.1	29058	24	76.0	58016
12	56.6	97060	25	78.4	58032
13	56.9	58033			

[a] A 2-way tie

TABLE 7 Comparison of Budget Allocations by INCBEN and Sufficiency Rating Methods

Granted Budget	Expected Value of	INCBEN[b]	Sufficiency-Rating Method[a]: Economic Analysis	All Replace.	<50 Rep. <80 Reh.	Reh. If $ < 50%	Worst Case
All Figures in Thousands of Dollars Except Percentages							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
250	Total Benefits	8,651	7,673	7,673	7,673	7,673	7,673
	Budget Allocated	249	249	249	249	249	249
	Net Benefits	8,402	7,424	7,424	7,424	7,424	7,424
			(-11.6%)[c]	(-11.6%)	(-11.6%)	(-11.6%)	(-11.6%)
2,120	Total Benefits	14,437	14,437	14,437	14,437	14,437	14,437
	Budget Allocated	2,120	2,120	2,120	2,120	2,120	2,120
	Net Benefits	12,317	12,317	12,317	12,317	12,317	12,317
			(0.0%)	(0.0%)	(0.0%)	(0.0%)	(0.0%)
2,142	Total Benefits	29,254	29,254	14,437	14,436	29,254	14,437
	Budget Allocated	2,142	2,142	2,120	2,123	2,142	2,120
	Net Benefits	27,112	27,112	12,317	12,313	27,112	12,317
			(0.0%)	(-54.6%)	(-54.6%)	(0.0%)	(-54.6%)
5,000	Total Benefits	41,926	37,413	17,287	17,286	33,943	8,060
	Budget Allocated	4,979	4,959	4,977	4,980	4,993	4,878
	Net Benefits	36,947	32,454	12,310	12,306	28,950	3,182
			(-12.2%)	(-66.7%)	(-66.7%)	(-21.6%)	(-91.4%)
8,036	Total Benefits	47,529	47,529	19,125	19,328	42,962	11,273
	Budget Allocated	8,036	8,036	8,023	7,991	8,009	7,945
	Net Benefits	39,493	39,493	11,102	11,337	34,953	3,328
			(0.0%)	(-71.8%)	(-71.3%)	(-11.5%)	(-91.6%)
11,836	Total Benefits	47,529	47,529	36,540	36,501	47,923	13,504
	Budget Allocated	8,036	8,036	11,772	11,823	11,604	11,836
	Net Benefits	39,493	39,493	24,768	24,678	36,319	1,668
			(0.0%)	(-37.3%)	(-37.5%)	(-8.0%)	(-95.8%)
17,698	Total Benefits	47,529	47,529	51,596	50,084	47,923	13,504
	Budget Allocated	8,036	8,036	17,570	17,698	11,604	11,836
	Net Benefits	39,493	39,493	34,026	32,386	36,319	1,668
			(0.0%)	(-13.8%)	(-18.0%)	(-8.0%)	(-95.8%)
≥20237	Total Benefits	47,529	47,529	55,168	51,460	47,923	13,504
	Budget Allocated	8,036	8,036	20,237	19,272	11,604	11,836
	Net Beneftis	39,493	39,493	34,931	32,188	36,319	1,668
			(0.0%)	(-11.6%)	(-18.5%)	(-8.0%)	(-95.8%)
Cum. Net Benefits at all Budget Levels		242,750	237,279 (-2.3%)[c]	149,195 (-38.5%)	144,949 (-40.3%)	219,713 (-9.5%)	43,572 (-82.1%)
Cum. Net Benefits at Budget Levels ≤ $8,036		124,271	118,800 (-4.4%)[c]	55,470 (-55.4%)	55,697 (-55.2%)	110,756 (-10.9%)	38,568 (-69.0%)
Performance Ranking		1	2	4	5	3	6

[a] All methods first fund the least-cost alternatives in the order of priority rankings. If all least-cost alternatives are funded, the balance of the granted budget is then allocated according to their specific criteria and in priority-ranking order. Specific allocation criteria are defined in the text.

[b] INCBEN first considers only the least-cost alternative for every bridge. After all least-cost alternatives are funded, the granted budget balance is then allocated to other alternatives using their *marginal* benefits and costs over their corresponding least-cost alternatives.

[c] Net-benefits percentage variance from net benefits expected from the INCBEN selections.

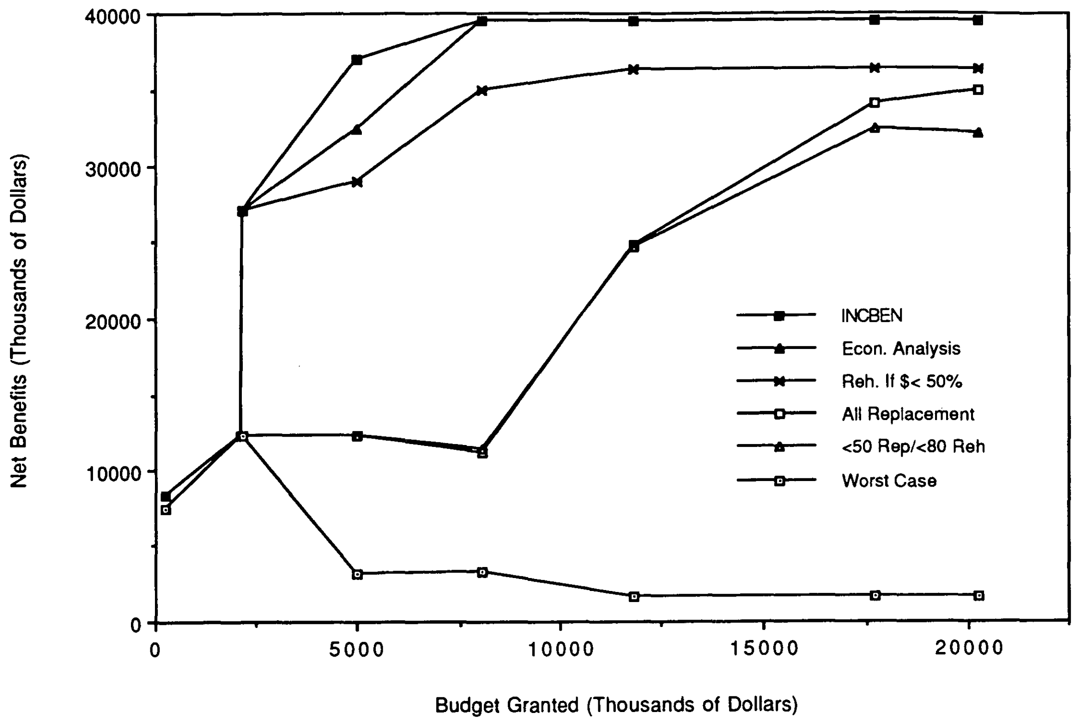

FIGURE 6 Comparison of budget allocations by INCBEN and sufficiency rating methods.

The results in Table 7 are also depicted in Figure 6. The economic analysis method at the bridge level produces the best results among all sufficiency rating methods. This is because it selects the best improvement alternative at the bridge level on the basis of the same economic principles used by the incremental benefit-cost analysis. Of course, the INCBEN analysis at the system level produces superior results under budget constrains. For example, at a $5,000,000 budget granted, net benefits expected from the economic analysis allocation are nearly 12 percent lower than those expected from the INCBEN allocation. For granted budgets of $8,036,000 or more, INCBEN and economic analysis produce the same results because the most cost-effective improvement alternative for every bridge can be funded at these levels. Thus, economic analysis at the bridge level produces results identical to those produced by the INCBEN analysis at the system level under unlimited budgets.

The all-replacement method produces net benefits that are as much as 72 percent lower than those produced by INCBEN. The <50-replacement/<80-rehabilitation method produces results that are up to 70 percent inferior to those produced by the INCBEN analysis. The Rehabilitation-if-$<50% method produces results that are up to 22 percent worse than the INCBEN analysis. The worst-case method, of course, produces the lowest net benefits at every budget level.

The cumulative net benefits expected from each method at all budget levels are also presented in Table 7. These cumulative net benefits provide an approximate measure of the overall performance of various methods. The percentages shown inside the parentheses here indicate the variance of the cumulative net benefits expected from the corresponding method from those expected from the INCBEN analysis. From these cumulative data, the INCBEN analysis produces better results than the five sufficiency rating methods evaluated. The sufficiency rating methods are expected to produce cumulative net benefits that are 2 to 82 percent lower than those expected from the INCBEN analysis, as indicated in Table 7.

The data presented in Table 7 and Figure 6 can also be used for preparing and justifying budget requests. Figure 6 clearly indicates that budget requests of more than $8,036,000 are not economical. The cumulative net benefits expected from every method at budget levels of $8,036,000 or lower are given in Table 7, immediately below the same data at all budget levels. These data confirm that INCBEN selections produce significantly higher cumulative net benefits than any of the sufficiency rating methods under more realistic levels of budget granted.

Figures 6 and 1 confirm that budget requests lower than $2,142,000 are not prudent. At such low budget levels, many of the least-cost improvement alternatives must be selected at the expense of the more cost-effective alternatives.

CONCLUSIONS

The INCBEN program can be used for optimal allocation of limited budgets to maintenance, rehabilitation and replacement of bridges. Major conclusions of the INCBEN application to a sample of 25 bridges in North Carolina include the following:

1. INCBEN generates priority rankings of the improvement alternatives in the decreasing order of their incremental benefit-cost ratios. These rankings are superior to those generated by the sufficiency rating methods. INCBEN rankings are superior not only

because they are based on sound economic principles but also because INCBEN selects specific improvement alternatives for deficient bridges.

2. INCBEN recommends near-optimal sets of bridge improvement alternatives under limited budgets. INCBEN selections under unlimited budgets are optimal and identical to the alternatives selected by the economic analysis at the bridge level.

3. Results of the budget allocations by INCBEN are only slightly sensitive to the discount rate, remaining life, and service life of a bridge. Variations in net benefits expected are small. Changes in improvement alternatives selected, or in their priority rankings, are minimal.

4. The replacement alternative is never cost-effective for any of the 25 bridges if user costs are excluded. Thus, both agency and user costs must be considered in any realistic bridge management system.

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DISCUSSION

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The paper illustrates a good example of life-cycle cost analysis of user costs and benefits in bridge management. However, the authors provide very little description of the INCBEN software methodology, especially the benefits and user costs. A significant amount of life-cycle user costs are related to the vehicle operating costs, which increase when the bridge deck condition deteriorates. This is a rational and quantified approach because this user cost component is directly a function of vehicular traffic and operating speed (*1*). Therefore, vehicle operating costs also reflect the user costs due to a decrease in the level of service.

Finally, it is recommended that user costs and benefits should be included for objective evaluation of competing maintenance, rehabilitation, and reconstruction/replacement alternative strategies in all areas of management systems identified in the Intermodal Surface Transportation Efficiency Act of 1991.

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AUTHORS' CLOSURE

The discussant's interest in this paper, and in highway management systems in general, is appreciated. INCBEN and the required input data are described in the section of the paper entitled "INCBEN Program." This section further states: "Farid et al. have provided a detailed description of INCBEN (*4*; another paper in this Record), and a complete description of INCBEN has also been presented by McFarland et al. (*3*)." Since this and the companion paper by Farid et al. will appear in the same Record, readers can easily refer to the companion paper for additional information on INCBEN. Both papers were presented in Session 115, and their preprints were available side by side, at the 73rd Annual Meeting of the Transportation Research Board. Thus, it is unclear why the companion paper has been overlooked.

User costs and benefits are covered in the subsection entitled "Forecasting Input Data." Again, this section states: "Farid et al. (*4*; another paper in this Record) describe the techniques used for developing the improvement alternatives and their life-cycle costs." Increases in operating costs of vehicles traveling over long bridges with deteriorated decks may prove significant enough to be included in user costs. But, the USER microcomputer program (*1*) cannot be used for estimating bridge user costs because bridge decks deteriorate differently from highway pavements. Further, this research was conducted and its final report published long before the discussant's paper (*1*) was published. Only references actually used are cited in the references. Many other publications have made significant contributions to infrastructure management systems. Space limitations preclude publication of bibliographies in technical papers such as those published by TRB. Interestingly enough, Uddin and George (*1*) did not reference Farid et al. (*2*) or Chen and Johnston (*3*), even though these reports have been widely distributed by FHWA and frequently cited in publications on bridge management systems.

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Bridge Deterioration Models for States with Small Bridge Inventories

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In a bridge management system the estimation of bridge performance is a key tool for devising the optimal strategies for the maintenance, rehabilitation, and replacement of bridges. Therefore it is essential that deterioration models that accurately estimate the remaining performance of bridges be developed. This is a difficult task for states that have a limited number of bridge structures. Deterioration models and modifications to data bases that produced reasonable projections for bridge deterioration are described. Four different levels of modeling are described. The models are different because of the number of parameters included. Therefore the amount of data used to develop each model also changes. In the project 277 data sets were obtained from the four levels of modeling. The models were nonlinear with exponential decay functions and spikes for rehabilitation. The equations incorporate the following parameters: average daily traffic, bridge age, time of rehabilitation, environmental factors, type of structure, and bridge component. Examples of the deterioration models developed are given, as are two examples showing the application of the models. The results of the models indicated some basic trends in bridge deterioration and also pointed to the need for the regionalization of deterioration models. The regionalization of deterioration models would permit states with similar bridges and environmental conditions to combine their data bases. This would provide a larger data base from which to produce more accurate bridge deterioration models.

All states are required to have initiated a bridge management system by October 1, 1994. To do so they must be able to predict the deterioration of their bridges so that they can plan for their maintenance, rehabilitation, and reconstruction. This requires an analysis of the state's past inspection records and an inventory of the existing bridges to establish patterns, types, and rates of deterioration under the specific conditions in that state.

Several individual states are developing their own bridge management systems, whereas others are examining PONTIS or are waiting for the results of NCHRP-12-28. One common thread in all of the work that has been done or that is under way is that it is geared to be used on an individual-state basis. Therefore they are geared toward states that have large bridge inventories. The state of Nevada is 285,000 km^2 (110,000 mi^2), yet it has only 887 bridges [structures over 6 m (20-ft) long excluding culverts]. Of these 887 bridges, 587 are reinforced concrete, 159 are prestressed concrete, 137 are steel, and 4 are timber bridges. Nevada has fewer bridges than many other states, and the bridges in its inventory are very young. Most of the bridges in Nevada have been built since the early 1960s. This is a result of the construction of the Interstate highway system and the rapid growth in Nevada since the early 1960s. Inspection data that are consistent with current inspection data are available back to 1979. Therefore there is not a substantial amount of data for establishing deterioration models.

This paper describes the challenges in establishing the deterioration models for small bridge inventories. The model variables that were included and the modifications done to the data base to develop reasonable results are also discussed (*1*). One conclusion from the paper is that even though effective models for small bridge inventories can be developed, the regionalization of deterioration models would be an effective means of improving deterioration predictions.

FACTORS AFFECTING BRIDGE DETERIORATION AND EXISTING MODELS

The deterioration models developed in the past included linear (*2*), piecewise linear (*3*), linear regression coupled with a spike reflecting a single rehabilitation (*4*), nonlinear with exponential decay functions coupled with spikes to reflect the effects of rehabilitations (*5*), and methods that incorporate the Markov chain (*6*). For an initial examination of deterioration trends and an investigation of the data base, it was decided that nonlinear deterioration models that included spikes for rehabilitations would be used. The linear curves do not provide sufficient accuracy, and the effort to develop the Markov chain was not justified until an initial examination and a model of the data base were completed.

Several factors influence deterioration. Those that were considered in the model included the bridge's age, structural component, structure type, rehabilitation history, average daily traffic (ADT), and environmental conditions. Bridge maintenance was not included in the models. A specific study would need to be undertaken to determine the impacts of different maintenance procedures. The bridge's age is the most pertinent factor for establishing deterioration. Modeling of the structural component can be done in two ways: (a) generally (substructure, superstructure, and deck) and (b) specifically (stringers, bearings, parapets, expansion joints, abutment wings, abutment backwalls, etc.). The more general approach was selected for the Nevada models. The bridges were divided into three categories: reinforced concrete, steel, and prestressed concrete. These general categories include 99 percent of the bridges in Nevada.

Rehabilitation plays a major role in bridge deterioration. An early minor rehabilitation has less of an effect on a bridge than a later major rehabilitation. The data base does not retain all of the rehabilitation dates for a bridge nor a record regarding the nature of the rehabilitation. The work that has been done can be found by examining the original contract documents. This requires significant effort even for only 900 bridges. Therefore if a bridge

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rehabilitation was recorded in the data base it was assumed that all of the major components of a bridge were rehabilitated. The Nevada Department of Transportation's definition of a rehabilitation is an event that corrects most or all of the structural deficiencies. Major maintenance (e.g., resurfacing of a bridge deck) is not considered a rehabilitation.

Environmental factors also play a significant role. Exposure of bridges to certain environmental conditions, both natural and artificial, can drastically shorten a bridge's service life. It is very difficult to include all of the environmental effects directly in a model. Therefore environmental factors are included indirectly in the model through geographical location. According to the National Oceanic and Atmospheric Administration, Nevada is divided into four regions (Figure 1). Upon further review the northwestern and northeastern regions are very similar both in climate and geography. The extreme southern region is warmer and on average has significantly less precipitation than the northern regions. The south-central region combines features of the northern and southern regions and has only a few bridges. Therefore the state was divided into two geographical regions for modeling the environmental effects. Bridges above 38 degrees north latitude were considered to be in the northern region; those below that latitude were considered to be in the southern region.

DETERIORATION MODEL DEVELOPMENT

Once the variables affecting bridge deterioration were identified, the researchers could then incorporate them into a deterioration

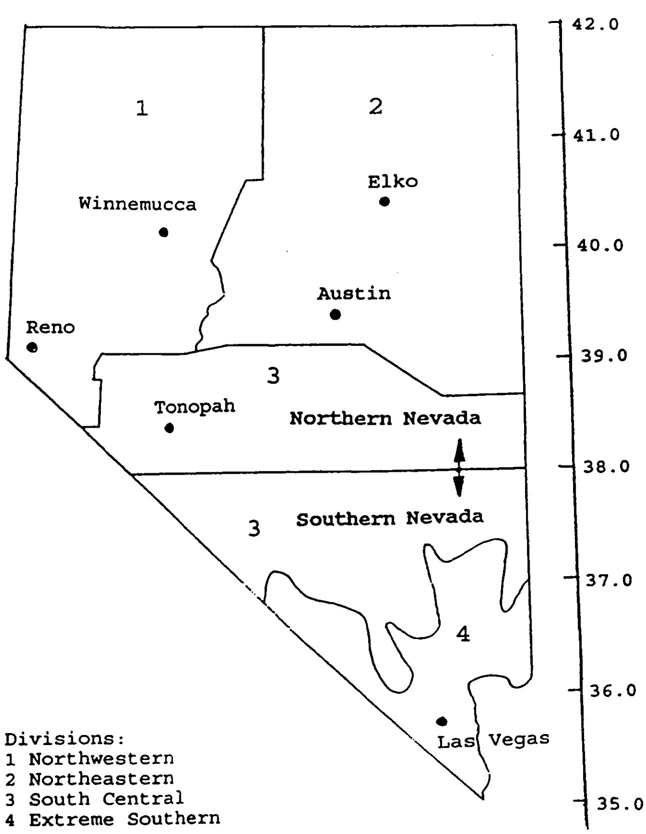

FIGURE 1 Geographical division of Nevada.

analysis. Independent variables are either discrete (constant) or continuous (changing) in nature. The discrete variables include the bridge components, structure type, rehabilitation history, and geographical location, whereas age and ADT are continuous variables. However the ADT can be translated from a continuous to a discrete variable by grouping it into three ranges, such as 0 to 1,000, 1,001 to 10,000, and more than 10,000 vehicles per day. When the bridges are divided into ADT groups, age remains the only continuous variable.

To include all of the variables, separate deterioration models were developed for each combination of the five discrete variables. Therefore the records that have the same bridge component, structure type, rehabilitation history, ADT range, and geographical location were combined. A group is the result of each combination of these five discrete variables. Once the bridge deterioration models (bridge performance curve of condition rating versus age) were developed, the effects that the six variables have on bridge deterioration could be evaluated. The initial model version (version 1) included all five discrete variables (Table 1): (a) three bridge components, (b) three structure types, (c) four rehabilitation statuses, (d) three ADT levels, and (e) two geographical locations. If all of the possible combinations had been used there would have been a total of 216 groups.

The data base was obtained from the Nevada National Bridge Inventory (1991) and from FHWA (1979 to 1990). The total data base contained 16,760 inspection records of all of the bridges in Nevada for the years 1979 to 1991. The inspection years before 1979 were not used because of inconsistencies in condition rating descriptions. Each inspection record consists of one inspection report that includes more than 90 coded variables describing the features and characteristics of a structure. The 10 variables selected from each record because of their impact on the bridge deterioration analysis were structure number; geographical location; structure type; ADT; numerical condition ratings for the bridge deck, superstructure, and substructure; construction date; last reconstruction year; and inspection year.

There were four procedures for data filtering. Initial filtering deleted records for structures other than bridges (e.g., culverts) and limited bridges to reinforced concrete, steel, and prestressed concrete. The initial filtering reduced the data base from the original 16,760 records to 11,536 records. The data were then screened for duplicate records caused by bridges located over highways; this decreased the data base from 11,536 to 8,601 records. Duplicate records from 2 or more consecutive years existed for almost every bridge since each structure was typically inspected every 2 years. Repeat records from the bridge's noninspection years were deleted. The removal of these duplicate records left 4,237 records for reinforced concrete, steel, and prestressed concrete bridges inspected from 1979 to 1991. The final filter eliminated records containing missing or miscoded information and excluded records containing condition rating increases of two or more between inspections without corresponding histories of rehabilitation. This was done because, most likely, a bridge received a rehabilitation that was not recorded. If these records were not removed these types of data would encourage an upward trend in the deterioration curves. After the final filter, 4,180 records were available for use in the bridge deterioration models. Of the 4,180 records, 2,457 were for reinforced concrete bridges (59 percent), 1,038 were for steel bridges (25 percent), and 687 were for prestressed concrete bridges (16 percent). Once

TABLE 1 Model Parameters

	Model Version			
Parameter	1	2	3	4
Component	1) Deck 2) Superstructure 3) Substructure	1) Deck 2) Superstructure 3) Substructure	1) Deck 2) Superstructure 3) Substructure	1) Deck 2) Superstructure 3) Substructure
Bridge Type	1) Reinforced Conc. 2) Steel 3) Prestressed Conc.	1) Reinforced Conc. 2) Steel 3) Prestressed Conc.	1) Reinforced Conc. 2) Steel 3) Prestressed Conc.	1) Reinforced Conc. 2) Steel 3) Prestressed Conc.
Rehabilitation	1) Non-or Rehab. before 10 yrs. 2) 10 to 24 yrs. 3) 25 to 39 yrs. 4) ≥ 40 yrs.	1) Non-or Rehab. before 10 yrs. 2) 10 to 24 yrs. 3) ≥ 25 yrs.	1) Non-or Rehab. before 10 yrs. 2) ≥ 10 yrs.	Not a Variable
ADT	1) 0 to 1,000 2) 1,001 to 10,000 3) >10,000	1) 0 to 1,000 2) >1,000	Not a Variable	Not a Variable
Location	1) Northern NV 2) Southern NV	Not a Variable	Not a Variable	Not a Variable
Groups- Max.	216	54	18	9
Groups- Actual	120	42	15	9

the data base was filtered the data were placed into 216 modeling groups.

Of the original 216 groups, many of them had no data or very few data (fewer than 10 inspection reports). Seventy-five groups had no data, and 21 groups had fewer than 10 inspection reports. Those that had fewer than 10 inspection reports were combined with the next closest group. This reduced the actual number of groups to 120. This shows the difficulty in dividing the data into precise groups. As the number of groups increased the amount of data per group was reduced; this can significantly affect the accuracy of the models. The philosophy of including as many variables as possible works well when there is a significant amount of data. The inclusion of many variables can adversely affect the reliability and applicability of the models when there are limited data. By using a few variables the amount of data in each group increases and the reliability also increases. This gives rise to a trade-off between providing groups with the greatest amount of detail or fewer groups with sufficient data for each group. The problem was solved by establishing three more versions of the variable combinations (Table 1). In versions 2, 3, and 4 there were 42, 15, and 9 data sets, respectively. The purpose of these other versions was to increase the amount of data per group and to provide a secondary model for cases in which the results from version 1 were not correct (e.g., increasing condition rating with time and condition ratings greater than 9). The four models created a total of 186 data sets. Since the number of bridges was limited, all of the data were used to develop the model. Therefore there were no data available for a verification set.

The basic model used was an eight-parameter model (Equation 1).

$$Y(t) = (1 - A)(1 - B)(1 - C)\alpha_1 e^{-t/\beta_1} + A\alpha_2 e^{-t-t_{r1}/\beta_2} + B\alpha_3 e^{-t-t_{r2}/\beta_3} + C\alpha_4 e^{-t-t_{r3}/\beta_4} \quad (1)$$

where

$Y(t)$ = projected bridge condition rating;

t = bridge age (years);

t_{r1}, t_{r2}, and t_{r3} = bridge ages in the years when a major rehabilitation was conducted on a bridge (for the Nevada model these years were set at t_{r1} = 10 years, t_{r2} = 25 years, and t_{r3} = 40 years);

α_1 = condition rating intercept at age zero;

β_1 = exponential decay coefficient for bridges that have not been rehabilitated or have rehabilitation ages of less than 10 years;

α_2, α_3, and α_4 = condition ratings at the ages of 10, 25, and 40 years for bridges that have been rehabilitated between the ages of 10 and 24, 25 and 39, and 40 years and older, respectively; and

β_2, β_3, and β_4 = exponential decay coefficients after rehabilitations for the same respective intervals.

If a bridge has not been rehabilitated or has been rehabilitated at an age of less than 10 years, *A*, *B*, and *C* are 0. If a bridge has been rehabilitated between the ages of 10 and 24 years, then *A* equals 1 and *B* and *C* stay 0. If a bridge has been rehabilitated between the ages of 25 and 39 years, then *B* equals 1 and *A* and *C* are 0. If the rehabilitation occurs after the age of 40 years, then *C* equals 1 and *A* and *B* are 0. The coefficients for the model were determined statistically by using the Statistical Analysis System (*7*). Figure 2 shows the basic eight-parameter model.

As the number of variables is reduced, including the number of rehabilitation intervals, the model becomes simpler. Equations 2, 3, and 4 are the general equations for the six-, four-, and two-parameter models, respectively (*5*).

$$Y(t) = (1 - A)(1 - B)\alpha_1 e^{-t/\beta_1} + A\alpha_2 e^{-(t-t_{r1})/\beta_2} + B\alpha_3 e^{-(t-t_{r2})/\beta_3} \quad (2)$$

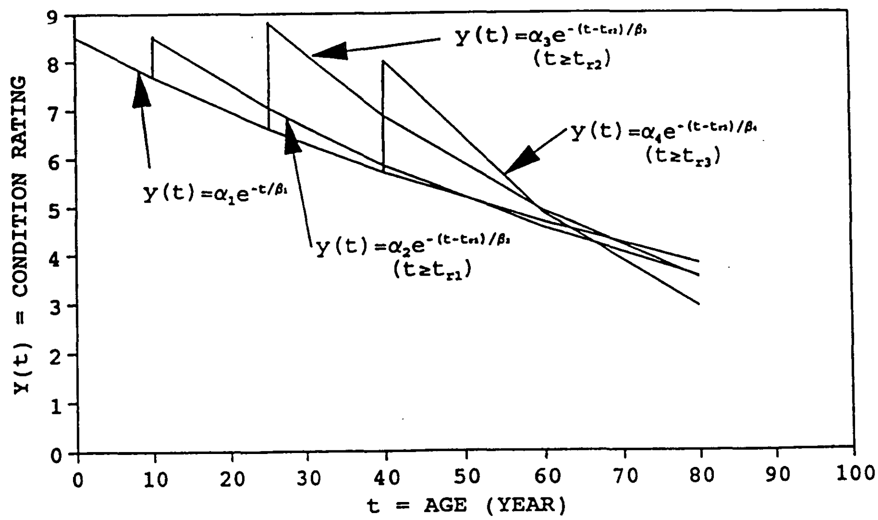

FIGURE 2 Eight-parameter deterioration model.

$$Y(t) = (1 - A)\alpha_1 e^{-t/\beta_1} + A\alpha_2 e^{-(t-t_{r1})/\beta_2} \tag{3}$$

$$Y(t) = \alpha_1 e^{-t/\beta_1} \tag{4}$$

Figures 3, 4, and 5 show examples of the deterioration curves that were developed by using the eight-parameter model. It is possible to obtain curves over a wide range of years since the bridge ages during the years from which data were collected (1979 to 1991) varied widely. Problems with the models that were developed included unrealistically high condition ratings at the ages of 10, 25, and 40 years because of rehabilitations, flat deterioration curves, and curves with increasing condition ratings with increasing age. To combat these problems the data were examined to see if these problems were caused by poor data, insufficient data, or improper use of the data.

MODEL DIFFICULTIES AND MODIFICATION

Two primary problems with the data and with the models developed from those data were identified:

1. The spike for the model was at discrete points in time (10, 25, and 40 years), and
2. Few data existed near the time of rehabilitation.

The result of the first problem is that if there are rehabilitation data along the entire interval the curve will be very flat (Figure 6). If the bridge data within the interval show that most of the rehabilitations were done toward the end of the interval, a high spike for the effect of the rehabilitation can occur (Figure 7). For

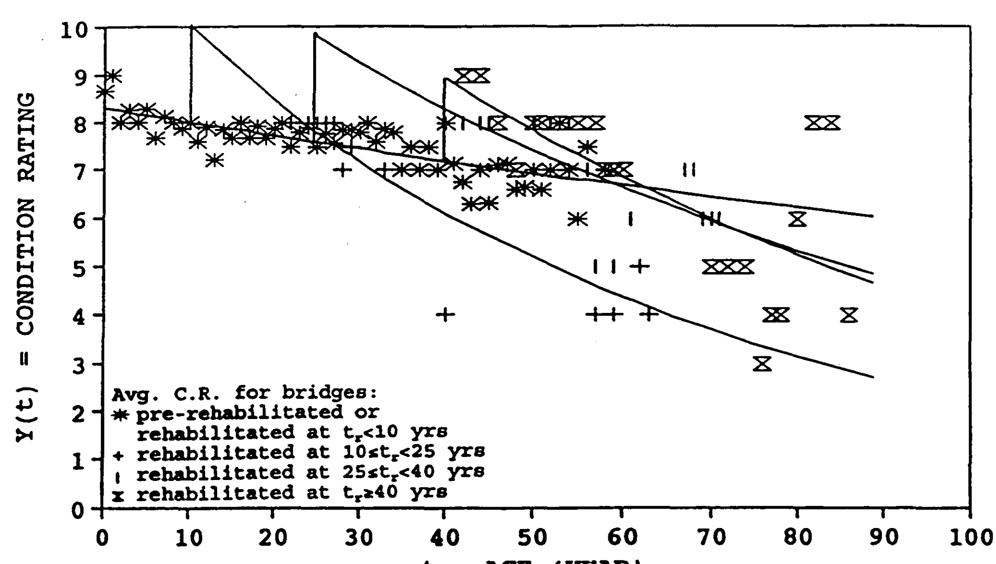

FIGURE 3 Deck deterioration, reinforced concrete bridges in northern Nevada with ADT ≤ 1,000 (unmodified data).

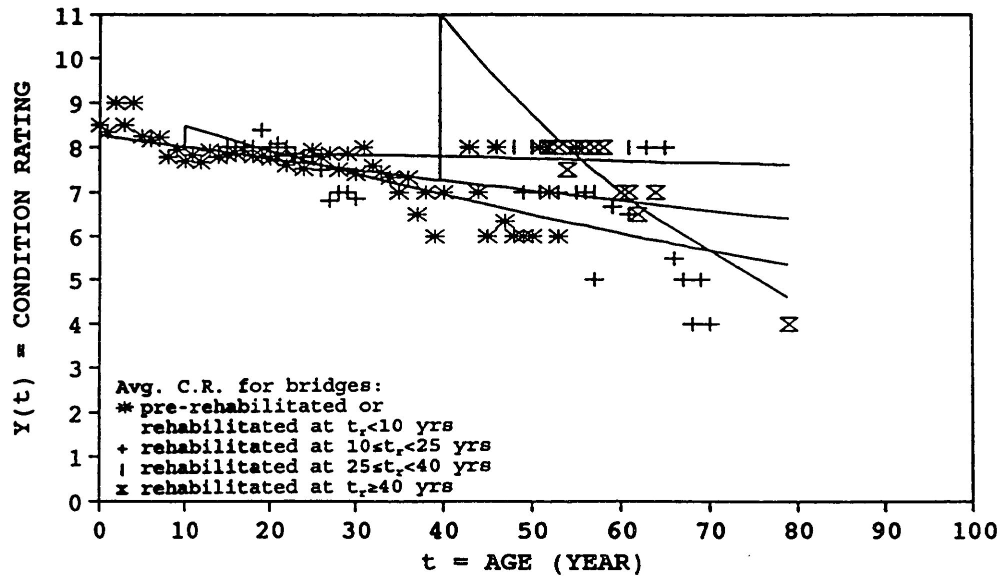

FIGURE 4 Deck deterioration, reinforced concrete bridges in northern Nevada with 1,000 < ADT ≤ 10,000 (unmodified data).

instance in Figure 3 there are no datum points (plus signs) between the ages of 10 and 20 years for the development of the second curve (rehabilitated between the ages of 10 and 24 years). The plus signs were concentrated between the ages of 20 and 24 years. Since the regression line is the best-fit line through the data, a high spike occurs.

If it is assumed that the deterioration rate after a rehabilitation is approximately the same within each interval (10 to 24, 25 to 39, and 40 or more years), the data can be shifted to the one year chosen as the rehabilitation year for each interval. Figure 6 shows several different inspection data sets for bridges rehabilitated between the ages of 10 and 24 years. Figure 8 shows the new curve that results from the shifting of the individual curves so that all of the rehabilitations occur at 10 years. For example to develop a deterioration curve for a bridge that was rehabilitated at 18 years of age, the rehabilitation year would be shifted to 10 years and all the inspection dates after rehabilitation would be shifted back 8 years. This concentrates the data and starts the deterioration from the same point in time.

The second problem was caused by too few data being available immediately after the rehabilitations. Therefore the regression analysis frequently predicted rehabilitation spikes that were much

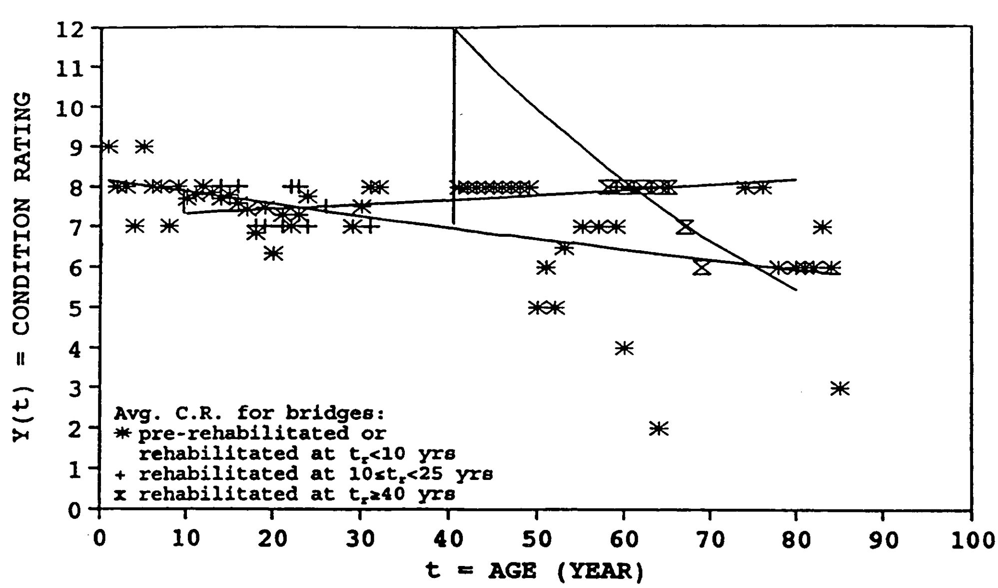

FIGURE 5 Deck deterioration, reinforced concrete bridges in northern Nevada with ADT > 10,000 (unmodified data).

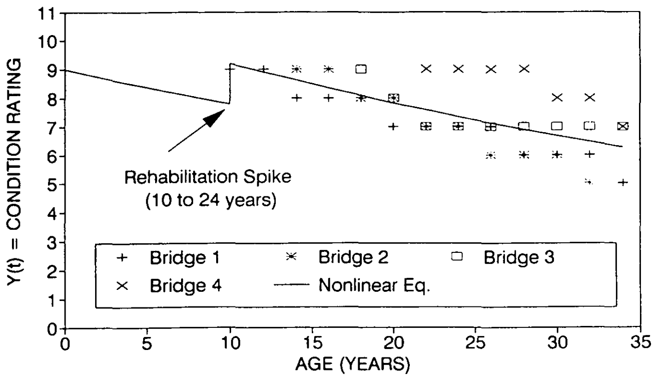

FIGURE 6 Rehabilitation spread along entire interval.

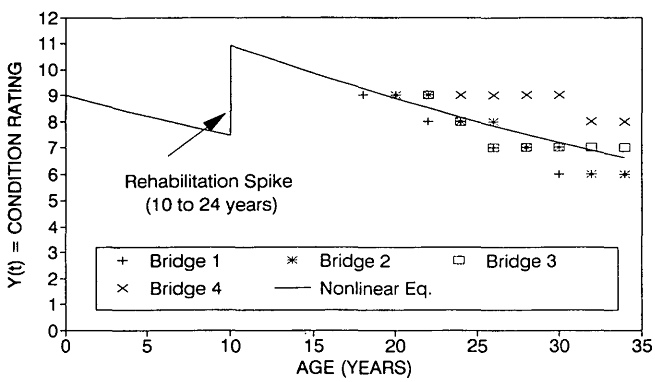

FIGURE 7 Rehabilitation concentrated near end of interval.

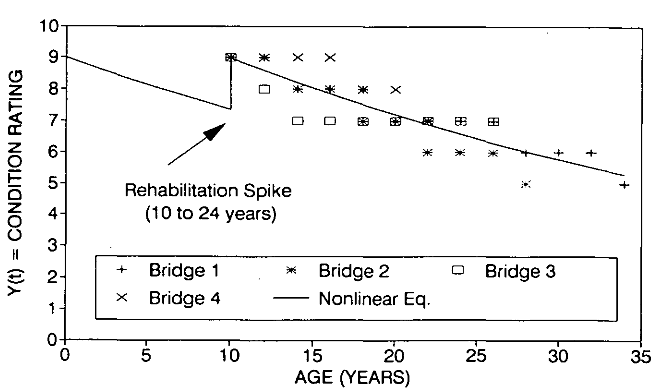

FIGURE 8 Effect of rehabilitation shifting.

too high: inspection rating scores of 10 and 11 on a scale of 1 to 9. Figure 9 shows what can happen. With no data near the rehabilitation year, the best-fit curve is dominated by the later inspection data and the curve is above 9 at the rehabilitation time.

In almost all cases a component of a bridge that has been rehabilitated will receive a rating of 8 or 9 for the first inspection score after a rehabilitation occurs. These data were not available for many of the bridges since they were rehabilitated before 1979. To overcome this shortcoming data were added for the rehabilitation inspection year for bridges for which data were not available. The number of datum points added was equal to the number of bridges within a rehabilitation interval for which inspection data were not available for the years immediately after rehabilitation. The magnitude of the inspection condition rating that was added was 8. A score of 8 is a conservative estimate of the post-rehabilitation inspection score. An example would be a bridge rehabilitated at 18 years of age for which an inspection record for that year was not available. This bridge would first have its inspection data shifted back 8 years, and then a datum point of 8 would be added at 10 years. Figure 9 shows an example of the data after they have been modified to include the inspection year data. The modified curve (W/Rehab Insp. Data) has a more realistic rehabilitation spike at 10 years.

Figures 10, 11, and 12 are the deterioration curves for the same categories in Figures 3, 4, and 5, respectively, except that the curves are based on the modified data set. For all of the deterioration curves there is a drop in the maximum inspection rating at the spike. In comparing Figures 3 and 10, the most significant change is in the deterioration curve for bridges rehabilitated after the age of 40 years. The modified data predict a much higher deterioration rate. This is due to the shifting of the rehabilitation to the same year (40 years). Figure 11 shows all three of the deterioration curves with decreasing condition ratings, whereas Figure 4 shows the deterioration curve for 10 to 25 years going upward. Figure 4 also shows a very large spike for the deterioration curve for age 40 years and older. Figure 12 is an example of how a small data set may still cause problems even with modifications. The deterioration curve for rehabilitations done at between 10 and 25 years of age is increasing with time. In this case the version 2 curve would be used to predict deterioration.

APPLICATION

The deterioration curves that were developed can be used as a predictive tool for an individual bridge as well as a group of bridges. In most cases the more detailed curves should be used (version 1). As seen in Figure 12, for cases in which version 1 gives unreasonable results, a more general model (version 2, 3, or 4) should be used. The following examples show how the deterioration curves are used effectively.

Example 1

A reinforced concrete substructure in northern Nevada has an ADT of less than 1,000. The substructure was rehabilitated at the age of 15 years. It is currently 20 years old and has a condition rating of 7. Two curves are shown in Figure 13 (*top*). The solid line is the average curve developed from the data base. The dotted curve is the average curve shifted to match the current data from the reinforced concrete bridge. By using a bridge-specific deterioration curve a more accurate model is possible for the individual bridge.

Example 2

A reinforced concrete superstructure in northern Nevada has an ADT of between 1,000 and 10,000. The superstructure was rehabilitated at the age of 30 years. It is currently 36 years old and

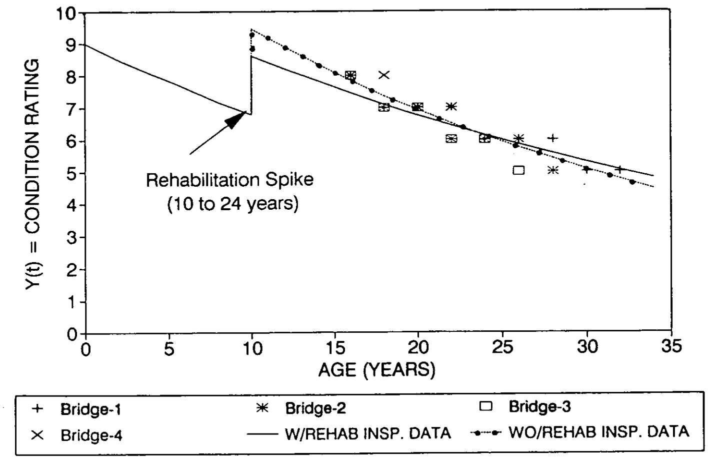

FIGURE 9 Inspection year data addition.

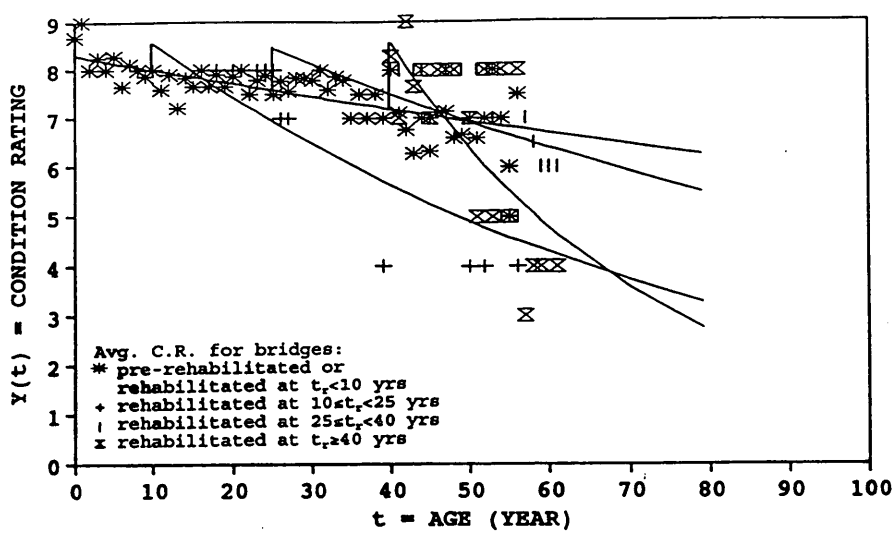

FIGURE 10 Deck deterioration, reinforced concrete bridges in northern Nevada with ADT ≤ 1,000 (modified data).

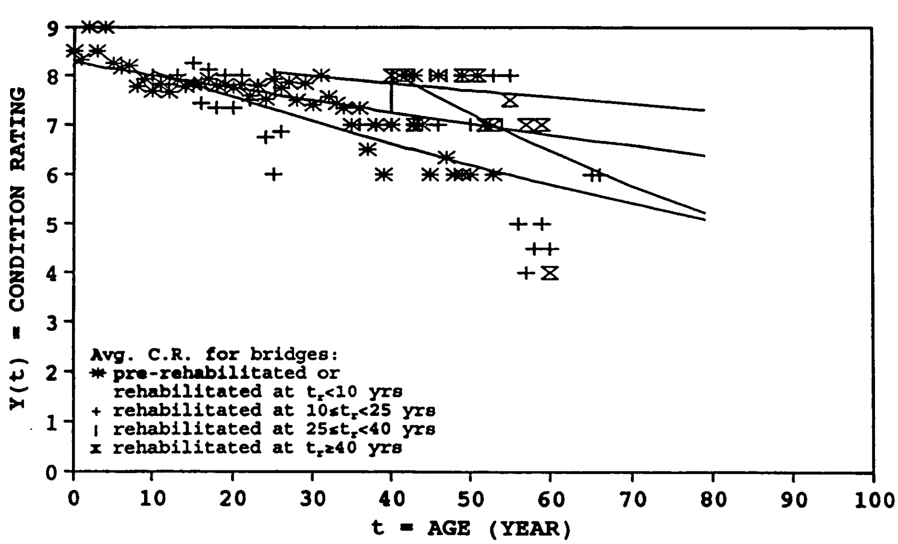

FIGURE 11 Deck deterioration, reinforced concrete bridges in northern Nevada with 1,000 < ADT ≤ 10,000 (modified data).

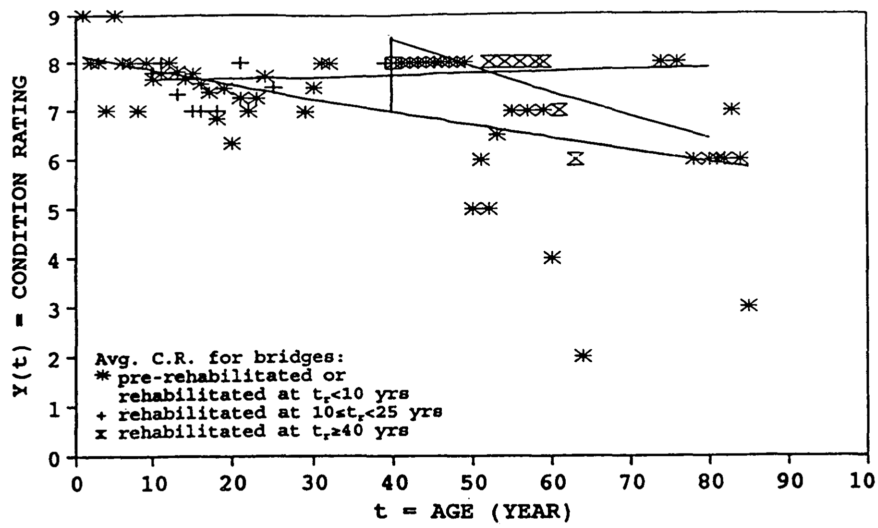

FIGURE 12 Deck deterioration, reinforced concrete bridges in northern Nevada with ADT > 10,000 (modified data).

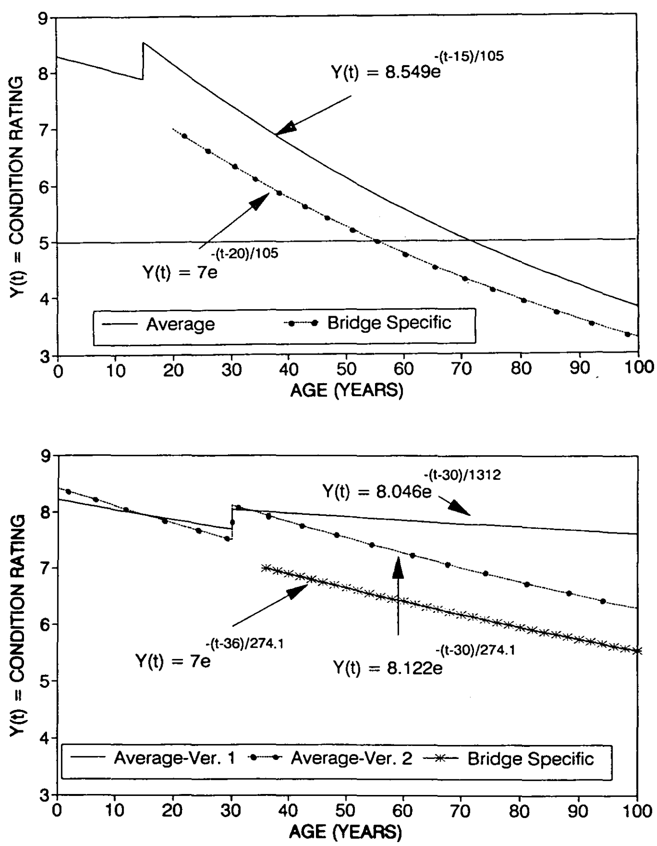

FIGURE 13 Deterioration curve: *top*, example 1; *bottom*, example 2.

has a condition rating of 7. Even after the modification of the data the deterioration curve (version 1) is still invalid because of the small data set [Figure 13 (*bottom*)]. It is necessary to use the second version of the deterioration curve to have a valid curve. The second version eliminates the environmental parameter (geographical location) and has two ADT intervals (≤1,000 and >1,000). The second version of the curve is then shifted to correspond to the current bridge data.

SUMMARY AND RECOMMENDATIONS

The ability to estimate bridge performance is a key aspect of a bridge management system and is necessary for devising optimal strategies for the maintenance, rehabilitation, and replacement of bridges. Therefore it is essential that models of bridge deterioration that accurately estimate remaining performance be developed. This is a difficult task for states that have just a few bridges of each type.

This paper provided a description of deterioration models and modifications to the data base that produced deterioration models that resulted in reasonable projections for deterioration. Occasionally (example 2) coarser models (e.g., version 2) must be used to establish reasonable trends. Such strategies are necessary for states like Nevada that have small bridge inspection data bases.

A conclusion from the study is that the data base could be enlarged by cooperating with other states. Some states have sufficiently large bridge inventories to easily establish accurate bridge deterioration models. Other states have small inventories but have much in common with their surrounding states. For example the bridge environment in northern Nevada is very similar to those in eastern Oregon, southern Idaho, and western Utah. The bridge environment in southern Nevada is similar to those in northern New Mexico and Arizona. It would be very easy to exchange the bridge inventory data, since they are stored in approximately the same format, and to develop common deterioration models. Cooperative agreements would be a great asset to all of the states involved. This would allow enough data to permit a verification data set and more confidence in the deterioration curves. This technique for increasing the size of small data bases would work in many parts of the country.

The deterioration models that were developed established some basic trends in bridge deterioration. They are as follows.

1. Postrehabilitation decay is greater than decay of a new bridge. This implies that although it is possible to increase the bridge condition rating through rehabilitation, the rehabilitated bridge is not new and will experience an accelerated rate of deterioration compared with that for a new bridge. The bridges with rehabilitations at a bridge age of more than 40 years had the greatest postrehabilitation decay rates. Therefore the earlier that rehabilitations are done the better the postrehabilitation performance. The result and the comparison that may be made are affected by unrecorded rehabilitations. These unrecorded rehabilitations cause a reduction in the projected decay of unrehabilitated bridges and cause a perception that a rehabilitated bridge will have a shorter life than a unrehabilitated bridge. The unrecorded bridge rehabilitations are a big problem. The only way to solve this problem is to go back through the contract data. This would be a very large task.
2. The prestressed concrete bridges are especially sensitive to ADT.
3. Bridges in northern Nevada deteriorate faster than those in southern Nevada. Northern Nevada has a significantly harsher winter environment (freezing, thawing, and salt application) than southern Nevada.
4. Decks deteriorate faster than the superstructure and the substructure.

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Bayesian Updating of Infrastructure Deterioration Models

YUN LU AND SAMER MADANAT

Deterioration forecasting plays an important role in the infrastructure management process. The precision of facility condition forecasting directly influences the quality of maintenance and rehabilitation decision making. One way to improve the precision of forecasting is by successive updating of deterioration model parameters. A Bayesian approach that uses inspection data for updating facility deterioration models is presented. As an empirical study with bridge deck data indicates, the use of this methodology significantly reduces the uncertainty inherent in condition forecasts.

The process of infrastructure management consists of three activities: data collection and inspection, deterioration modeling and forecasting, and maintenance and rehabilitation (M&R) decision making (*1*). After facility condition data are collected, deterioration models are developed by using these data to forecast future facility performance; both data and models are later used to support M&R decision making. One fact that should be emphasized is that the relationship between condition data and deterioration models has traditionally been a static one; that is, once a deterioration model is developed, subsequently collected data are not used for model updating.

In contrast to this approach this paper presents a method that exploits the condition data collected during facility inspections to improve the precision of deterioration models. In this method data are used not only for deterioration modeling but also for model updating.

The advantage of this method is that one can start to develop a model even with limited data. Later the model can be updated as additional data become available. Therefore this approach increases the precision of forecasting and is expected to decrease facility life cycle costs.

The updating method used in this paper is the Bayesian approach, which will be introduced in the next section. PONTIS (*2*), the California Department of Transportation–FHWA network optimization system for bridge improvement and maintenance, used the same methodology to update the transition probabilities representing bridge deterioration. The only difference between the work presented in this paper and the updating procedure used in PONTIS is that the present research deals with a continuous deterioration model, whereas PONTIS is based on a discrete model that uses Markov transition probabilities as model parameters.

BAYESIAN APPROACH

Bayesian analysis is defined as the approach to statistics that formally seeks to use prior information. In statistics the Bayesian approach is widely used to estimate an unknown parameter Θ. Bayesian analysis is performed by combining the prior information and the sample information (x) into what is called the posterior distribution of Θ given x, from which all decisions and inferences are made (*3*).

Let $\pi(\Theta)$ denote the prior distribution of Θ and let $L(x, \Theta)$ denote the likelihood function, then Bayes's theorem can be expressed as follows:

$$\pi(\Theta|x) = \frac{L(x, \Theta)\pi(\Theta)}{\int_{-\infty}^{\infty} L(x, \Theta)\pi(\Theta)d(\Theta)} \quad (1)$$

where

$L(x, \Theta) = f(x|\Theta)$ = likelihood of experimental outcome x, that is, conditional probability of obtaining a particular experimental outcome assuming that the parameter is Θ;

$\pi(\Theta)$ = prior probability of Θ, that is, before availability of experimental information;

$\pi(\Theta|x)$ = posterior probability of Θ, that is, probability that has been revised in the light of experimental outcome x.

It is observed from Equation 1 that both the prior distribution and the likelihood function contribute to the posterior distribution of Θ. The prior information enters the posterior probability density function (pdf) via the prior pdf, whereas all of the sample information enters via the likelihood function. In this manner judgmental and observational data are combined properly and systematically (*4*).

Likelihood Function

For observed data, x, the function $L(x, \Theta) = f(x|\Theta)$ considered as a function of Θ is called the *likelihood function.*

Given a set of observed values $x_1, x_2, \ldots, x_n$, which represent a random sample from a population of X with underlying density $f_x(X)$, the probability of observing this particular set of values, assuming that the parameter of the distribution is Θ, is

$$f(x|\Theta) = L(x, \Theta) = \prod_{i=1}^{n} f_x(x_i|\Theta) \quad (2)$$

From Equation 2 it can be observed that the likelihood function $L(x, \Theta)$ is the product of the density function of X evaluated at $x_1, x_2, \ldots, x_n$.

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Prior Information

Generally the prior information comes from past experience, the nature of the problem, or previous work. A prior distribution for which $\pi(\Theta)$ can be easily calculated is the so-called conjugate prior. For example the class of normal priors is a conjugate family for the class of normal (sample) densities. That is if X has a normal density and Θ has a normal prior, then the posterior density of Θ given x is also normal.

A conjugate prior greatly simplifies the application of Bayes's theorem for determination of a posterior distribution. Conjugate distributions provide a convenient model that may be realistic in many situations.

Posterior Distribution

The posterior distribution is the combination of the prior information and the likelihood function. Just as the prior distribution reflects beliefs about Θ prior to experimentation, so $\pi(\Theta|x)$ reflects the updated beliefs about Θ after (posterior to) observing the sample x. In other words the posterior distribution combines the prior beliefs about Θ with the information about Θ contained in the sample x to give a composite picture of the final beliefs about Θ.

The two important quantities of the posterior distribution are the mean and the variance. The mean value of Θ that is used as the Bayesian estimator of the parameter is

$$E^{\pi(\Theta|x)}(\Theta) = \int_{-\infty}^{\infty} \Theta\pi(\Theta|x)d\Theta \tag{3}$$

and the variance is given by

$$V^{\pi(\Theta|x)}(\Theta) = \int_{-\infty}^{\infty} \Theta^2\pi(\Theta|x)d\Theta - [E^{\pi(\Theta|x)}(\Theta)]^2 \tag{4}$$

In the case of a conjugate distribution, once the mean and the variance of the posterior distribution are calculated, one can directly write its probability density function.

The Bayesian approach has many advantages in the area of engineering planning and design. It systematically combines uncertainties associated with randomness and those arising from error of estimation and prediction. It provides a formal procedure for systematic updating of information and increases the prediction precision.

APPLICATION OF UPDATING METHODOLOGY

In this section the Bayesian approach is applied to the problem of updating facility deterioration models. A logistic model representing the fraction of bridge deck area delaminated is used as an example.

The logistic model has an S shape; its attractive mathematical property is that it has a bounded function that lies between 0 and 1; it is therefore well-suited for representing the progression of the damaged fraction of a bridge deck. Figure 1 shows an application of the logistic model that represents the percentage of a

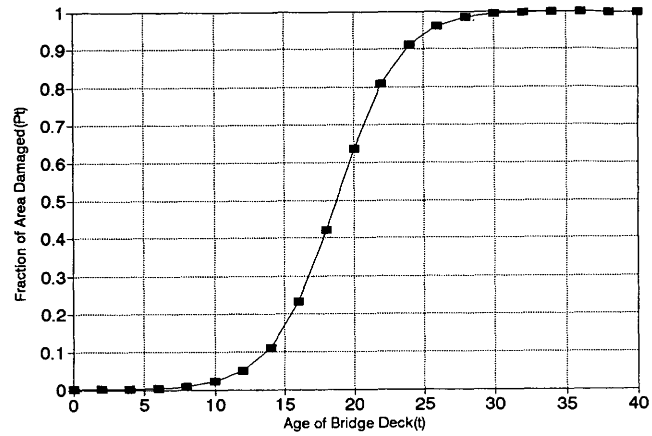

FIGURE 1 Fraction of bridge deck area damaged.

bridge deck area damaged (P_t) from year 0 to year 40 during which no maintenance or rehabilitation is performed. It can be observed from this graph that between years 0 and 10 there is very little damage since the bridge is new; after year 10 the rate of deterioration increases rapidly and P_t reaches 0.94 at year 25. After that the deterioration tends to slow again. This trend is consistent with observations of bridge deck deterioration over time (*5,6*).

The mathematical function of the logistic model is of the form

$$P_t = \frac{1}{1 + e^{a+bt+\epsilon}} \tag{5}$$

where

P_t = fraction of area of bridge deck damaged;
t = age of bridge deck;
a, b = parameters specific to each bridge deck type; and
ϵ = a random error term that captures the uncertainty associated with the deterioration process; it is usually assumed to be normally distributed, with mean μ_ϵ equal to 0 and variance σ_ϵ^2 (the variance σ_ϵ^2 is bridge deck type specific).

In Equation 5 there are two parameters, a and b, that determine the rate of deterioration. Generally if sufficient observations of bridge decks of a given type consisting of t and P_t are available, these two parameters can be estimated statistically.

In reality no deterioration model is perfectly accurate because of the limited sample size, the inherent randomness of the process, observation errors, and so on. The Bayesian approach can be used to update the deterioration model parameters to increase the precision of forecasting. For mathematical simplicity this paper considers updating parameter b only while treating parameter a as constant. Extending the method to update both parameters simultaneously is conceptually straightforward, but somewhat mathematically cumbersome.

Derivation of Prior Distribution and Likelihood Function

To use Bayes's theorem one needs to find out the prior distribution of b and the likelihood function $L(P_t, b)$.

The prior distribution of b is easy to determine. If regression is used to estimate the parameters of Equation 5, then the parameter b usually can be assumed to be normally distributed with mean μ_b and variance σ_b^2 (7). Therefore the prior density function of b can be written in the form

$$\pi(b) = \frac{e^{-\frac{1}{2}\frac{(b-\mu_b)^2}{\sigma_b^2}}}{\sqrt{2\pi}\,\sigma_b} \tag{6}$$

To obtain the likelihood function the density function of P_t must first be generated.

In general if δ is normally distributed with mean a and variance σ^2 and δ is log τ, then τ will be log-normally distributed (8). In Equation 5 let $x = e^{a+bt+\epsilon}$ and $\Theta = a + bt + \epsilon$, then $x = e^{\Theta}$. Since $\Theta = a + bt + \epsilon$ is normally distributed and $\Theta = \log x$, x is therefore log-normally distributed according to the above rule. Its density function is of the form

$$f(x) = \frac{e^{\frac{-(\log x-a-bt)^2}{2\sigma_\epsilon^2}}}{\sqrt{2\pi}\,x\sigma_\epsilon} \tag{7}$$

Generally, if the pdf of x is known as $f(x)$ and $y = h(x)$, one can obtain the pdf of y by the following relation:

$$f(y) = \left|\frac{\partial x}{\partial y}\right| f(x) \tag{8}$$

Therefore the density function of P_t is given by

$$f(P_t) = \left|\frac{\partial x}{\partial P_t}\right| f(x) = \frac{e^{\frac{[\log\left(\frac{1}{P_t}-1\right)-a-bt]^2}{-2\sigma_\epsilon^2}}}{\sqrt{2\pi}\,\sigma_\epsilon P_t(1-P_t)} \tag{9}$$

The likelihood function is the product of the pdf of P_t over all observations:

$$L(P_t, b) = \prod_{i=1}^{k} f(P_{t,i}|b) = \frac{e^{\sum_{i=1}^{k}\frac{[\log\left(\frac{1}{P_{t,i}}-1\right)-a-bt]^2}{-2\sigma_\epsilon^2}}}{\prod_{i=1}^{k}\sqrt{2\pi}\,\sigma_\epsilon P_{t,i}(1-P_{t,i})} \tag{10}$$

Derivation of Posterior Distribution

According to Equation 1 the posterior distribution of b is

$$\pi(b|P_t) = \frac{L(P_t, b)\pi(b)}{\int_{-\infty}^{\infty} L(P_t, b)\pi(b)d(b)} \tag{11}$$

Substituting $L(P_t, b)$ and $\pi(b)$ with Equations 10 and 6, respectively, one obtains

$$\pi(b|P_t) \propto \frac{e^{\sum_{i=1}^{k}\frac{\left[\log\left(\frac{1}{P_{t,i}}-1\right)-a-bt\right]^2}{-2\sigma_\epsilon^2}}}{\prod_{i=1}^{k}\sqrt{2\pi}\,\sigma_\epsilon P_{t,i}(1-P_{t,i})} \cdot \frac{e^{\frac{(b-\mu_b)^2}{-2\sigma_b^2}}}{\sqrt{2\pi}\,\sigma_b} \tag{12}$$

It is tedious to use this equation. Fortunately the prior pdf is a conjugate normal distribution. Therefore the posterior distribution is also a normal distribution, with the mean and the variance shown in Equations 13 and 14, respectively (the derivation of these two equations is omitted for simplicity; it can be shown that Equation 12 simplifies to a normal distribution).

$$\mu' = \frac{\sigma_b^2 t\left\{E\left[\log\left(\frac{1}{P_{t,i}}-1\right)\right]-a\right\}+\frac{\mu_b\sigma_\epsilon^2}{k}}{\frac{\sigma_\epsilon^2}{k}+t^2\sigma_b^2} \tag{13}$$

$$\sigma'^2 = \frac{\sigma_\epsilon^2\sigma_b^2}{\sigma_\epsilon^2 + kt^2\sigma_b^2} \tag{14}$$

Therefore the pdf of the posterior distribution is of the form

$$\pi(b|P_t) = \frac{e^{\frac{(b-\mu')^2}{-2\sigma'^2}}}{\sqrt{2\pi}\,\sigma'} \tag{15}$$

PARAMETRIC ANALYSIS

In this section a parametric analysis is performed to evaluate the effect of performing Bayesian updating on the forecasting precision of a logistic model of bridge deck deterioration. The deterioration model studied in this section has the following form (9):

$$P_t = \frac{1}{1+e^{a+bt+\epsilon}} = \frac{1}{1+e^{8.72-0.44t+\epsilon}} \tag{16}$$

Bayes's theorem was used to update the parameter b to increase the model's prediction precision. Hence the parameter b is treated as a random variable instead of a constant. Model estimation results indicated that the prior distribution of b is normal, with mean μ_b equal to -0.44 and standard deviation σ_b equal to 0.2 (9). By calculating the posterior mean and variance of b through Equations 13 and 14, the parameter b can be updated repeatedly, thus reducing the standard deviation of b. Therefore the standard deviation of P_t is also expected to decrease. Monte Carlo simulation was used in the study to compute the standard deviation of P_t after each model update. The use of Monte Carlo simulation was necessitated by the form of the deterioration model. As Equation 16 shows the relationship between the parameter b and P_t is strongly nonlinear, which makes the derivation of the variance of P_t as a function of σ_b analytically rather difficult.

Two cases are compared: one is without updating and the other is with updating. In the first case parameter b is normally distributed with mean μ_b equal to -0.44 and standard deviation σ_b equal to 0.2. Since no updating is performed in this case the mean and standard deviation of b will remain constant from year 2 to year 20, which is the horizon used in the analysis. In the second case Bayesian updating is performed every 2 years after each inspection cycle. The number of observations k collected in every inspection period in the parametric study is assumed to be 10. Each observation consists of a pair (t, P_t) pertaining to a particular bridge deck among the population of bridges. In this case the parameter b is still normally distributed, but the mean and standard deviation are updated every 2 years according to Equations 13 and 14, respectively. Figure 2 depicts the standard deviation of b under these two scenarios. It can be observed from Figure 2 that the standard deviation of b decreases significantly as a result of updating, especially during the first update.

To show that the prediction accuracy is increased by updating the parameter b, the standard deviation of P_t needs to be compared under these two cases. Monte Carlo simulation was used to calculate the standard deviation of P_t after each update. The results are summarized in Figure 3, which shows that the standard deviation of P_t becomes substantially smaller when the parameter b is updated. The shapes of the two curves in Figure 3 are instructive. The upper curve, corresponding to the nonupdating case, shows the standard deviation of the forecast of P_t increasing up to year 20, after which it decreases until it becomes 0 at year 40 (not shown in the figure). This behavior stems from the fact that the P_t function is bounded from below by 0 and from above by 1, which forces the standard deviation to 0 at these two extremes. The lower curve, which corresponds to the updating case, shows a similar behavior, except that the maximum forecast standard error occurs earlier because of the contribution of Bayesian updating to reducing the standard deviation of b.

A study of the change in infrastructure life cycle costs with the level of uncertainty in condition forecasting can be found in another study (*10*). Figure 4 is adapted from that study. In Figure 4 the x-axis shows the standard error of conditional forecasting measured in PCI (Pavement Condition Index) units, a measure of pavement performance. The y-axis shows the expected life cycle cost, which is the sum of agency costs and user costs over the

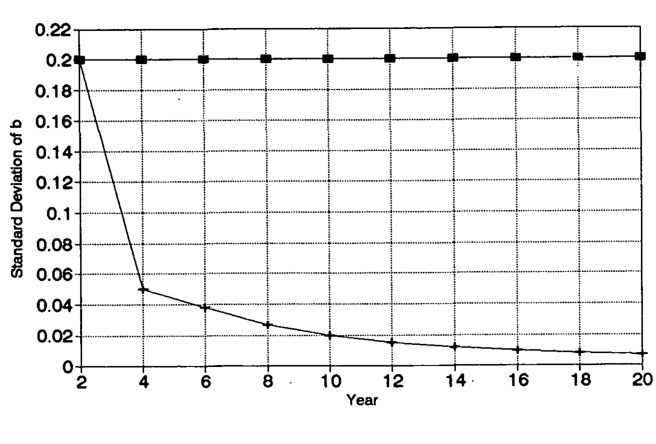

FIGURE 2 Standard deviation of *b* for updating and nonupdating cases.

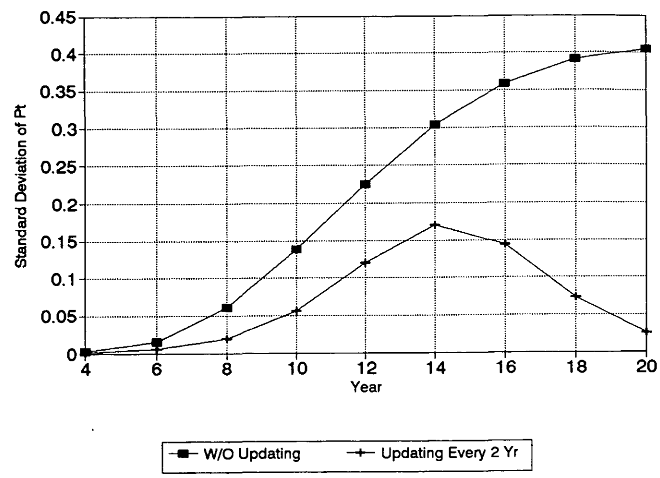

FIGURE 3 Standard deviation of P_t for updating and nonupdating cases.

planning horizon, associated with the optimal policies. The M&R decision model used to compute these minimum life cycle costs is a Markov decision process–based optimization model. Under this decision model the inspection frequency is predetermined once per time period. Figure 4 shows that increasing the uncertainty in forecasting of the condition leads to a substantial increase in the value of the minimum expected cost. This result demonstrates the economic benefits of improving the precision of the deterioration models used in pavement management. Although the application in the present study (bridge decks) is different from the one used in that study, a similar result should hold because of the similarities in the cost structures of the two problems.

CONCLUSION

In this paper a Bayesian methodology for updating deterioration models in infrastructure management was presented. This method has the following advantages:

1. It significantly decreases the uncertainty inherent in the forecasting of facility condition, thus decreasing the expected facility life cycle cost, and
2. It allows the development of a deterioration model even with limited data; the model is repeatedly updated and improved as inspection data are incorporated.

A deterioration model representing the damaged fraction of a bridge deck area was chosen as an application example of this methodology. As the empirical study indicated, the use of the methodology presented in this paper for updating the deterioration model parameter reduces the uncertainty associated with forecasting bridge deck condition significantly. Therefore the facility life cycle cost can be expected to decrease.

Although the sample problem dealt with a deterioration model for bridge decks, this methodology is also applicable to highway pavements as long as an appropriate deterioration model can be developed.

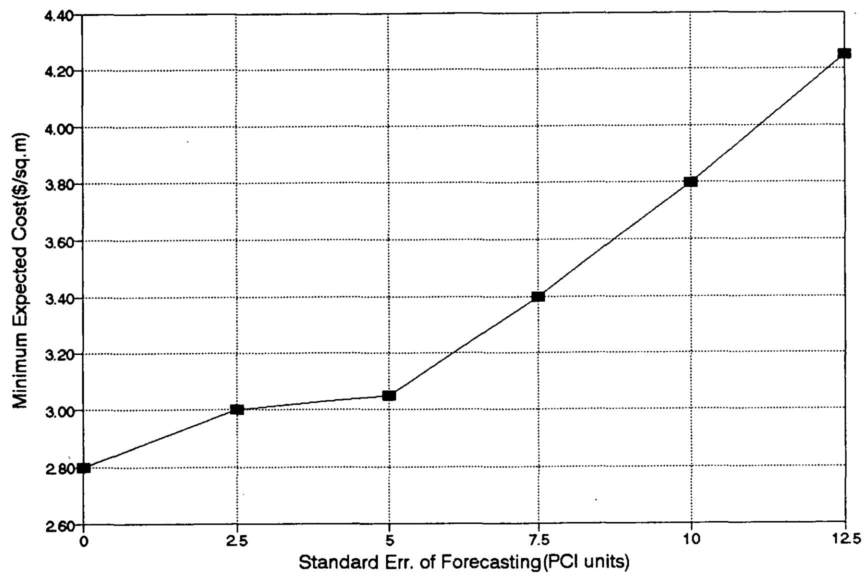

FIGURE 4 Minimum expected life cycle cost versus standard error of forecast.

To make this methodology operational and simplify the implementation, the following considerations should be taken into account:

1. The likelihood function plays an important role in Bayes's theorem. Therefore the choice of functional form for the deterioration model is a critical issue in the implementation of this methodology. To simplify the application of Bayesian updating, deterioration models whose forecasts follow a commonly known distribution such as a normal or a log-normal distribution are recommended.
2. Although many choices for the prior distribution are often available, a conjugate prior distribution is recommended because it facilitates the implementation of this methodology.

In this paper only parameter b was updated. Parameter a was treated as a constant for the sake of mathematical simplicity. The empirical study indicated that through updating parameter b the forecasting precision increased significantly. It is expected that the result will be even more significant if parameters a and b are updated simultaneously. The basic updating procedure will be the same as the one presented in this paper.

Generally deterioration models are classified into two categories: disaggregate and aggregate. PAVER (*11*) is an example of an aggregate model. The deterioration model described in this paper is a disaggregate model that uses an individual damage measurement as a measure of performance. Since an aggregate deterioration model represents a combination of different types of damage, updating of such model may be more difficult. The appropriate way to update such a model may be to update each damage model first and then to combine the results properly if individual damage models exist. On the other hand if the deterioration model used consists of a single condition index it will be necessary to perform Bayesian updating on that aggregate model directly. The application of Bayes' theorem for this case needs further study.

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Complete Package for Computer-Automated Bridge Inspection Process

S. S. KUO, DAVID A. CLARK, AND RICHARD KERR

A complete package for the Automation of Bridge Inspection Process, developed by the University of Central Florida for the Florida Department of Transportation (FDOT), is presented. FDOT's Inspection Forms A, B, and C for fixed bridges only were described previously. Form A contains the Report Identification and a Condensed Inspection Report. Form B is a comprehensive List of Deficiencies. Form C contains the Evaluation of Previous Corrective Action. The recently developed complete package covers fixed, culvert, and movable bridges. In addition to the previously reported forms, the package also includes Form D, Recommendations for Corrective Actions, and an automated work order system to replace Form E, formerly entitled Methods and Quantities. The automated system comprises five commercially available software programs and 25 developed programs; divided into two components, these are referred to as the *field* and *office systems*. The field system is used to collect all field inspection data. The inspection routines are operated with the use of a notebook computer and pen-based acquisition programs. The office system is used to process field data and to produce a final inspection report established by the National Bridge Inspection Standard. The system requires a desktop computer, a scanner, and a video capture card. Any graphical data collected in the field inspection are processed by use of the video capture, scanning, and image editing techniques. Automatic work orders are generated from deficient bridge elements specified in the inspection reports. Since the completion of the project five FDOT districts have implemented the automated system. Consequently the time spent producing the final report was substantially reduced, and inspectors were able to devote more time to performing field structural inspector to help ensure the safety and welfare of the public.

As published previously (*1*) a computer-automated system was developed for the purpose of creating a cost-effective bridge inspection program for the Florida Department of Transportation (FDOT). This preliminary system was developed for fixed bridges only, which used FDOT Inspection Forms A, B, and C. The recently completed package includes fixed, culvert, and movable bridges, an additional Form D, and an automated work order system that replaces Form E, formerly entitled Methods and Quantities. The complete package consists of 25 developed programs and five commercially available software programs (see Figures 1, 6, 7, and 8). The system can offer considerable time savings for inspectors while they are performing office work. With this system bridge inspection reports are standardized and work orders are effectively scheduled by the personnel involved.

Bridge inspection procedures require field and office work; therefore, the computer-automated system is divided into two components, referred to as the *field* and *office systems*. The field system uses a notebook computer to run field acquisition programs, whereas the office system uses a desktop computer, a scanner, and a video capture card to process the field data and to produce the final inspection report. Office programs were developed to work in conjunction with commercially available software to process field data and work orders. Field data such as video pictures and sketches can be digitized through a video capture card and scanner and can be printed in the inspection reports. The field and office bridge inspection procedures performed with the automated system and the results from a trial implementation by several district inspectors are discussed in the following sections.

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FIELD INSPECTION PROCEDURES

Field inspections performed with the developed automated system are thorough, accurate, and efficiently stored. The previous (old) bridge inspection report is downloaded to the field notebook computer (GRiDPAD) before the inspection routine is performed. Old inspection data are available for comparison and input if desired. The field system gives the inspector the opportunity to perform an inspection of a culvert, fixed, or movable bridge. The inspection routines are initiated with the use of a bat file and a program named M.BAT and FIELD.EXE (Figure 1). M.BAT checks for the field computer configuration and then loads FIELD.EXE, which displays three choices on the notebook computer. The choices are C, F, or M. The acquisition programs for culvert, fixed, or movable structures are invoked by tapping the corresponding letter on the computer screen. The inspection routines developed for each of these inspections collects the data that will be used to generate

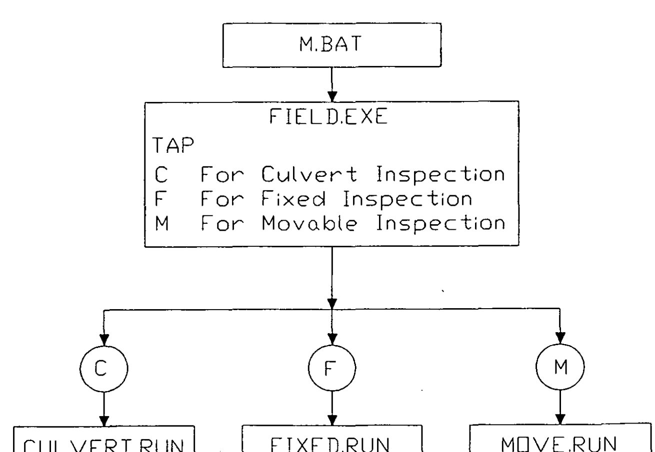


FIGURE 1 Flow chart of field system programs.

FDOT inspection reports on the office system. The field data are also used to produce work orders for bridge deficiencies cited as requiring service. FDOT uses inspection reports to fulfill the historical data requirements established by the National Bridge Inspection Standard. The report sections that are automated with the computer automated system are Forms A, B, C, and D. Form A comprises (a) a Cover Sheet (Report Identification) and (b) a Condensed Inspection Report (CIR). The Cover Sheet is used to identify geographical details such as bridge location and the beginning mile marker. The Cover Sheet also lists the bridge inspectors' names with their corresponding Certified Bridge Inspection numbers. The reviewing supervisor and confirming professional engineer with signatures are also assimilated in the Cover Sheet. The CIR lists every bridge element inspected along with a corresponding numerical condition rating (NCR). The NCR is a numerical value ranging from 0 to 9 for structural elements and from 0 to 4 for nonstructural elements. For the structural rating a 9 would represent the best value and a 0 would correlate to a failure condition. NCR definitions are given in Table 1. Form B is the Comprehensive List of Deficiencies, which identifies all deficient bridge elements ascertained during the bridge inspection. Graphical images of deficient elements may accompany Form B in media such as videos, photographs, or sketches. Form C is the Evaluation of Previously Recommended Corrective Actions. The purpose of Form C is to evaluate all elements that were previously reported as deficient and that required service during the last bridge inspection. The inspector specifies whether the corrective action has been performed satisfactorily or if continued service is required. Form D is entitled Recommendations for Corrective Actions, which denotes deficient bridge elements that require repair.

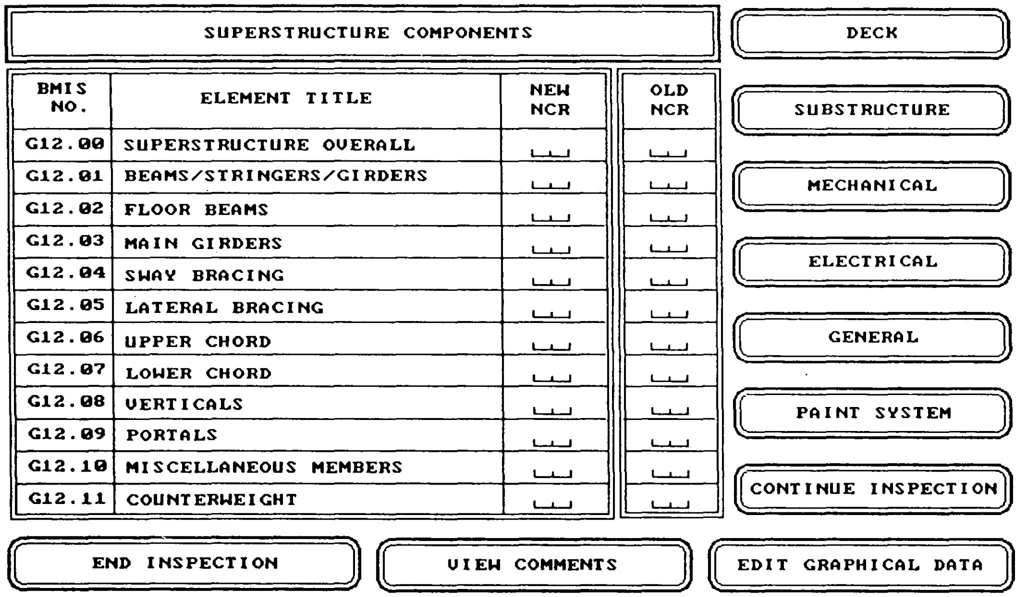
The field system uses a three-step data collection procedure. Step 1 collects data for Form A and Form B, while Steps 2 and 3 collect data for Forms C and D, respectively. The acquisition program provides flexibility while performing a field inspection. The field software was developed in Pen Pal Version 1.1 (*2*) and incorporated pen technology for use on the electronic notebook, yet desktop computer execution is possible. The programs are made up of a series of forms that contain their own independent source code. The forms take advantage of pen objects such as buttons, radio buttons, lists, fields, and text. Program execution is event driven, which means that a pen down, pen up, or drag with the notebook pen will trigger program logic execution. The source code pertaining to each form works in conjunction with the global code. The global code contains initiation routines and other procedures that can be "called" by any form.

All inspection reports from culvert, fixed, and movable bridges are divided into components such as the Superstructure, Substructure, Deck, Mechanical, Paint Systems, and so on. Each component has a list of bridge elements that require inspection as specified in the Bridge Management Inventory System (BMIS). For example the Deck component would contain elements related to the decking and expansion joints. The forms used in Step 1 data collection are divided into the same bridge components that appear on the FDOT inspection report. These forms display the BMIS elements that correspond to the component of interest. Figure 2 shows the Superstructure form with its accompanying elements. During a new inspection old inspection data are available for review or as input. Old NCRs are displayed in a column adjacent to the new NCRs or the NCRs from the current inspection. A series of forms works with each bridge component to assist with entering data or viewing or editing comments. An asterisk issued with an NCR denotes that a comment on that particular element will appear in Form B. When an asterisk is denoted by the inspector the acquisition programs will invoke Pen Pal forms to collect Form B data. The inspector has the ability to jump to any component of the bridge at any time while in Step 1 data collection. After all BMIS elements have been addressed, Step 1 data collection will then verify that all elements have been evaluated and that for all elements for which an asterisk was issued there is a corresponding comment in Form B. When the verification process is complete, Step 2 (Form C) data collection begins.

Step 2 data collection sequentially displays the previously recommended corrective actions and then allows the inspector to specify whether the service performed is adequate or inadequate. Two buttons are used in Form C to process a standard response, such as "Recommended corrective actions have been satisfactorily completed." If the BMIS element still requires service, a button titled "ADD TO FORM D" provides the ability to add this item to the current Form D (Recommendations for Corrective Actions) by tapping the button with the pen. An illustration of the form used to process Step 2 data collection is given in Figure 3. After all of the previously recommended corrective actions have been evaluated, Step 3 (Form D) data collection is initiated.

TABLE 1 NCR Definitions by Category

STRUCTURAL		NON-STRUCTURAL	
NCR	Description	NCR	Description
N	Not Applicable	N	Not Applicable
9	Excellent Condition	4	Good Condition
8	Very Good Condition	3	Fair Condition
7	Good Condition	2	Marginal Condition
6	Satisfactory Condition	1	Poor Condition
5	Fair Condition	-	Not Defined
4	Poor Condition	-	Not Defined
3	Serious Condition	-	Not Defined
2	Critical Condition	-	Not Defined
1	"Imminent" Failure Condition	-	Not Defined
0	Failed Condition	-	Not Defined



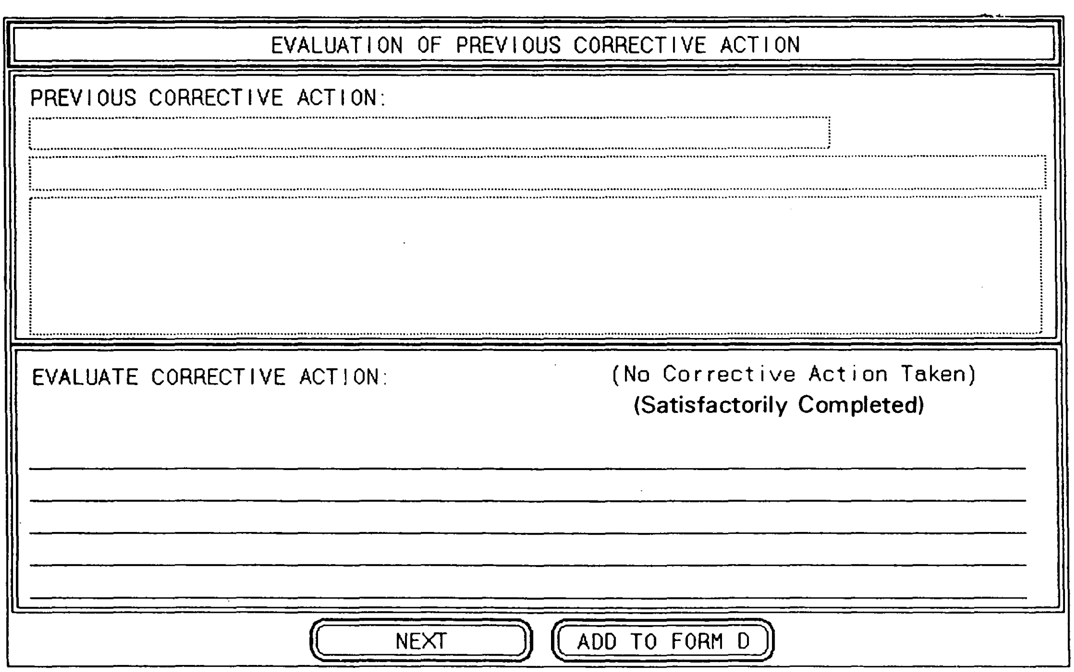
SUPERSTRUCTURE COMPONENTS			
BMIS NO.	ELEMENT TITLE	NEW NCR	OLD NCR
G12.00	SUPERSTRUCTURE OVERALL		
G12.01	BEAMS/STRINGERS/GIRDERS		
G12.02	FLOOR BEAMS		
G12.03	MAIN GIRDERS		
G12.04	SWAY BRACING		
G12.05	LATERAL BRACING		
G12.06	UPPER CHORD		
G12.07	LOWER CHORD		
G12.08	VERTICALS		
G12.09	PORTALS		
G12.10	MISCELLANEOUS MEMBERS		
G12.11	COUNTERWEIGHT		

DECK
SUBSTRUCTURE
MECHANICAL
ELECTRICAL
GENERAL
PAINT SYSTEM
CONTINUE INSPECTION
END INSPECTION
VIEW COMMENTS
EDIT GRAPHICAL DATA

FIGURE 2 Example form Superstructure used in Step 1 data collection.

Step 3 data collection processes the BMIS elements denoted as deficient in Form B. Forms are used to display each deficient element sequentially, and the inspector is given the choice of recommending or not recommending corrective action or viewing the new comment to reassess the deficiency. Figures 4 and 5 illustrate the forms used in Step 3 data collection. If the inspector selects "YES" to recommend corrective actions, the element is displayed in the form shown in Figure 5. The inspector then assigns an activity code from a specific list pertaining to the BMIS element and then estimates the number of units, such as square feet or pounds of steel. After all deficient elements have been processed, the acquisition program closes all corresponding data files, the fields are reinitiated, and the inspector is given the opportunity to quit or perform another bridge inspection.

The data obtained by the field data acquisition programs are stored as files in the notebook computer until the inspector returns to the office. Office procedures will then be performed on the desktop computer and its peripherals.



EVALUATION OF PREVIOUS CORRECTIVE ACTION

PREVIOUS CORRECTIVE ACTION:

EVALUATE CORRECTIVE ACTION: (No Corrective Action Taken)
(Satisfactorily Completed)

NEXT
ADD TO FORM D

FIGURE 3 Example form used in Step 2 data collection.

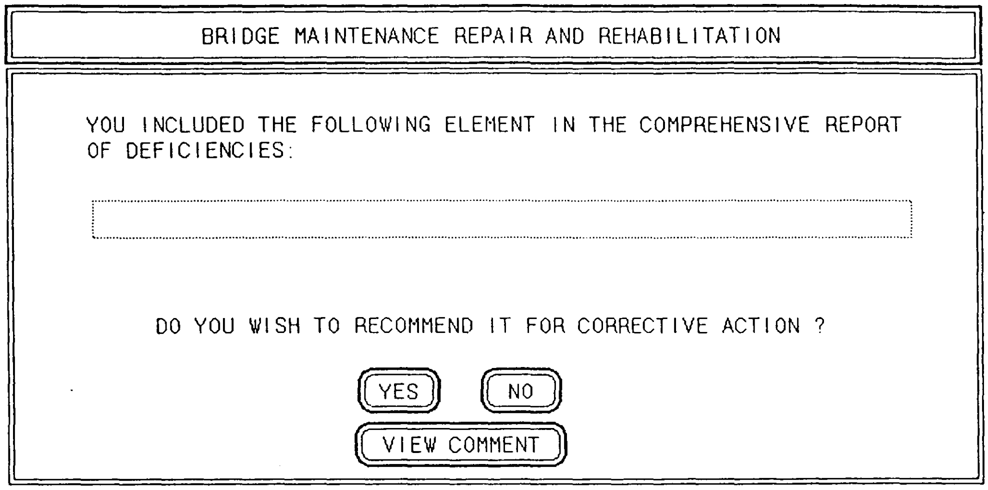


FIGURE 4 Example form used to denote service required in Step 3 data collection.

OFFICE REPORT PROCEDURES

The office procedures are performed from a desktop computer with the office system. The programs in the office system were developed with Microsoft BASIC Version 7.0 (*3*), Jetform's JFDESIGN (*4*) for form design, and FILLERG (*5*) for form editing and printing of inspection forms. The three main functions of the office system are to (a) upload, download, edit, and print inspection reports, (b) process graphical (image) data such as photographs and video, and (c) process work orders from bridge inspection data. There is also a menu system to control the execution in each function. The three menu names are FDOT Main Menu, Image Editing Menu, and Work Order Processing Menu. The FDOT Main Menu is the main control program from which the other menus are accessed. System control is always passed back to the FDOT Main Menu.

FDOT Main Menu

All office functions are performed or accessed from the FDOT Main Menu. Figure 6 displays the flow chart for the FDOT Main Menu.

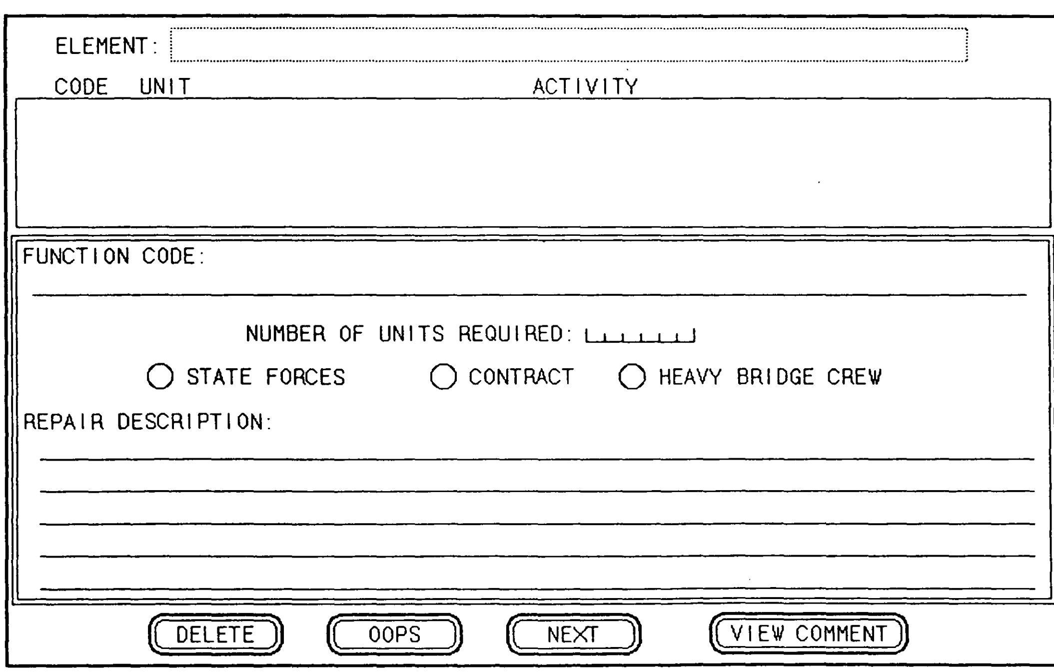


FIGURE 5 Example form used to assign corrective actions in Step 3 data collection.

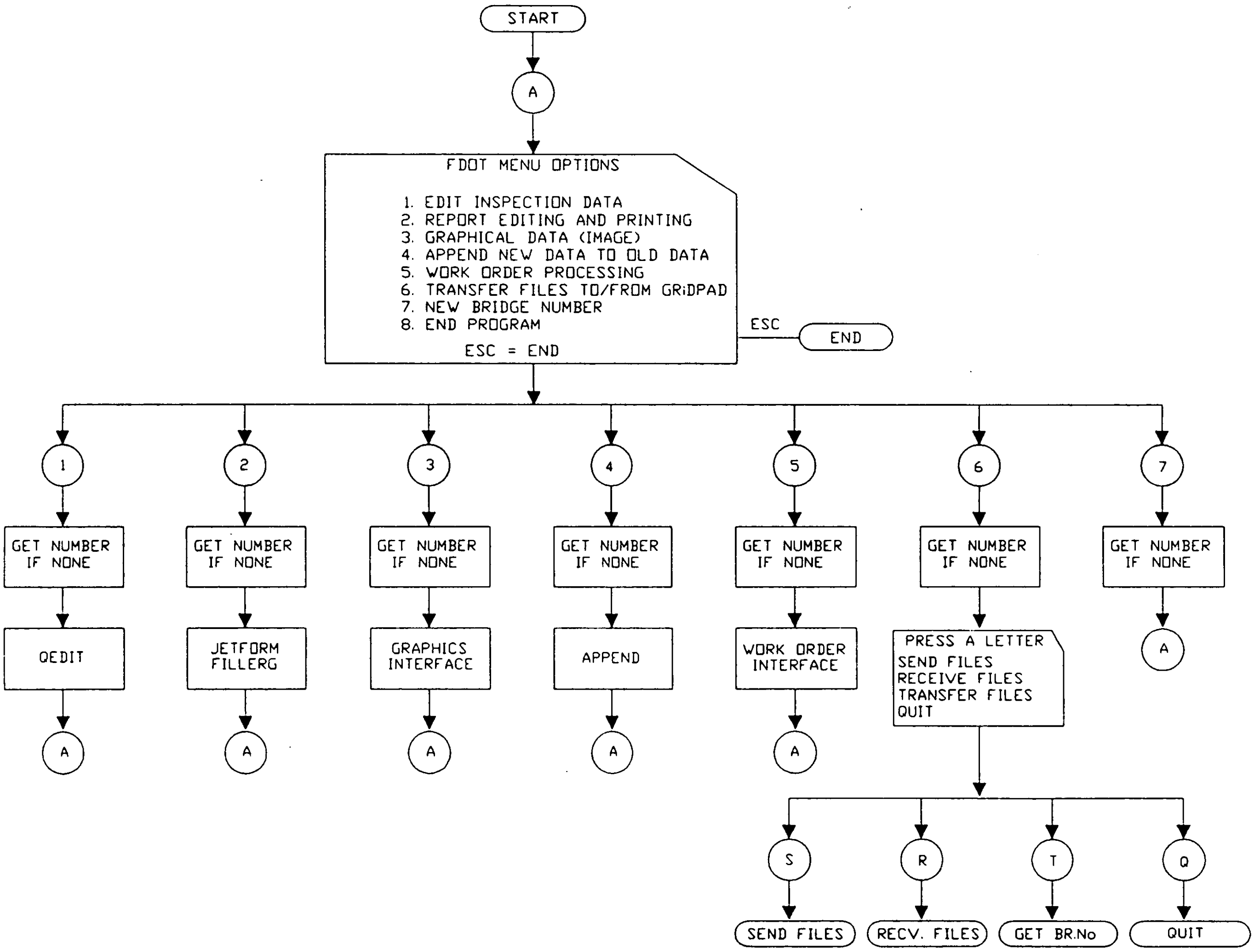


FIGURE 6 Flow chart for FDOT Main Menu.

Data Transfer

Option 6 of the FDOT Main Menu is used to perform data transfers. Data transfers consist of downloading the previous (old) inspection reports to the field notebook computer or uploading new inspection reports to the desktop office computer from the field computer. Field data are stored in a series of delimited files that must be recompiled into Jetform files for processing of the inspection report. The conversion of the delimited files to Jetform files and Jetform files back to delimited files takes place during the transfer and is accomplished with the use of two programs that were developed. The programs that were developed provide the ability to combine multistructure inspections into one standard report. Multistructure inspections occur when a bridge comprises a fixed and a movable span. If these spans are inspected concurrently they must be assimilated into one standard report. The bridge number is used to designate which files are to be transferred.

Text Editing, Inspection Report Form Filling, and Printing

The FDOT Main Menu provides two options for editing data. Option 1, "Edit Inspection Data," in the Main Menu is QEdit (*6*). When this option is selected the raw data used in the inspection report are displayed as ASCII text and can be edited. This option provides a quick method for modifying inspection data and should only be used by inspectors with text editor experience. Option 2 of the Main Menu is "Report Editing and Printing." This option provides a full-screen what you see is what you get (WYSIWYG) environment, using Jetform's FILLERG application. The selected bridge's inspection report is displayed exactly as it will be printed. From Option 1 the inspector can edit and print the final inspection report.

Image Editing Menu

The Image Editing Menu is used to coordinate the use of the developed programs with the commercial programs to perform functions such as digitizing, annotating, and printing images of deficient bridge elements. Two DOS batch programs are used along with the menu that was developed in Microsoft BASIC 7.0. Figure 7 presents the flow chart for the Image Editing Menu. This menu is accessed from and returns control back to the FDOT Main Menu.

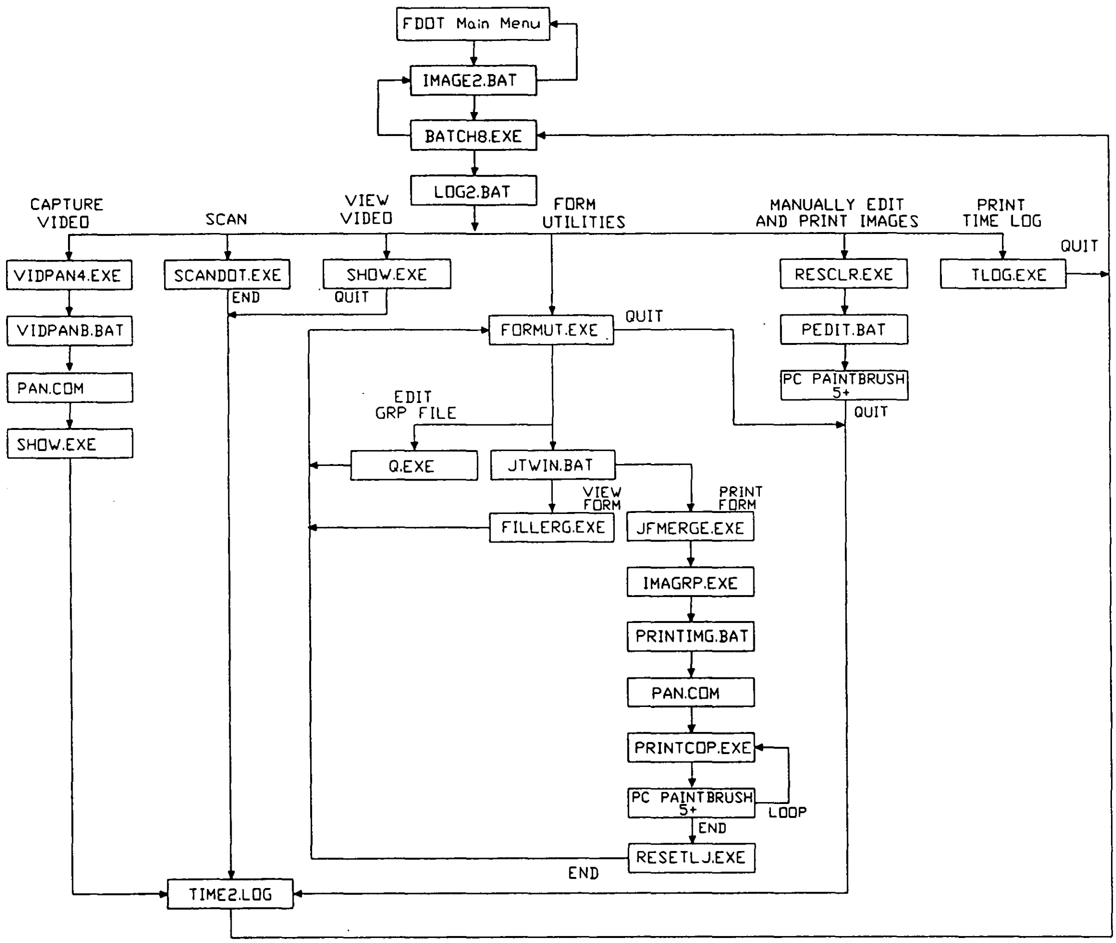


FIGURE 7 Flow chart for Image Editing Menu.

Capturing and Converting Video Images

Images recorded on video are captured and converted to a digital format with automated software that controls the capture process and a Jovian Super VIA Video Input Adapter. The captured images are stored as .PCX files with 640-by-480 color resolution. These images can be manipulated, displayed, and printed with the inspection report from the Image Editing Menu.

Scanning Photos and Sketches

The scanning process is automated with the developed program called ScanDOT. ScanDOT reads graphical text stored with the inspection files and sequentially displays descriptions for each image to be scanned. The inspector places the corresponding picture or sketch on the scanner and then invokes the scan by pressing the ''S'' key.

Video Data Base of Structures

The images stored in the desktop computer can be displayed on a VGA monitor by using SHOW.EXE, which is a utility program provided with the Jovian Super VIA Adapter. When the Video Data Base of Structures option is selected from the Image Editing Menu, the inspector can view all captured images for a specified bridge.

Image Form Utilities

The Image Form Utilities selection in the Image Editing Menu provides the ability to edit and print graphical images and annotative text. The first option in the utilities menu is ''View Image Forms,'' which allows viewing of image forms as they will actually be printed with FILLERG. The second option, ''Edit Image Data,'' facilitates the editing of the graphical text that pertains to the images (annotation). The third option, ''Print Image Forms,'' prints out the images with text on the designated forms.

Manually Editing and Printing Images

The Manually Editing and Printing Images option automatically loads Zsoft's PC Paintbrush Plus (7), which is used to manually edit and print photographs, sketches, and video images. The program has many sophisticated image enhancement functions, if required, and is easy to use.

Work Order Processing Menu

During the development of the system it became increasingly more important to incorporate an automated work order system. FDOT officials mandated the development of a computer-automated work order system following a tragic incident that apparently involved a deficient element that was discovered during bridge inspection but that was not communicated effectively to maintenance personnel. In response to this mandate the Work Order Processing Menu was developed.

The Work Order Processing Menu is accessed by the FDOT Main Menu. When the inspector completes all work order tasks program control is returned to the FDOT Main Menu. The Work Order Menu was written in Microsoft BASIC 7.0. The functions available on the menu are Generate Work Orders, Edit and Print Work Orders, Desktop to Mainframe Format, Mainframe to Desktop Format, Select a Different Bridge, and Exit. Deficient elements requiring corrective actions are quickly communicated to maintenance personnel by using this system. Once the work orders are uploaded to the mainframe they are distributed to the proper maintenance personnel via computer communications in a timely manner, and prompt responses are mandated. Therefore management is assisted with scheduling the time of delivery of resources and tracking supplies. Efficient communication is essential in work order processing to prevent time delays for service. These delays can cause tragic incidents because of unrepaired bridge elements. Figure 8 presents the flow chart for the Work Order Processing Menu.

Generate Work Orders

Generate Work Orders is used to create a work order for every deficient element requiring corrective action. Inspection reports provide the initial input for creating work orders. The key work order fields used for processing are set to a default value if the input does not exist in the inspection report.

Editing and Printing Work Orders

When work order generation is complete, the Edit/Print Work Orders option of the Work Order Processing Menu is used for editing, printing, or both. Selection of this option invokes Jetform's FILLERG, which fills the work order forms with the generated data and provides editing and printing capabilities.

Desktop to Mainframe Format

While on the desktop computer work orders are in a special Jetform data format that uses specific field identifiers. This data arrangement conflicts with the mainframe input processing requirements, which necessitate the use of ASCII files with field delimiters. The Desktop to Mainframe Format selection is used to compile work order data into a format suitable for mainframe processing.

Mainframe to Desktop Format

After work orders have been processed on the mainframe, the Mainframe to Desktop Format option is used to extract information from the mainframe-delimited file and arrange it in the special Jetform data format. After the updated work orders are compiled into the Jetform format they can be edited and printed from the desktop computer exactly as performed after generating the work orders initially.

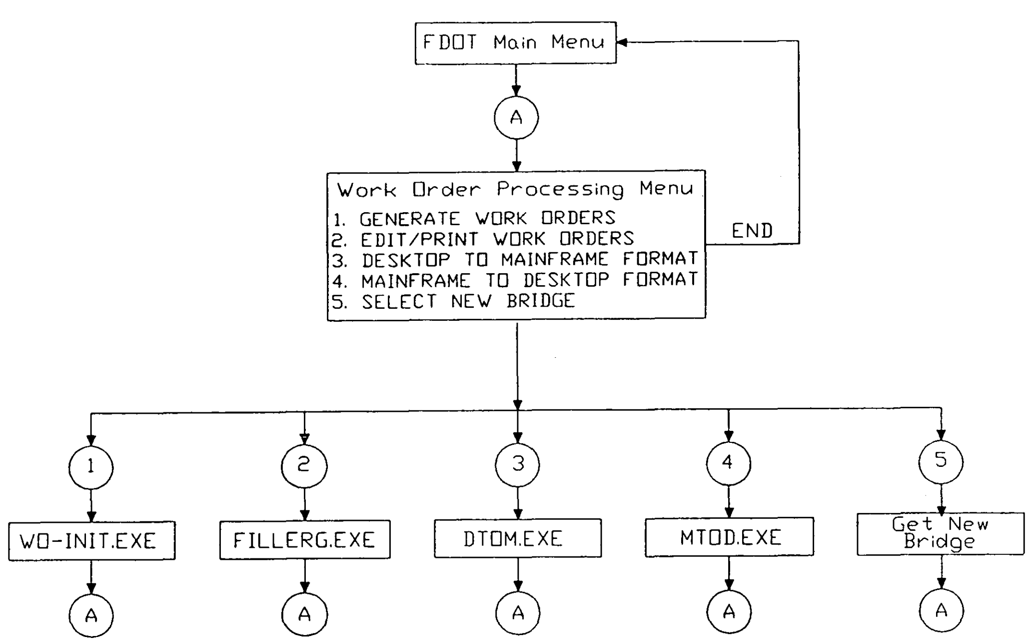


FIGURE 8 Flow chart for Work Order Processing Menu.

RESULTS AND CONCLUSIONS

The computer-automated bridge inspection process has been tested and implemented in several FDOT districts. The time spent on field procedures was not significantly different from that by the conventional method; however, a substantial reduction in the time required for office procedures was realized. A typical final inspection report can be produced in 10 to 30 min by the automated procedure. The conventional procedure required the inspectors to rewrite and rearrange field notes in a manner suitable for report compilation. The conventional method of producing a final report can take from 45 min to several weeks. The significant time savings realized by using the computer-automated system in the office enables inspectors to devote more time to performing field inspections to ensure the safety of every bridge component.

The benefits of using the computer-automated system include but are not limited to the following:

1. Thorough and accurate field inspections are performed.
2. Inspection reports are efficiently produced with a consistent format.
3. Inspection data are stored efficiently as computer disk files.
4. Deficient bridge elements requiring service are recorded, and the need for service is communicated to maintenance personnel in a timely manner.
5. Work order data are accessible on the FDOT's BMIS and Maintenance Management System.
6. Inspection reports can be transferred between offices electronically.

ACKNOWLEDGMENTS

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Span-Based Network Characterization for Bridge Management

DIMITRI A. GRIVAS, B. CAMERON SCHULTZ, DAVID J. ELWELL, AND ANTHONY E. DALTO

A methodology for a span-based approach to bridge network characterization is presented. Spans are organized and evaluated by using five distinguishing factors, namely, size, type of service, continuity, superstructure material, and design type. Emphasis is placed on making the methodology useful for a wide range of bridge management tasks. Network characterization is pursued through the aggregation of spans with similar characteristics into "span families." The historical trends of various condition measures are evaluated for bridges, spans, and families of spans, and preliminary network-level trends are investigated. The methodology is applied for the case of the New York State Thruway bridge network. It is concluded that the decomposition of bridges into their constituent spans and the classification of spans into families provide a viable approach to the development of detailed condition prediction models.

The New York State Thruway Authority (NYSTA) and Rensselaer Polytechnic Institute (RPI) are cooperating to develop a bridge management system (BMS) for the authority's inventory of 808 bridges. The computerized system will unite expert knowledge on deterioration processes and maintenance practices with available data and analytical methods to aid in bridge preservation decision making (*1*). The findings of the present study provide a framework for the more detailed development of the network-level analysis components of the authority's BMS.

OBJECTIVES

It is the broad objective of the present study to develop a detailed approach to bridge network characterization. Specific objectives are to

- Develop an appropriate grouping of spans for use in network-level analysis, and
- Investigate historical network-level condition trends for bridges, spans, and families of spans.

Early products of this effort also serve to (a) establish the current status of the condition of the Thruway bridge network, (b) provide insight into the historical rate at which network condition has changed, and (c) provide input for the evaluation of the effectiveness of maintenance program initiatives.

BACKGROUND

The majority of Thruway bridges were built between 1954 and 1960. A diversity of sizes, designs, and materials exists. The most common type of structure is the simply supported composite I-beam, although many other types—such as built-up girders, continuous I-beams, trusses, box culverts, concrete frames, and concrete arches—are represented. Many bridges are composed of spans of more than one type. Across the network span lengths vary significantly, ranging from 6.1 m (20 ft) for ordinary box culverts or frames to 369.4 m (1,212 ft) for the main truss of the 4.8-km (3-mi) Tappan Zee Bridge.

The condition of each bridge is assessed at least biennially in accordance with the procedures developed by the New York State Department of Transportation, consistent with the National Bridge Inspection Standards (*2*). Inventory and inspection data, including element ratings and general recommendations for the bridge and selected components, are available for 1978 through 1992. For the purposes of the present study, the most recent inspection records for each bridge and span are used to represent the condition of the structure for a given year. This practice ensures that the entire bridge population is represented in the statistics calculated for each year.

In essence the 56 types of inspection ratings recorded during each inspection represent discrete assessments of aspects of bridge condition at different points in time. Although such information is useful for project-level analysis, condition must also be summarized to support network-level activities. This synthesis has typically been pursued through a weighted-average calculation methodology that combines the minimum ratings of 13 significant elements into a bridge condition rating (BCR) index (*3*). For the purposes of the present study a span condition rating (SCR) index was calculated by applying the same methodology to individual spans rather than to entire bridges.

SPAN CLASSIFICATION

Factors

The identification of factors suitable for classifying spans was achieved through extensive interaction between RPI and NYSTA personnel. In addition to the information obtained from studies documented in the literature, Thruway knowledge and experience were solicited through a series of questionnaires and discussion group meetings. Potential factors were screened with respect to (a) their influence on the rate of condition rating change, (b) their relevance to current maintenance decision-making practices, (c) the ability to quantify factors with the available data, and (d) interactions between factors. This screening process reduced the list of factors of interest to size, type of service, continuity, su-

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perstructure material, and design type. The categories applicable to each factor are defined as presented in Table 1.

Families

Each span in the network is classified with respect to each of the five factors and is assigned to a ''family'' of spans with the same characteristics. There are 12,000 possible families defined by the combinations of categories listed in Table 1. However, the 3,372 spans in the Thruway inventory fall into only 116 families. Table 2 gives the 12 most populated families of Thruway bridges.

NETWORK CHARACTERIZATION

A meaningful characterization of bridge network condition is an important requirement for the development of optimization and other network-level analysis methodologies. Currently, the authority periodically reviews network condition to evaluate the efficacy of maintenance rehabilitation programs. For the purposes of the present study network characterization is pursued by dividing bridges into their component spans and then grouping individual spans with similar characteristics into span families. The historical trends of condition for various span families are examined, and the performances of the families are compared and contrasted with that of the network average. To enable evaluation of average condition trends from year to year, condition measures are determined for every bridge and span for every year of the analysis by using the most recent inspection records, which ensures that the entire bridge population is represented in the statistics for each year.

Figure 1 illustrates that historical average network condition trends are substantially similar regardless of whether the network is considered to be a population of 808 bridges (condition measured by BCR) or a population of 3,372 spans (condition mea-

TABLE 1 Summary of Factors and Categories for Defining Span Families

FACTOR	CATEGORIES		NUMBER OF SPANS
Size	Small		796
	Medium		1596
	Intermediate		385
	Tappan Zee		198
	Castleton		68
	Grand Island #1		31
	Grand Island #2	Large*	30
	Grand Island #3		33
	Grand Island #4		33
	Niagara Viaduct		202
Service Carried	Thruway (interstate)		1761
	Overpass highway		1525
	Railroad		42
	Pedestrian		44
	Other		0
Superstructure Material	Steel		3084
	Concrete		260
	Prestressed concrete		24
	Other		4
Continuity	Simple		2478
	Continuous		524
	Cantilever/suspended		250
	Other		120
Design Type	Arch		17
	Box		1
	Box culvert		100
	Frame		27
	I beam		26
	Moveable		1
	Orthotropic		3
	Pipe culvert		2
	Plate girder		498
	Rolled beam		2466
	Slab		54
	Tee beam		7
	Truss		103
	Tunnel		1
	Other		66

*The seven largest structures in the network are considered separately

TABLE 2 Sample NYSTA Span Families

NO. OF SPANS	MATL	SERVICE TYPE	SIZE	CONTINUITY	DESIGN TYPE
819	steel	overpass	medium	simple	rolled beam
398	steel	overpass	small	simple	rolled beam
335	steel	thruway	medium	simple	rolled beam
172	steel	thruway	large (TZ)	simple	rolled beam
127	steel	thruway	large (NV)	simple	rolled beam
122	steel	overpass	medium	contin	rolled beam
109	steel	thruway	small	simple	rolled beam
101	steel	thruway	intermed	simple	rolled beam
82	steel	thruway	intermed	simple	plate girder
61	steel	thruway	intermed	cant/susp	plate girder
57	concr	thruway	small	other	box culvert
52	steel	thruway	medium	simple	plate girder

sured by SCR). In either case average network condition generally decreases until 1989 and then begins a gradual increase. The performance behavior illustrated in Figure 1 represents the network average. However the performance of individual families may vary considerably from that of the network. In Figure 2 the average condition of all spans in the network is represented as solid squares. The condition plots for three selected families are provided as examples. It can be seen that the condition of the families code numbered 42, 72, and 105 vary significantly from the network average. In contrast the condition of family 42 closely parallels the network average. Thus network characterization by using span families provides potential insight into the behavior of portions of the network that perform considerably differently from the network as a whole, and it appears that this approach could form the basis for increasing the accuracy of network-level predictions.

ILLUSTRATIVE EXAMPLE

Observations such as those described previously indicate considerable promise for improved development and use of network-level analysis tools such as deterioration modeling and treatment needs estimation. A simplified example illustrates this point. If simple linear regression models are developed for condition as a function of time, then the expression for the entire network is

$$\overline{\mathrm{SCR}}_{\mathrm{network}} = 101.29 - 0.0484t \qquad (r^2 = 0.86) \tag{1}$$

where $\overline{\mathrm{SCR}}_{\mathrm{network}}$ is the mean span condition rating index for the network, and t is the time, in years (1990, 1991, etc.). Similarly the expression for Family 105 is

$$\overline{\mathrm{SCR}}_{\mathrm{Fam105}} = 250.36 - 0.1237t \qquad (r^2 = 0.88) \tag{2}$$

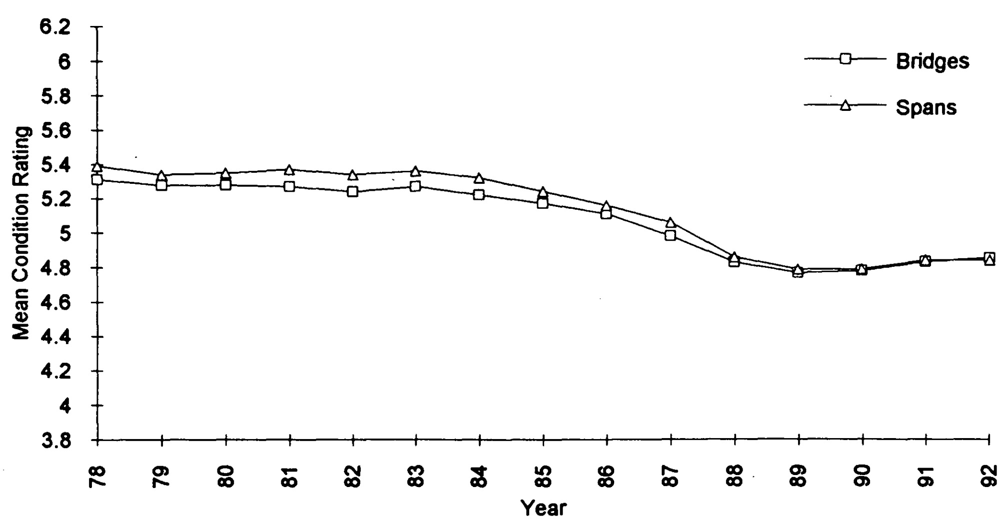

FIGURE 1 Historical average network condition trends.

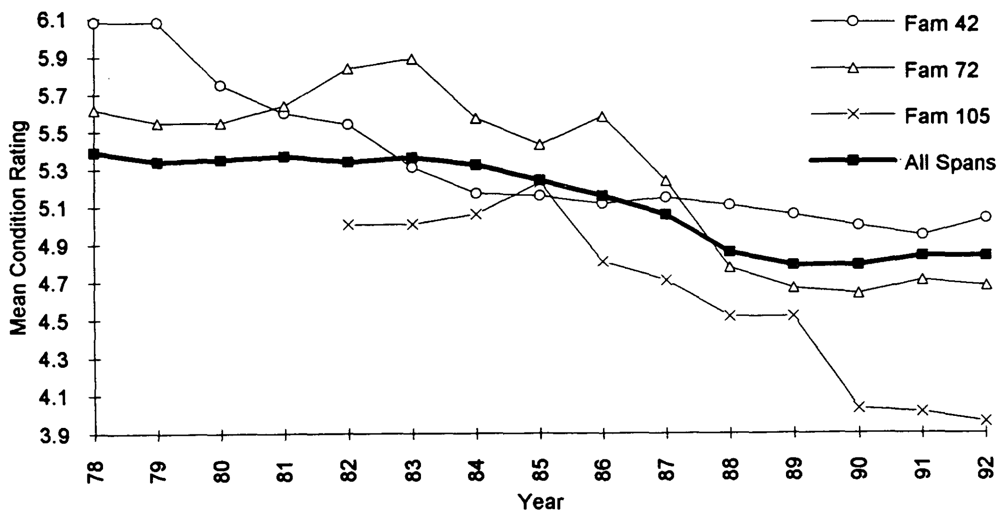

FIGURE 2 Comparison of historical average condition trends for network and selected span families.

where $\overline{SCR}_{Fam105}$ is the mean span condition rating index for Family 105.

Although Equations 1 and 2 are extremely simplistic models, the slope terms reveal that the performance of Family 105 is significantly different from the network average performance. This difference has implications for predictions of future condition, treatment needs, budgeting, and so on. For example if Family 105's condition ($\overline{SCR}_{1992} = 3.96$) is actually decreasing at a rate of 0.1237 SCR points per year and the network "average" rate of 0.0484 SCR points per year is used to make a 10-year prediction, then the predicted average condition of Family 105 for year 2002 will be 3.48, while the actual condition will be 2.72. This overestimation of condition will most likely be associated with a considerable underestimation of the required funds. Thus a span-based approach to network characterization appears to provide a promising basis for improving network-level predictions, particularly as treatment types, performance, and costs are also expected to vary among span families.

DISCUSSION OF RESULTS

The illustrative example demonstrates the potential of the span-based approach to bridge network characterization. However this approach is not a completely sufficient prerequisite for analyzing historical network condition trends or projecting future needs. It is recognized that condition trends can vary significantly among components and elements (*4,5*), and it is thus essential to further decompose span families to investigate the performance of deck, superstructure, and substructure components. Historical condition trends for these components, as well as piers, wing walls, abutments and approaches, and numerous individual elements, were analyzed previously (*4*) and provide the basis for the development of more refined network-level condition models. To support project-level analysis information is synthesized on a span-by-span basis for individual bridge structures.

This decomposition and subsequent synthesis is consistent with the NYSTA BMS aim to provide a three-dimensional view of individual structures. In contrast to the traditional two-dimensional vertical cross-section view, the three-dimensional span-by-span view tracks the performance of individual members within each span. For example the condition and maintenance history of fascia and interior girders on a given span can be distinguished. The availability of such information supports more cost-effective decision making, because materials and design characteristics can vary between spans and condition and deterioration rates can vary both between spans and within the elements on a given span. The implications of expanded data requirements to support this approach are under study.

Figures 1 and 2 compare condition trends of the network and selected span families. It should be noted that although historical average network trends are substantially similar regardless of whether the network is considered a population of bridges (BCR) or a population of spans (SCR), the SCR could be considered a less conservative measure of aggregate condition than the BCR. The SCR for each span is generated from the minimum element ratings recorded on that span. In contrast the BCR is calculated from the minimum element ratings recorded on the structure, regardless of the span on which they occur. This method of calculation produces an index representative of the combined worst-case conditions on the bridge, but not necessarily of any individual span.

SUMMARY AND CONCLUSIONS

The major thrust of the study described here represents a departure from traditional bridge management practices. Whereas typical systems address the conditions of and needs for entire bridge structures, the present effort focused on individual bridge spans to provide a framework for a three-dimensional, span-by-span approach to bridge management. Spans were organized and evaluated by using five meaningful factors that affect deterioration, namely, size, type of service, continuity, superstructure material, and design type. Network characterization was pursued through the assessment of factor levels for each span in the network and the aggregation of spans with similar characteristics into families. Historical condition trends for the network and selected span families were presented. It is concluded that the decomposition of bridges into their constituent spans and the classification of spans into families provide a viable approach to the development of detailed condition prediction models.

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The research presented in this paper was sponsored by NYSTA. It is part of a broader effort to develop and implement the authority's bridge management system. The framework of this analysis and the interpretation of the results were developed in the context of commentary provided by authority maintenance personnel. Their assistance is gratefully acknowledged.

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Behavior of Reinforced Concrete Slab Bridges During and After Repair

BAHRAM M. SHAHROOZ, RICHARD A. MILLER, V. K. SARAF, AND B. GODBOLE

Three three-span bridges with various levels of deterioration were tested before, during, and after repair to understand the behavior during repair, the possibilities for permanent moment redistribution to bottom reinforcement as a result of the repair, and the effectiveness of the repair method. The stiffness was found to be smaller while the specimens were being repaired, particularly when the deteriorated shoulders had to be cut. If the shoulder was originally ineffective because of excessive damage, the response was not changed after its removal. For the specimen with the most damage, the stiffness during variable depth removal became appreciably smaller than the prerepair value. The maximum postrepair deflection of this bridge was only 2 percent less than that of its prerepair counterpart; that is, the repair method did not enhance structural performance. The postrepair stiffnesses of the other bridges were 11 and 40 percent larger than the prerepair values, but for all three bridges the largest deflection was only 1/3,100 of the span length. While under repair a larger portion of the moment was resisted by the bottom reinforcement, but it was still less than the available moment capacities. After repair the top bars contributed more toward resisting the loads. However the repair method did not always restore the contribution of the top bars for the bridge with the most extensive damage. The ratings of the repaired bridges were increased because of (a) reduced dead loads from removal of the existing asphalt overlay, (b) increased capacity, and (c) the larger strength factors that could be used.

Continuous structures use the negative moment region over the support to lessen the positive moment in the spans. If the continuity over the spans is fully or partially lost, the result will be an increase in the positive moments in the structure. For reinforced concrete structures to remain continuous, the top reinforcing bars must remain at least partially bonded to the concrete. Reinforced concrete slab bridges may be particularly vulnerable to the loss of continuity. Most of these bridges experience noticeable levels of deterioration, which typically involves exposed top reinforcing bars (those intended to provide resistance against negative moments in the slab) and spalled or damaged concrete. A common repair technique consists of scarifying the slab concrete wearing surface by about 6 mm (0.25 in.), removing the deteriorated regions to variable depths until sound concrete is reached, and overlaying the slab with new concrete. Since a large portion of the slab depth may need to be removed before reaching the sound concrete, variable depth removal can expose many of the top reinforcing bars over most of their length if the slab is badly deteriorated. This becomes particularly critical when large areas of the slab over the piers are badly deteriorated because the concrete may need to be chipped to depths significantly below the reinforcing bars. Such operations could lead to the temporary loss of continuity between adjacent spans, which would overstress the bottom reinforcement. It is possible that the larger positive moments are greater than the strength computed by using design provisions, making the structure at least technically, if not actually, unsafe. Unless the bridge is properly shored during repairs, the structure will carry the dead and live loads as two (or more) simple spans. Even if the repair fully restores continuity, the dead-load moments of the "simple spans" will remain permanently. These higher than anticipated dead-load positive moments could overstress the bottom reinforcement in the midspan when live-load positive moments are applied after completion of the repair, and hence could reduce the rating.

A study was undertaken to examine the behaviors of multispan reinforced concrete slab bridges at various phases of repair, to evaluate the possibilities for permanent moment redistribution to bottom reinforcement and its magnitude, and to examine the effectiveness of current repair methods for restoring the stiffness and load-carrying capacity. For this purpose three bridges with various degrees of deterioration were tested in as-is condition, during repair, and subsequent to repair. The global and regional responses at different stages were measured and compared. The paper summarizes the experimental and associated analytical studies that were conducted through the course of the study.

TEST SPECIMENS

All of the bridges were three-span reinforced concrete bridges. The overall dimensions of each bridge are summarized in Table 1. Each bridge consisted of one lane and a shoulder in each direction. In all three cases the bridges were tied to end abutments only by shear keys; no other attachment was used. The interface between the approach slab and the bridge deck consisted only of an expansion joint. The significant differences between the bridges were as described below.

TABLE 1 Overall Dimensions of Test Bridges (1 ft = 0.3 m, 1 in. = 2.54 cm)

BRIDGE	SPAN LENGTHS NO. 1	SPAN LENGTHS NO. 2	SPAN LENGTHS NO. 3	WIDTH	SLAB THICKNESS	THICKNESS OF ASPHALT OVERLAY
NO. 1	29'-9"	35'-0"	29'-9"	29'-2"	14"	3" TO 4.5"
NO. 2	20'-0"	25'-0"	20'-0"	32'-6"	11.5"	2.5"
NO. 3	32'-9"	40'-0"	32'-9"	36'-0"	16.25"	ZERO

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Bridge 1

Bridge 1 was 46 years old at the time of testing. The abutments and piers were reinforced concrete walls (Figure 1). Site inspections did not reveal any visible signs of damage except for some rusted reinforcing bars that could be seen on the bottom face. Bridge 1 was tested as the specimen with ''minor damage.''

Bridge 2

Despite being slightly newer (41 years old), Bridge 2 had experienced more damage than Bridge 1. The concrete in the shoulders had deteriorated appreciably, resulting in exposed reinforcing bars, as seen in Figure 2. Each pier line consisted of five HP 12 × 53 with a pile cap. This bridge served as the specimen with ''moderate damage.''

Bridge 3

The piers in 30-year-old Bridge 3 had seven steel-encased concrete piles [406-mm (16-in.)-diameter piles] and a pile cap. Damage had occurred along one of the shoulders, and primary reinforcing bars were exposed along the edge. In addition the bridge deck had experienced significant damage over both piers covering regions of approximately 3.7 m (12 ft) × 3.4 m (11 ft) and 3.7 m (12 ft) × 5.5 m (18 ft). As seen in Figure 3 these areas had been patched with asphalt. The damage in this bridge was classified as ''extensive.''

REPAIR METHOD

The repair methods were generally similar for the three bridges. The traffic was maintained while one-half of the bridge was being repaired. Upon removal of the asphalt overlay (if existing), the bridge deck was scarified 6 mm (0.25 in.). The entire portion of the deck under repair was sounded, and the areas to be removed were outlined. The areas with unsound concrete were subsequently removed by chipping or hand dressing. If more than one-half of the perimeter of a primary reinforcing bar was exposed, the adjacent concrete was removed to a depth that provided a minimum of 19 mm (0.75-in.) of clearance around the bar. Loose but otherwise undamaged bars were supported and tied back into place, and extensively rusted or damaged bars were replaced. The deck was patched with microsilica-modified concrete. Following a sufficient curing time the deck was overlaid with a 32-mm (1.25-in.)-thick layer of microsilica-modified concrete.

FIGURE 1 Overview of Bridge 1.

FIGURE 2 Damage in shoulder of Bridge 2.

Because the bridges had experienced different levels of damage, the amount of chipping required varied. For Bridge 1 the slab near the abutments was cut through its depth, as shown in Figure 4. However the good quality of the concrete over the piers did not necessitate chipping of the deck.

Each driving lane of Bridge 2 was repaired in two stages. In Stage 1 a 0.6-m (2-ft) width of the shoulder was cut completely along the entire length of the bridge. This was necessary because the shoulders had deteriorated rather significantly. During the shoulder removal the workers were careful not to cut the bars perpendicular to the traffic lanes because this steel was used to tie the new shoulder to the slab. New longitudinal reinforcing bar was placed and a new shoulder made up of class S concrete was poured to a depth of 32 mm (1.25 in.) below the final surface. After a 7-day curing time Phase 2 of the repair, which involved variable depth removal, was started. The slab at the approach slabs

FIGURE 3 Patched regions over pier lines in deck of Bridge 3.

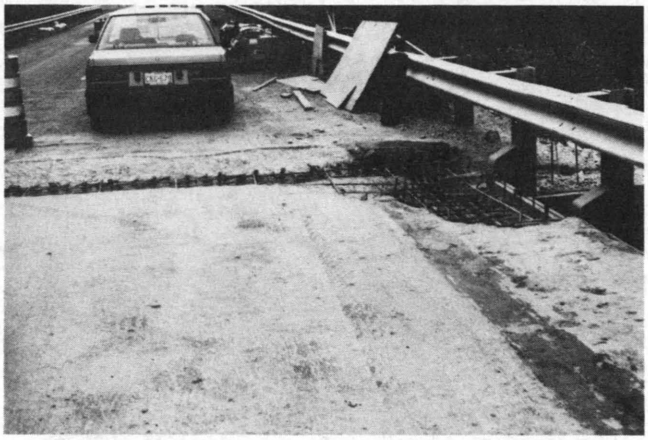

FIGURE 4 Cuts through deck of bridge 1 at abutments.

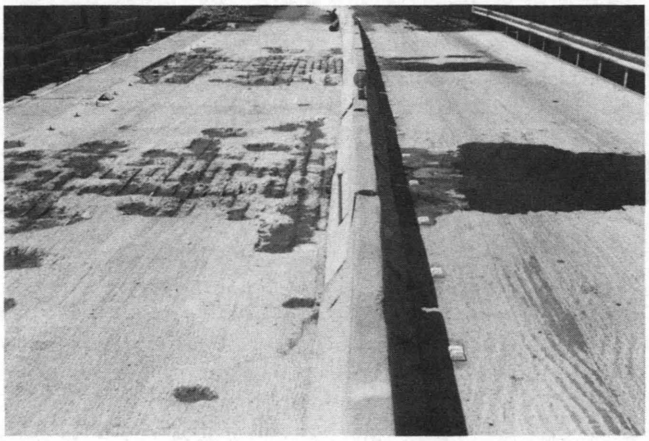

FIGURE 6 Condition of slab in Bridge 3 after variable depth removal.

was cut to a depth of 152 mm (6 in.), and the deck was chipped to a depth varying from 51 mm (2 in.) to 102 mm (4 in.). As a result many of the bars were exposed, as can be seen in Figure 5. The variable depth removal was patched with microsilica concrete, and after allowing the patches to cure the microsilica overlay was placed.

FIGURE 5 Condition of slab in Bridge 2 after variable depth removal.

Bridge 3 was also repaired in two phases in the same manner as Bridge 2, but a 0.9-m (3-ft) width of the shoulder had to be cut. The entire width of the slab (under repair) over the piers was chipped to a depth that varied between 76 mm (3 in.) and 102 mm (4 in.). Figure 6 shows the extent of chipping. As with the other bridges the variable depth removal was patched with microsilica concrete, and a microsilica overlay was placed on the deck.

EXPERIMENTAL PROGRAM

Each bridge was tested before repair, during repair (both repair phases for Bridges 2 and 3), and after repair. The bridge deck deflections were measured at 20 locations across two adjacent spans by using displacement transducers: either direct current displacement transducers or a wire potentiometer. A typical layout of the instrumentation plan is shown in Figure 7. The curvature of the bridge deck was obtained by measuring concrete strains on the top and bottom surfaces at the middle of each span and on each side of the pier caps or pier walls. These locations are shown by X's in Figure 7. Either concrete strain gauges or clip gauges [both with a 102-mm (4-in.) gauge length] were used for this purpose. On the basis of the calculated curvatures the positive moments for each span and the negative moments on each side of the pier were inferred.

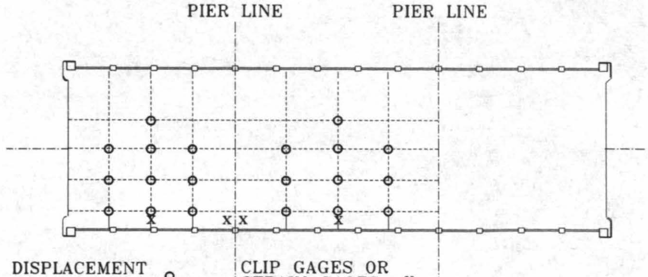

FIGURE 7 Instrumentation plan.

Four single-axle dump trucks loaded with gravel [each weighing about 14 530 kg (32,000 lb)] were used to load the bridges. The trucks were placed in three different positions to produce maximum deflection in the midspan (load case 1), maximum deflection in the end span (load case 2), and maximum negative moment at the pier (load case 3). The actual positions of the trucks were selected through finite-element analyses (*1*).

EXPERIMENTAL RESULTS

The experimental data for each specimen are presented separately. The behaviors of the bridges while under repair and the effectiveness of the repair method are discussed.

Bridge 1

Figure 8 illustrates the deflection profiles along grid line 2 [2.7 m (9 ft) from the edge] for load case 1, which caused the maximum deflection in the central span. The bridge was more flexible during repair than it was in the prerepair condition; for example, the maximum central deflection became 36 percent larger. This observation is expected by recognizing that the deck had been cut away from the approach slab and full-depth cuts through the slab had been made along the abutments (Figure 4). The largest defection anywhere on the deck occurred during repair under load case 1. This deflection corresponded to that of the central span along grid line 1 [0.9 m (3 ft) from the edge], and it was 3.4 mm (0.135 in.), or $L/3{,}100$, where L is the span length. This value is indeed very small. As seen in Figure 8 the overall stiffness of the repaired bridge was improved, leading to a 15 percent reduction in the midspan deflection along grid line 2.

The ratio of the negative moment (measured close to the pier) to the positive moment (in the middle of the end span) is compared for different stages in Figure 9. During repair, when the trucks were positioned to produce maximum moment at the pier (load case 3), the portion of the total static live-load moment resisted by the bottom reinforcement at the midspan became 58 percent larger. However the increased live-load moment was well below the available moment capacity. Truck load tests provide information only about the distribution of live loads, and it is not possible to quantify from these tests the changes that were due to dead loads. However the live loads were several times larger than the standard design loads, and the magnitudes of the live loads were close to the calculated magnitude of the dead load. The small values of the measured strains under live loads suggest that any increased dead-load moments would also be small.

Subsequent to the completion of the repair the ratio of the negative moment over the pier to the positive moment in the spans increased, which indicated that a larger portion of the total moment was being resisted by the negative reinforcement. The participation of the negative reinforcement in the end span increased by an average of 14 percent. Hence the repair method was beneficial in restoring the continuity over the piers.

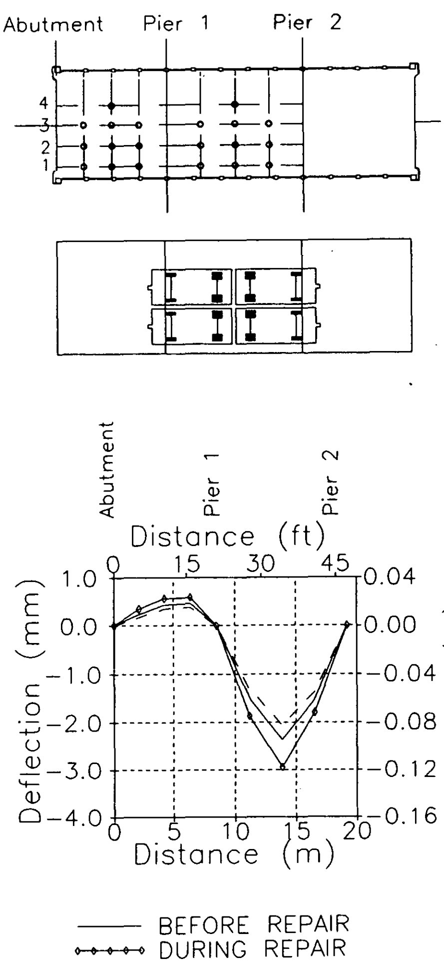

FIGURE 8 Deflection profiles for Bridge 1, grid line 2.

Bridge 2

As seen in Figure 10 the slab defection under load case 2 along grid line 1 [0.9 m (3 ft) from the edge] became larger, as expected, when the 0.6-m (2-ft)-wide potion of the slab had been cut along the entire length (phase one of repair). However the defection profile was not significantly different from that measured before removal of the shoulder. The maximum end span deflection (under

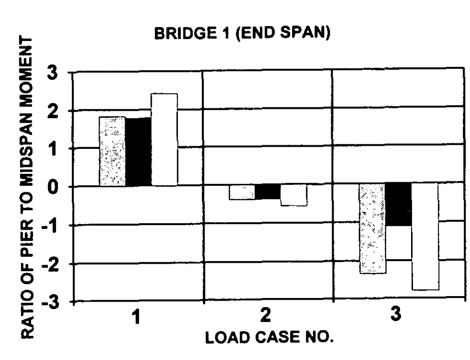

FIGURE 9 Ratio of pier to midspan moment for Bridge 1.

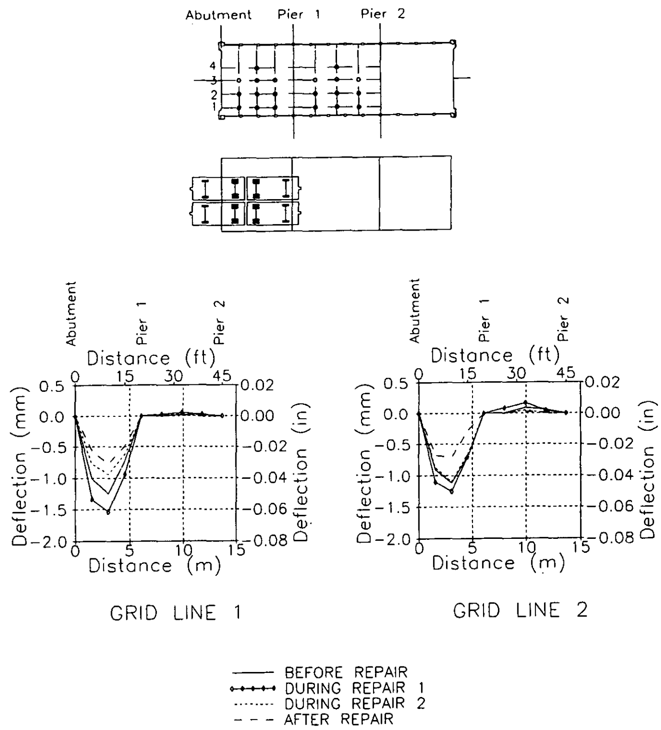

FIGURE 10 Deflection profiles for Bridge 2.

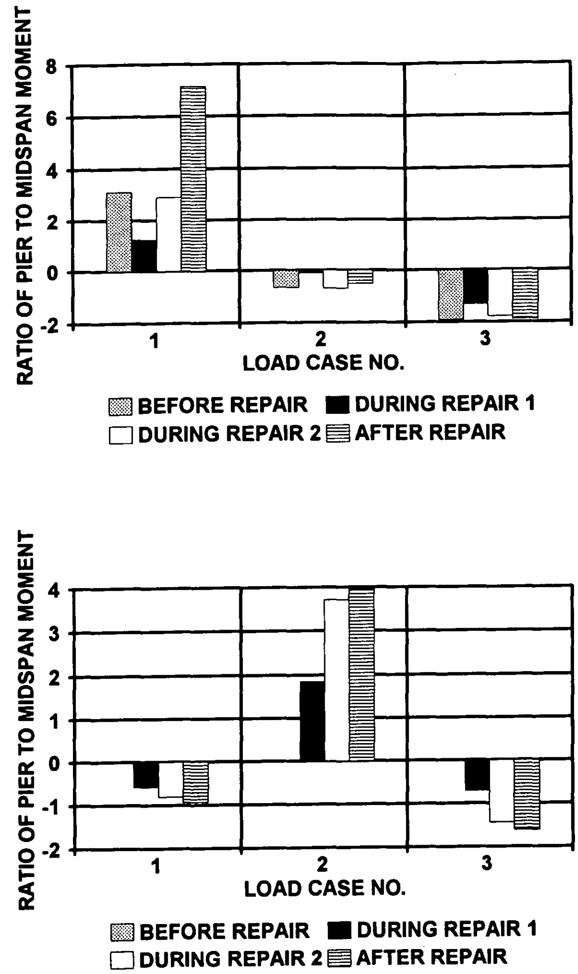

FIGURE 11 Ratio of pier to midspan moment for Bridge 2, end span (*top*) and midspan (*bottom*).

load case 2) was only 10 percent larger when the shoulder had been cut. The deteriorated shoulder in this bridge (Figure 2) did not contribute to the original prerepair stiffness, and its removal did not apparently alter the response. After a 7-day curing of the new shoulder, Phase 2 of repair involved chipping of the deck locally, as shown in Figure 5. The deflections for this phase were much smaller than those measured before repair. When the deteriorated shoulder was replaced, the stiffness must have been improved sufficiently so that the response was not adversely affected by local variable depth removal of the slab. For grid lines farther away from the edge [grid line 2, which was 2.7 m (9 ft) from the edge], the deflection profiles did not change between various stages of repair, and the effects of removing and replacing the deteriorated shoulder were not pronounced. Upon completion of repair the bridge exhibited a substantially larger stiffness. The maximum deflection of the repaired bridge was reduced by 40 percent.

An examination of the ratio of pier to midspan moment shown in Figure 11 supports the preceding observations. (Before repair, instruments for monitoring concrete surface strains in the middle of the central span were not installed. Hence the ratio of pier to midspan moment could not be computed for this span.) During the first phase of repair a large redistribution of the applied moment to the positive reinforcement was apparent. The participation of the bottom reinforcement under load case 1 was approximately 60 percent larger than the value before repair, but it was still smaller than the available positive moment capacity. For Phase 2 the ratios were not appreciably different from those measured before repair. After pouring the new shoulder the behavior was no longer influenced by chipping of the deck. This observation is consistent with the measured deflection profiles. At the conclusion of the repair a larger portion of the total applied moment could be resisted by the negative reinforcement at the piers; that is, the bridge acted more as a continuous system. The negative reinforcing steel in the end span participated 16 percent more than the prerepair value, and the top reinforcing bars in the central span resisted 35 percent more moment than during phase one of repair. The repair method was successful insofar as it improved the stiffness and increased the participation of the negative reinforcement at the piers are concerned.

Bridge 3

The deflection profiles under load case 3 (to produce maximum moment at the pier) are shown in Figure 12. When a 0.9-m (3-ft) width of the shoulder had been cut (phase one) the bridge experienced 40 percent larger deflection in the central span along grid line 2 [1.2 m (4 ft) from the edge]. In contrast for the same phase the maximum deflection for Bridge 2 was increased only by 20 percent (Figure 10). Unlike Bridge 2 the loss of stiffness from the shoulder removal influenced the deflection profiles of Bridge 3 across the width. For example along grid line 3 [3.2 m (10.5 ft) from the edge] the maximum midspan deflection for the first phase

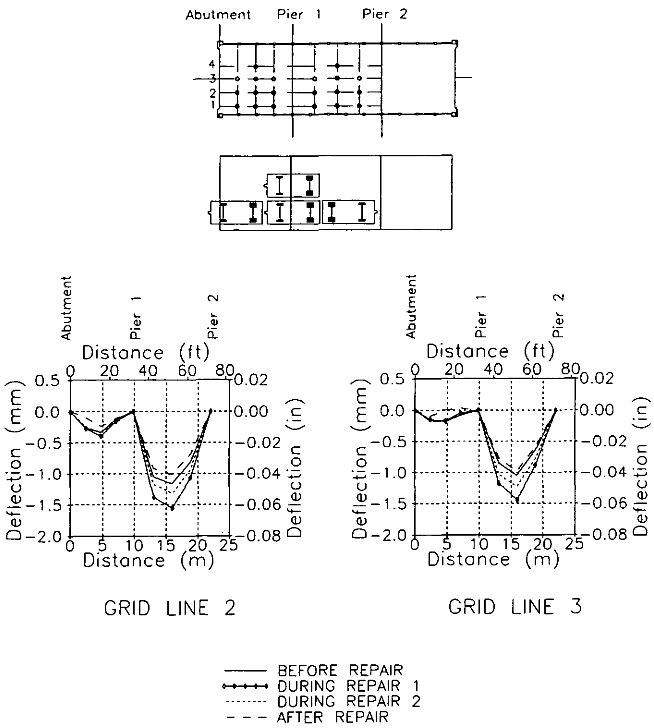

FIGURE 12 Deflection profiles for Bridge 3.

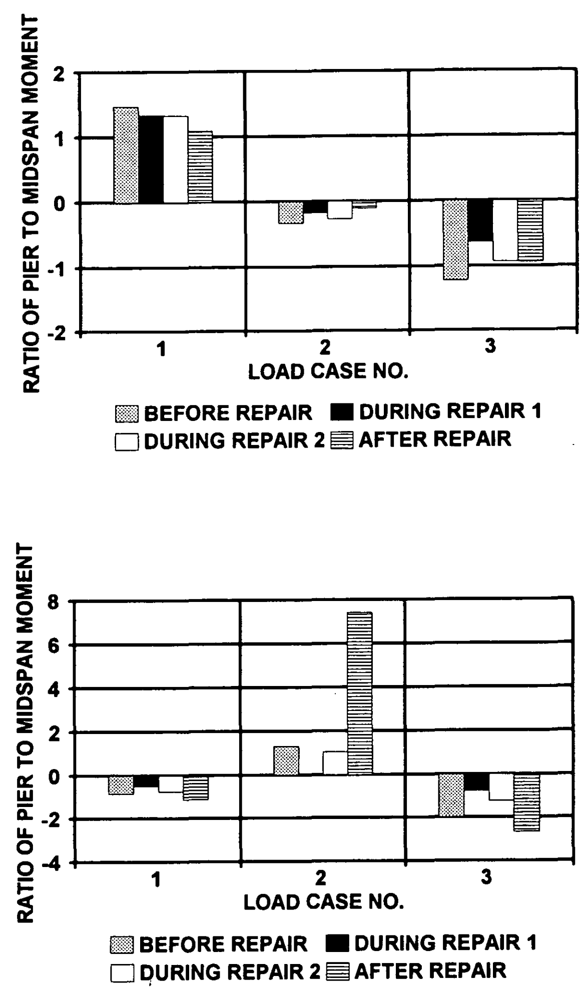

FIGURE 13 Ratio of pier to midspan moment for Bridge 3, end span (*top*) and midspan (*bottom*).

of repair became 35 percent larger (Figure 12), whereas in the case of Bridge 2 the effects of cutting the shoulder diminished rapidly away from the edge (Figure 10). These differences indicate that the shoulder in Bridge 3 was originally effective in resisting the loads, and its removal reduced the overall stiffness substantially. This is easily seen by examining the deterioration of this bridge. The deterioration over the piers was mostly confined to the driving lanes and did not really extent into the shoulders.

When a new shoulder had been poured and the deck had been chipped locally over a larger portion, mostly around and over the piers (phase two of repair), the deflections became larger than those measured before repair. The effects of local chipping of the deck were of such a magnitude that the stiffness became smaller than the original prerepair value, even though a new shoulder had been poured. Upon completion of the repair the deflections became expectedly smaller. However the maximum deflection was only 2 percent less than the prerepair value. This is in contrast to the cases for Bridges 1 and 2, for which the postrepair maximum deflections were 11 and 40 percent, respectively, less than their counterparts before repair. The repair method was apparently not as successful in restoring the stiffness of Bridge 3.

As evidenced from Figure 13 the repair procedures resulted in a reduction of moment transfer over the pier; that is, the ratio of pier to midspan moment became smaller. After the shoulder was cut, approximately 22 and 31 percent more moment was being resisted by the bottom reinforcement in the end and the middle span, respectively. Replacement of the shoulder increased the participation of the top reinforcement over the pier in resisting the total applied moment, yet it was smaller than the original level because of extensive chipping of the deck. Approximately 6 to 17 percent less moment than the original moment could be resisted by the negative reinforcement. The positive (bottom) reinforcement in the central span resisted a larger portion of the applied moment during both phases of repair. These observations are in contrast to those found for Bridge 2 (Figure 11). Although the repair method increased the contribution of the negative (top) reinforcement in the midspan by about 30 percent, the test results indicated a permanent redistribution of live-load moments to the bottom reinforcement in the end span. The contribution of the top reinforcement was about 25 percent less than the value before repair. As mentioned previously truck load tests do not permit assessment of the redistribution of dead-load moments, but the small strains that were measured suggest that the total redistributed moments owing to dead and live loads would also be less than the available moment strengths.

RATING FACTORS BEFORE AND AFTER REPAIR

By using the 1989 AASHTO guide specifications (*2*) the bridges were rated to assess the effectiveness of the repair on improving the rating factors. The dead- and live-load effects were computed either by finite-element analyses of a three-dimensional model of the bridge-pier-abutment system or by analysis of a continuous

TABLE 2 Rating Factors

Critical Case	Capacity (k-ft/ft)	Dead Load Effect (k-ft/ft)	Live Load Effect (k-ft/ft)	Rating Factor
	Bridge No. 1 (Beam Model)			
Moment over Pier Before Repair	102	20.5	13.0	3.8
Moment over Pier After Repair	102	18.7	12.9	4.3
	Bridge No. 1 (Finite Element Model)			
Moment over Pier Before Repair	102	20.6	9.6	5.2
Moment over Pier After Repair	102	17.8	9.3	6.1
	Bridge No. 2 (Beam Model)			
Moment in End Span Before Repair	46.7	4.8	9.6	2.8
Moment in End Span After Repair	54.9	4.5	9.6	3.8
	Bridge No. 2 (Finite Element Model)			
Moment over Pier Before Repair	54.1	8.1	7.6	3.8
Moment over Pier After Repair	54.1	7.2	6.5	5.2
	Bridge No. 3 (Beam Model)			
Moment over Pier Before Repair	103.4	26.7	16.2	2.8
Moment over Pier After Repair	103.4	30.7	16.2	3.2
	Bridge No. 3 (Finite Element Model)			
Moment over Pier Before Repair	103.4	25.5	12.2	3.8
Moment over Pier After Repair	103.4	29.5	12.2	4.3

beam (referred to as a *beam model*) in which the beam width was computed as described by AASHTO (*3*), that is, $E = 0.063S + 4.65$, where E is the beam width and S is clear span. The details of the analytical models are discussed elsewhere (*1*). The measured material properties were used to compute the capacities, and AASHTO guidelines (*2*) on load and resistance factors were followed. The finite-element analyses were conducted by using models that were calibrated to match the measured responses (*1*).

The ratings before and after repair are summarized in Table 2. The capacities were computed by using the provisions of code ACI 318-89 of the American Concrete Institute (*4*), in which the contribution of concrete in tension was ignored. The new microsilica overlay would not increase the negative moment (the top of the slab is in tension) capacities over the piers because the concrete above the neutral axis is assumed to be cracked and hence ineffective. However the positive moment capacities in the midspan of the repaired bridges would be larger. A combination of reduced dead-load effects upon removal of the asphalt overlay (for Bridges 1 and 2), the larger resistance factors that could be used after repair, and the increased positive moment capacity because of the new overlay (if the rating factor was controlled by the moment in the midspans) led to larger rating factors after completion of the repair. In the case of Bridge 1 the rating was controlled by the negative moment over the pier. The rating was increased between 13 percent (if the results from the beam model are used) to 17 percent (if the results from the finite-element model are used). The observed differences between ratings from the beam and finite-element models are beyond the scope of this paper and are discussed by Shahrooz et al. (*1*). The rating factors for Bridges 2 and 3 were increased by 36 and 13 percent, respectively.

SUMMARY AND CONCLUSIONS

A study was carried out in an effort to (a) understand the behavior of deteriorated reinforced concrete slab bridges during repair, (b) determine the possible adverse effects of a common repair method on moment redistribution to bottom reinforcement, and (c) examine the effectiveness of the repair method in enhancing the bridge's performance. For this purpose three bridges with various levels of deterioration (from minor to extensive) were tested in as-is condition, under repair, and after repair.

While being repaired the bridges lost stiffness, as expected, particularly when a portion of the shoulders had to be cut because of excessive deterioration. For Bridge 2 the shoulder had apparently deteriorated to an extent that its removal did not change the response from what was measured before repair. Local chipping of the deck in this bridge over the piers did not appreciably reduce the stiffness after the new shoulder had been poured. The shoulder in Bridge 3 was effective originally, and its removal reduced the overall stiffness substantially. Variable depth removal of the deck over the piers was of such a magnitude that the stiffness became smaller than the original prerepair value, even though a new shoulder had been poured. Despite the apparent loss of stiffness the largest deflection for the three specimens was $L/3{,}100$, which is considerably smaller than the maximum allowable value and is not critical. When completely repaired the bridges became stiffer, although the additional stiffness was not always significant.

Redistribution of the applied moment from the negative to positive reinforcement was evident during repair. Increases of as much as 60 percent were measured. Nevertheless the larger moments were still smaller than the available capacities. When the bridges were completely repaired a larger portion of the applied moment could be resisted by the negative reinforcement. The continuity over the pier was generally improved at the conclusion of repair for the bridges with minor to moderate damage, but some permanent redistribution of moments to the bottom reinforcement was measured for the bridge with extensive damage.

The ratings of the repaired bridges were also increased because of a combination of reduced dead loads, increased capacity, and larger strength factors. The additional rating was most significant for the bridge with moderate damage.

The repair procedure reported here can enhance the stiffnesses, moment transfers, and ratings of reinforced concrete slab bridges if they are moderately deteriorated. If the bridge is lightly or extensively damaged the repair does not improve the performance as much. Considering that the additional deflections during repair were very small and that the additional redistributed moments to the bottom reinforcement were considerably smaller than the available capacities, no changes in the current repair practice would appear to be necessary.

ACKNOWLEDGMENTS

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Polymer-Concrete Bridge Deck Overlays

MICHAEL E. DOODY AND RICK MORGAN

The results of a survey of polymer-concrete bridge deck installations (test patch and overlay) in New York and the results of a canvassing of the experiences of other states with these overlays are summarized. In New York two types of polymer overlay materials—thin epoxies and a thicker polyester—are used, with one thin epoxy used in New York State Department of Transportation Region 1 (Albany), two polyesters used in Region 10 (Long Island), and two thin epoxies used in Region 11 (New York City). In inspections during the summer of 1991 the conditions and performances of most of these overlays were found to be satisfactory.

Polymer-concrete (PC) overlays are an alternative bridge deck treatment that can be used to correct or prevent the corrosion of reinforcing steel. The objective of the study reported here was to document their performances to date in New York and other states. This paper describes the conditions of the PC overlays used in rehabilitating bridge decks in New York after various periods of service and summarizes the results of an informal survey of the experiences of other states with such overlay materials.

BACKGROUND

Bridge deck deterioration due to reinforcement steel corrosion caused by chloride infiltration continues to be a major problem for most state highway agencies. The methods currently used by New York to protect the existing steel in rehabilitation work include overlays with low-slump, silica fume, and latex-modified concrete and coating of the reinforcing steel with epoxy for new deck construction. The limitations of concrete overlays include their use in (a) situations in which the existing structure cannot adequately support the additional dead load of a concrete overlay, (b) instances in which reduced clearance cannot be tolerated, and (c) urban areas where rapid construction is essential because of heavy traffic, excessive costs for traffic control, or both.

PC overlays overcome many limitations of other types of overlay materials because of their high early compressive strength and excellent bond strength (*1,2*). These overlays are used on bridges with dead load or vertical clearance restrictions. Most interest is in urban areas because of the quick-curing, high-early-strength characteristics of PC overlays, which result in shorter times for detouring traffic and lane closures, which are extremely costly. Also attractive are their lighter weight, their flexibility, and their ability to restore skid resistance to polished decks (*3*). Thin overlays [up to 125 mm (0.5 in.)] have an additional advantage, in that modification of expansion joints or building up the approaches can be dispensed with, which can result in significant cost savings (*4*).

PC consists of a resin binder and an aggregate filler. Initially polyesters and epoxies are polymers in which the initial polymerization of the liquids is terminated at some point while they are still in the liquid phase. After the addition of an initiator they become a solid through a chemical reaction called *polymerization*. Methylmethacrylates (MMAs) are monomers that are polymerized by adding promoters and initiators. The rate of polymerization or cure depends on many factors, including temperature, humidity, and chemical additives.

There are two types of PC overlays: epoxy PC and polyester PC. Epoxy PC overlays can be MMA or epoxy concrete. Epoxy and MMA overlays have been used on bridges at thicknesses of from 6.5 to 40 mm (0.25 to 1.5 in.), and polyesters have been used on bridges at thicknesses of from 6.5 to 80 mm (0.25 to 3 in.) or more. The PC types used in New York State have included both thin epoxy and thick polyester. One of three methods of construction is typically used:

1. *Multiple layer*, which consists of two or more layers of polymer binder and gap-graded, clean, dry angular broadcast aggregate.
2. *Slurry*, which is a polymer aggregate slurry struck off with gauge rakes and covered with broadcast aggregate.
3. *Premixed*, which is a PC mixture consolidated and struck off with a vibratory screed.

Although the first two methods have been used, New York now prefers a premixed automated application.

MATERIALS

Early Work

New York has tried various PC types in overlays since 1961 (*5–8*). A wide variety have been used, with most containing epoxies or polyesters. Also tried were a few applications of polyurethanes, latexes, neoprenes, and silicone rubbers. Periodic inspections of early installations determined that surface overlays developed appreciable distress within 2 to 3 years after application. Thin overlays could not withstand exposure to the damaging effects of traffic and weather (*5*). A new generation of products was introduced in the late 1970s, but overlays once again exhibited distress in the form of debonding and cracking within 2 to 3 years of application (*6*).

Test Patch Program

Further refinements from 1980 to 1984 resulted in the highly flexible epoxies and MMAs now being used. As manufacturers continued to improve their products, New York has continued to be a site of PC testing. Three test patches were installed on the lower

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roadway of the Queensboro Bridge in May 1980 (Duracryl and Flexolith by Dural International Corp. and Silikal R7 by Transpo Materials). In September and October 1983 a test section [2800 m^2 (30,000 ft^2) of Flexolith] was placed on the Brooklyn Bridge, and test patches (Silikal Urethane Modified Acrylic Overlay by Silikal North America, Dural 317 and Flexolith by Dural International, Concresive 2020/2042 by Adhesive Engineering, T17XA by Transpo Industries, and Flexogrid by Roadway Safety Service/ Polycarb) were placed on the lower roadway of the Manhattan Bridge. In August 1985 five test patches (Transpo T17X, Dural Flexolith, Dural Coal Tar Epoxy, Dural Methyl Methacrylate, and Polycarb Flexogrid) were installed on the westbound I-90 bridge over I-787 in Albany.

The St. Lawrence Seaway Authority has a program to evaluate test patches on the Cornwall Bridge and Thousand Island Bridges over the St. Lawrence River between New York and Ontario, Canada. The products installed on the Cornwall Bridge in August and September 1991 included Nitobond (Fosroc), FX781 (Fox Industries), Sternflex, Transpo T-38 and T-48, Degadur 330, Sikadur 81-32, Flexolith, Flexogrid, and Bridge Master. Those installed on the Thousand Island Bridges included Sikadur, Flexolith, and Transpo T-45 and T-48 in September 1992, with Flexogrid, Degadur, and Bridge Master scheduled for installation in May 1993.

As these new products were developed and laboratory testing proceeded (*3,9–18*), experimental overlays were installed to relate test results to field performance. Two types of PC bridge deck overlays are now in place in New York: (a) thin PC, which uses either epoxy or MMA as a binder and which is placed in a thin layer [6.5 to 13 mm (0.25 to 0.5 in.)], and (b) blended polyester in an overlay 20 to 40 mm (0.75 to 1.5 in.) thick.

Polyester Overlays

Two overlay sites in Suffolk County on Long Island used polyester resin with basalt aggregate. An overlay consisting of 145 m^2 (15,500 ft^2) was placed on Yaphank Avenue (BIN 1064160) over the Long Island Expressway in 1982. In 1983 1125 m^2 (12,100 ft^2) was placed in another overlay near Yaphank, on east Main Street (BIN 1064180) over the Long Island Expressway. Seven additional polyester overlays of various designs are in Suffolk County near the Robert Moses Causeway (Deer Park Avenue over the Sunrise Highway, Higbie Lane over the Sunrise Highway, the Sunrise Highway over Howells Road, Fifth Avenue over the Sunrise Highway, Brook Avenue over the Sunrise Highway, and Brentwood Road over the Sunrise Highway). After the premature failure of the Brook Avenue overlay, the others were overlaid and the project was discontinued. Sealing of the decks was this project's primary objective.

Thin Epoxy Overlays

On the basis of the successful results of the thin overlay test patch program, 8305 m^2 (89,388 ft^2) of Flexolith was placed on the south upper roadway of the Queensboro Bridge under Contract D250039, with work starting in October 1984. A small area near the Manhattan anchor pier was completed in June 1985, and the bridge opened to traffic that July (*7*). In July 1985 work began on the suspended-span Manhattan-bound and Brooklyn-bound roadways of the Brooklyn Bridge. Under Contract D251251, 17 050 m^2 (183,500 ft^2) of Flexolith was placed (*8*). In July 1988, 8315 m^2 (89,500 ft^2) of Flexolith was installed on the north upper roadway of the Queensboro Bridge under Contract D500191. In October 1990 5325 m^2 (57,342 ft^2) of Transpo T17X was to be placed on the Crown Point Bridge to Vermont under Contract D253114; this work was only partially completed, with the remainder installed in September 1991. In July 1991 1215 m^2 (13,077 ft^2) of Flexolith was placed on the West 207th Street Bridge over the Harlem River (the University Heights Bridge) under Contract D500777. Polymer systems currently on the New York State Department of Transportation Materials Bureau's Approved List are manufactured by Dural (Flexolith), Transpo (T17X), and Silikal (urethane-modified acrylic overlay). All are thin overlay materials.

INVESTIGATIVE PROCEDURES

Survey of Other States

An electronic mail (e-mail) survey was conducted to determine other states' experiences with these products.

Adhesion Testing

Overlay bonding to an existing concrete surface (substrate) is an extremely important consideration in placing any overlay, because any bond deficiencies may lead to later delamination or punchout of the overlay. To test this tensile bond of the epoxy overlays, equipment was built to specifications established by the American Concrete Institute (*9*). The apparatus used for this surface adhesion test is shown in Figure 1. Tests involved partial-depth coring through the overlay into the existing slab. After cleaning and drying the overlay surface, steel plugs were epoxied to the surface of the partial-depth core. A reaction frame and calibrated load cell measured the direct force to pull the overlay from the existing substrate.

Distress Survey

The PC overlays in place in New York were visually inspected by two project engineers in August and September 1991. Overlay condition was classified as (a) good, (b) peeled because of a poor bond, (c) cracked or worn, or (d) patched. Estimates of the surface area of each type of distress were mutually agreed upon by the two engineers. No chain drag or other means were used to determine delaminated areas. Each type of overlay distress is shown in Figure 2.

RESULTS AND DISCUSSION OF RESULTS

Survey of Other States

Several different PCs have been used in various parts of the country on projects involving several types of polymers with various properties and methods of application (*16,17*). An informal survey to evaluate their experiences produced responses from 25 agencies (60 percent) that varied in form (fax, phone, e-mail, reports, specifications), as summarized in Table 1. Of the respondents, only 4

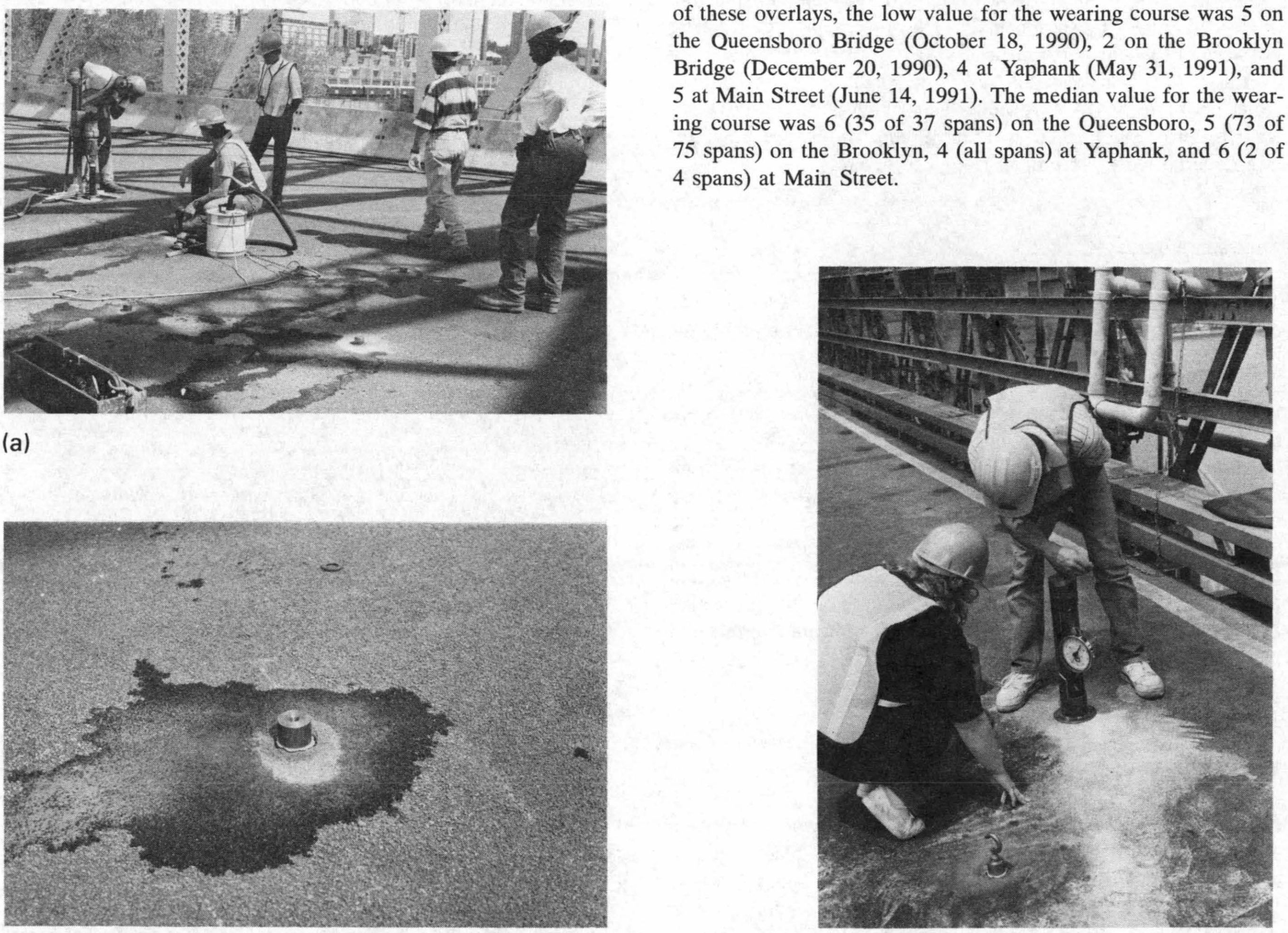
(16 percent) use PC overlays as a standard treatment and 10 (40 percent) use no PC overlays of any type. Two states—Wisconsin and Oklahoma—use no PC overlays, after experiencing failures of experimental installations. Idaho uses a polymer material (MMA) as a crack sealer but not as a deck overlay. The other states with the most experience with these materials and the most extensive programs are California and Virginia. California uses polyester routinely. Virginia is the leading user of thin epoxy overlays (about $1 million to $2 million annually since 1989), but it has discontinued use of polyester. Polyester overlays seem to be the optimum choice for concrete decks, but epoxy bonds as well to steel as to concrete, and adhesion should thus be as uniform on steel-grid decks and epoxy should be the choice for those installations.

PC Overlay Performance in New York

PC deterioration occurs in many forms because of structural deficiencies, thermal stresses, moisture, or other factors. Common forms of early deterioration are raveling or delamination and cracking, which can occur anywhere over the deck. Where the surface cracks, the potential for accelerated deterioration is present because moisture can cause the overlay to delaminate from the deck surface. Typical comments during the visual inspections included the following:

- Coating badly peeled and cracked;
- Surface worn, with abrasive material missing;
- Wear, some peeling of coating;
- Some wear of coating, worn away at patches;
- Peeling at joints, some shrinkage cracks; and
- Satisfactory except for small spalls at transverse joints.

As these comments illustrate, distressed areas often exhibited more than one type of distress. Such localized, ''patchy'' failures with multiple distress types are probably related to construction practices, with material failures likely to be more uniform across the deck. The visual distress survey is summarized in Table 2. Overlay construction was observed on the Crown Point and University Heights bridges, but distress was not surveyed because they had not been opened to traffic. In the most recent inspections of these overlays, the low value for the wearing course was 5 on the Queensboro Bridge (October 18, 1990), 2 on the Brooklyn Bridge (December 20, 1990), 4 at Yaphank (May 31, 1991), and 5 at Main Street (June 14, 1991). The median value for the wearing course was 6 (35 of 37 spans) on the Queensboro, 5 (73 of 75 spans) on the Brooklyn, 4 (all spans) at Yaphank, and 6 (2 of 4 spans) at Main Street.



(a) (b) (c)

FIGURE 1 Adhesion testing included (*a*) partial-depth coring through overlay to underlying slab, (*b*) epoxying a steel plug to core, and (*c*) usng a reaction frame and load cell.

Queensboro Bridge, South Upper Roadway

The delamination and patching that occurred on one deck section are attributable to malfunction of the contractor's automated mixing equipment and are not included in the distress analysis because they are not a materials problem.

Queensboro Bridge, North Upper Roadway

The section is performing well.

Brooklyn Bridge

Cracking was observed at the roadway relief joints and was found to result from structural inadequacies of the floor system at the roadway joint, but not from any inherent deficiencies of the Flexolith (*8*). These cracks have since been patched (Figure 3) and also are not included in the distress analysis since they are not a materials problem.

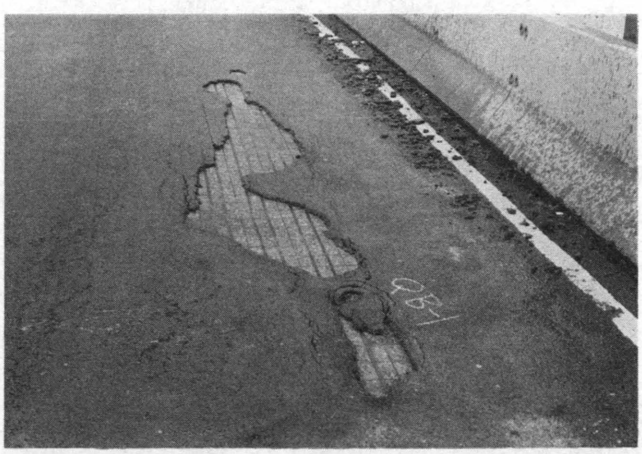
Crown Point Bridge

The PC wearing course was not well suited to this bridge. The product on the Approved List was obsolete, and the manufacturer had to prepare a special batch for this job. Because of the bridge's steep grade and the PC's flow characteristics it tended to run, and it proved difficult to achieve a smooth riding surface. Part of the first application of the wearing course was leveled off or removed and replaced because of an unacceptable riding surface caused by the product's tendency to run. The manufacturer modified the product a number of times to try to minimize this problem.

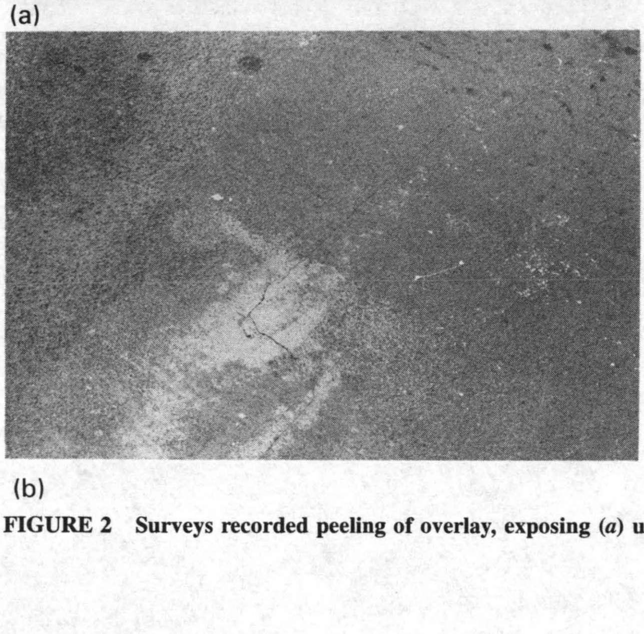
To characterize the resulting ''washboard'' effect (Figure 4), roughness was measured with a Soiltest road roughness indicator (roughometer). Measurement of pavement roughness is a primary indicator of riding quality, a general reading that translates the effect of all distress into the road user's frame of reference. Roughness caused by any factor can lead to additional deterioration by inducing more vertical movement of vehicles, producing more frequent and increasingly severe impact loads.

The roughometer ran twice on the approach section, midspan, and leave sections in both eastbound and westbound directions, and the readings were averaged to determine average roughness in millimeters per meter (inches per mile). Reading less than 3 mm/m (190 in./mi) are considered ''good.'' For comparison readings were taken on a bridge with similar geometry on Congress Street (Route 2) over the Hudson River in Troy (BIN 1004279). Average roughness on the Crown Point Bridge was 2.21 mm/m (140 in./mi) eastbound and 2.12 mm/m (134 in./mi) westbound. Average roughness on the Route 2 bridge was 1.72 mm/m (109 in./mi) eastbound and 1.88 mm/m (119 in./mi) westbound. Both surfaces are performing adequately. In the most recent inspections of these overlays the wearing course low value was 6 on the Route 2 bridge (September 21, 1990) and 6 at Crown Point (January 21, 1992). The wearing course median value was 6 (all spans) on Route 2 and 6 (all spans) at Crown Point. On the basis of these measurements the performance at Crown Point is comparable after only 1 year of service to that at Route 2 after 7 years of service.

After 10 months the initial partial installation was in good condition except for transverse cracking (Figure 5). Its cause is unclear, but it may be occurring because of flexing of the deck.



(a)



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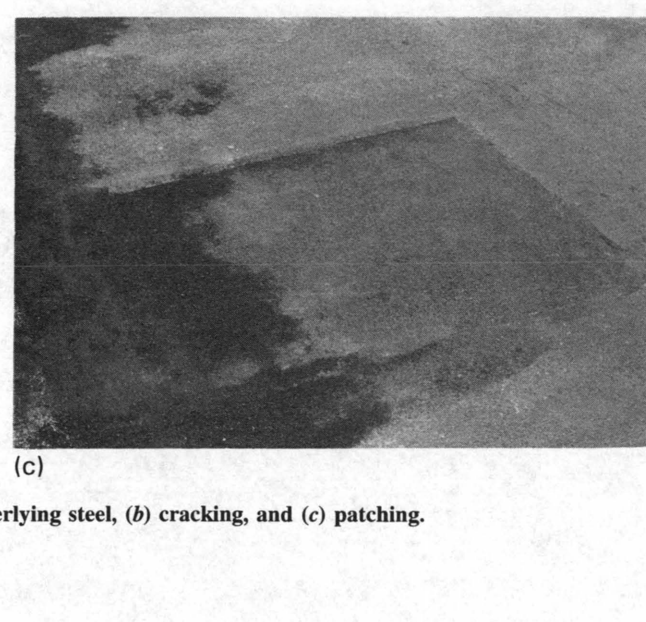

(c)

FIGURE 2 Surveys recorded peeling of overlay, exposing (*a*) underlying steel, (*b*) cracking, and (*c*) patching.

TABLE 1 Responses to E-Mail Survey (*19*)

State	PC Used?	PC Type Used	Remarks
Alabama	Provisionally (4 yr)	Polyester	"No problems"
Arkansas	Not used		
California	Standard (10 yr)	Polyester	Used extensively
	Experimental	Epoxy	
Idaho	As crack sealer only	MMA	No Overlays
Illinois	Special installations	Epoxy	"Not truly impermeable"
Indiana	Standard (12 yr)	Polyester	"Success with only one thin overlay"
Kansas	Not used		
Kentucky	Not used		
Louisiana	Experimental	Epoxy	Being evaluated
Minnesota	Not used		
Missouri	Experimental (2 yr)	Epoxy	30± bridges
Nebraska	Not used		
Nevada	Standard (3 yr)	Polyester	"No problems"
New Mexico	Not used		
N. Carolina	Experimental	Epoxy	1 installation
N. Dakota	Experimental	MMA	4 years service
Oklahoma	Not used		1 experimental deck (replaced)
Pennsylvania	Experimental	Epoxy	3 bridges
S. Dakota	Not used		
Texas	Experimental	Polyester	1 installation
Vermont	Experimental	Epoxy	3 installations
Virginia	Standard	Polyester	Used extensively
Washington	Experimental Feature	Epoxy	Since 1984
		Polyester	Since 1989
Wisconsin	Not used		2 experimental deck failures
Wyoming	Experimental	MMA	2 installations

Results of the surface adhesion tests (*9*) on the epoxy overlays are summarized in Table 3. In addition the 2790-m^2 (30,000-ft^2) Flexolith test section on the Brooklyn Bridge has been in service for 8 years and averaged 1420 kPa (206 lb/in^2) for nine tests. Similarly the Flexolith test patch on the Manhattan Bridge has been in place for 8 years and averaged 945 kPa (137 lb/in^2) for six tests. The minimum desired bond strength for this particular adhesion test is 1725 kPa (250 lb/in^2). None of the installations with more than 3 years of service attained this value, nor did the University Heights bridge, which was not open to traffic. These low values are not reflected in the distresses reported in Table 2, but may indicate more rapid deterioration in the future.

CONCLUSIONS AND RECOMMENDATIONS

The objectives of the study reported here were to outline what is already known about PC bridge deck overlays and to document their performance in New York State and elsewhere:

1. Earlier generations of PC overlays had a poor performance record. Testing to date supports optimism for the suitability and durability of newer polymer systems, although there is no way to predict their long-term performance accurately at this time.

2. The performance of these systems is limited by the surface on which they are placed. Successful use depends on proper surface preparation. The deck to which the overlay is applied must be sound. The substrate as well as the coarse aggregate used to extend the mix must be dry and clean.

4. New York, Virginia, and California have the most experience with these materials, with generally favorable results. The e-mail survey documented mixed results from other states, which had only limited experience.

4. PC overlays in New York appear to meet expectations, showing good performance during their first 5 to 7 years. The principal long-term concerns to be resolved are whether they will retain an adequate bond to concrete and resist wear where traffic volumes remain high. Because of the variability of field conditions, lab screening tests may be poor indicators of performance when the

TABLE 2 Surface Overlay Condition

	Structure and % of Surface Failed				
	Queensboro				
Distress	North Upper Roadway	South Upper Roadway	Brooklyn	Yaphank	Main St.
Peeled	0.19	0.09	0.00	1.55	0.20
Patched	1.21	0.00	0.13	0.00	0.00
Worn	0.48	0.25	0.32	0.00	0.00
Total	1.88	0.34	0.45	1.55	0.20

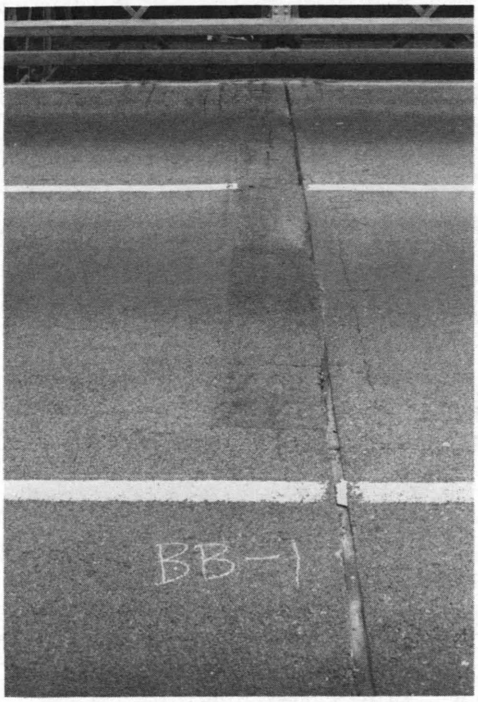

FIGURE 3 Cracking at relief joint has been patched.

overlays are installed in the field. Continued monitoring and expansion of the test patch program thus seem necessary.

On the basis of this review of the past performance of PC overlays and the types of distress noted on bridge decks, it seems advisable that these systems be considered only in two special cases: (a) for bridges where the weight of the overlay is critical, such as movable spans, or (b) where extended traffic disruptions are intolerable, such as in urban areas.

Use of PC overlays with high-strength, fast-curing characteristics and reasonable durabilities can result in minimal traffic delays and improved safety, and in some cases may eliminate the need for expensive detours. These desirable characteristics must be weighed against the need for continuing maintenance patching of the overlay to prevent possible failures because of the loss of adhesion. There is no apparent difference in the effectiveness of the various materials used in New York. Long-term studies should continue to investigate the nature of deterioration of the polymer after application and of polymer-deck concrete interactions. This could lead to the development of life cycle models for the various polymer products. Continued testing is also necessary to identify changes in deck conditions and to monitor the performances of existing overlays.

Further investigation of polyester overlays seems warranted on the basis of the results for the overlays placed on Long Island and the positive experiences of other states.

ACKNOWLEDGMENTS

The study described herein was performed under the administrative direction of Robert J. Perry and the technical supervision of Wes Yang, Engineering Research and Development Bureau, New York State Department of Transportation (NYSDOT). The work reported here was partially funded by FHWA, U.S. Department of Transportation. This paper reports work on projects of NYSDOT and the results of an e-mail survey of transportation engineers in other states. Internal NYSDOT memoranda and other unpublished documents, and correspondence and telephone conversations with other states are listed and cited in detail in the authors' *Polymer-Concrete Bridge-Deck Overlays* (*19*).

The authors thank the many contributors to this work. Data collection and laboratory analysis were performed by the Materials Bureau at the request of Dave Richards and Cathy Cowan. Regional personnel were highly cooperative during the field observations, especially Lou Adams and Barry Galowin of Region 10; Carlos Pinlac, John Kwok, Richard Lee, and Tom Bowers of Region 11; and Elias Haddad of Region 1. Marie Gaudreault of the St. Lawrence Seaway Authority arranged for observation of test patch installations and continues to provide information on the sites. The research suggestion that led to this work was submitted by Joe Savoie of Region 1. Contributions to planning of the project by Daniel C. Merkel are also gratefully acknowledged.

FIGURE 4 "Washboard" deterioration on Crown Point Bridge.

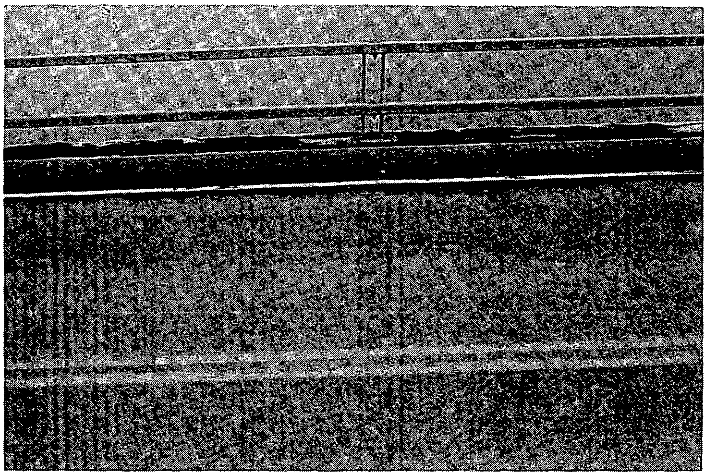

FIGURE 5 Cracking possibly resulting from deck flexure.

TABLE 3 Adhesion Test Results (psi = lb/in^2 = 6.89 kPa)

Location	Age, years	Overlay Material	Total Tests	Avg Bond Strength, psi
Queensboro Bridge				
South Upper Roadway	7	Flexolith	6	239
South Upper Roadway	7	Flexolith	8	99
North Upper Roadway	3	Flexolith	7	336
Brooklyn	6	Flexolith	9	224
Crown Point	1	Transpo T17X	9	267
Crown Point	0	Transpo T17X	9	324
University Heights	0	Flexolith	8	221

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Method for Detection of Chemical Reactions Between Concrete and Deicing Chemicals

HENRY J. GILLIS, JI-WON JANG, PAUL W. WEIBLEN, AND IWAO IWASAKI

An attempt was made to develop methods that can be used to detect chemical reactions between concrete and deicing chemicals and to determine if they actually occur. The results collected to date provide clear evidence of chemical reactions between concrete and the corrosion-inhibiting deicing salts. The different amounts of precipitates (chemical reaction products) found in the test cells are dependent on the type and the concentration of corrosion-inhibiting deicing salts.

Concrete degradation occurs through a variety of chemical and physical processes. Concrete may be degraded in three different ways distinguished by the prevalent signs of destruction (*1,2*) such as (a) decomposition of concrete by lime leaching, (b) chemical exchange reactions between hardened cement constituents and a solution, and (c) the accumulation, crystallization, and polymerization of reaction products.

It is well known that the use of deicing salts causes rebar corrosion in concrete and leads to structural failures. Corrosion inhibitors are mixed with plain rock salt (sodium chloride) to reduce or prevent rebar corrosion in concrete. Even though some reports (*3–8*) indicate the possibility of concrete degradation by deicing chemicals, the effects of the corrosion-inhibiting deicing salts and salt substitutes on concrete degradation are not well understood, and methods for determining the effects of the corrosion-inhibiting deicing salts and salt substitutes on concrete degradation are not available.

The result of previous research (*9*) suggest that the corrosion-inhibiting deicing salts interact with concrete and produce precipitates through a chemical reaction. If certain ions in the corrosion-inhibiting deicing salts that function as inhibitors are lost by precipitation, the effectiveness of the corrosion-inhibiting deicing salts on rebar corrosion could drop significantly. On the other hand the formation of precipitates in cracks may act as a barrier to the penetration of the salt solutions, thereby acting as an inhibitor. Alternatively some of the precipitates may form in the microcracks or pores of concrete and facilitate the propagation of cracks.

In the investigation described here an attempt was made to develop methods that can be used to detect chemical reactions between concrete and deicing chemicals and to determine if they actually occur.

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EXPERIMENTAL

Concrete slabs and cone-shaped concrete samples were designed to accelerate the deterioration of concrete in corrosion-inhibiting deicing salt and salt substitute solutions. The physical and chemical changes of the cone-shaped concrete samples are being monitored by visual examination and chemical and mineralogical analyses in an on-going research program.

Concrete Slab Samples

Concrete slabs [30 × 30 × 15 cm (12 × 12 × 6 in.)] [Figure 1(*a*)] were fabricated by using a 1.5-m^3 (2-yd^3) mix (183 × 183 × 183 cm) consisting of Portland Cement-Concrete [313 kg (691 lb)] with coarse [797 kg (1,757 lb)] and fine [523 kg (1,154 lb)] aggregates. The bottom halves of all of the slabs were cast from the mix as delivered, and the top halves were cast after adding 9 kg (20 lb) of NaCl to the remaining mix. After casting the slabs were placed in a moist room for a period of 28 days and were then dried in a chamber maintained at a temperature of 43 to 49°C (110 to 120°F) for 45 days. The air content determined by the linear transverse test was 13 percent. Originally the slabs were made for rebar corrosion testing with 3 percent solutions of corrosion-inhibiting deicing salts and salt substitutes.

Concrete slab samples ponded with 3 percent corrosion-inhibiting deicing salts and salt substitute solutions for 484 days were visually examined for rust stains, concrete cracks, and the roughness of the concrete surfaces caused by the corrosion-inhibiting deicing salts.

Cone-Shaped Concrete Samples

Cone-shaped concrete samples [Figure 1(*b*)] were made by mixing 374 kg (825 lb) of Type III cement, 635 kg (1,400 lb) of sand (Minnesota DOT Specification 3126), and 635 kg (1,400 lb) of quartzite meeting the CA-70 grade (Minnesota DOT Specification 3137). A paper mold was used to make the cone-shaped concrete samples. The cone shape of the samples was chosen to provide a large surface area exposed to the corrosion-inhibiting deicing salt solutions and to accelerate the chemical reactions at the tip. The physical changes in the concrete samples could readily be recognized at the tips. The concrete samples were placed in a moist room for 28 days after fabrication and were then air dried. The compressive strength of the concrete samples was 63.4 MPa (9,190 lb/in^2).

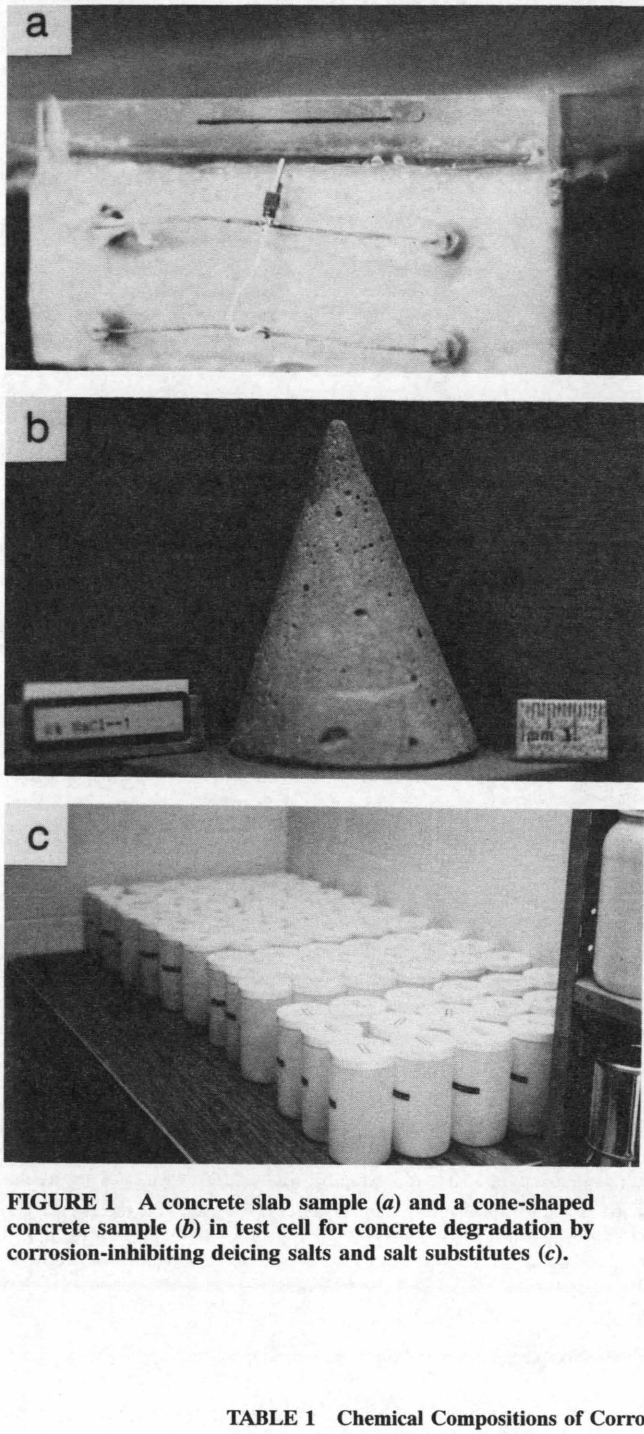

FIGURE 1 A concrete slab sample (*a*) and a cone-shaped concrete sample (*b*) in test cell for concrete degradation by corrosion-inhibiting deicing salts and salt substitutes (*c*).

The bottoms of the cone-shaped concrete samples were cut with a water-cooled diamond saw to make samples of uniform dimensions. The samples were cleaned with a Dayton 3Z856 sand blaster and graded Ottawa sand (ASTM C109) under 0.4 MPa (60 lb/in^2) of air pressure. The distance between the sand blaster nozzle and the samples was kept constant. The average dimension of the samples was a 5.08-cm (2-in.) bottom diameter by a 6.4-cm (2.5-in.) height. The initial dimension of each sample was measured by using a dial caliper, and the weight was measured with a Sartorius balance. The average weight of the samples was 120 g (0.26 lb).

Deicing Chemical Solutions

For the slab tests, CMA, Domtar, sodium formate, and plain salt were mixed into city water at a concentration of 3 percent. Plexiglas dams [Figure 1(*a*)] were bonded to the top surfaces of the slabs with silicone rubber, and the solution levels were kept constant on each slab.

For the cone-shaped concrete samples two corrosion-inhibiting deicing salts (A and B) and plain sodium chloride were mixed with deionized water at concentrations of 3, 6, and 20 percent (Table 1). The corrosion-inhibiting deicing salt solutions were filtered to remove insoluble constituents.

The concrete samples were immersed in individual polyethylene jars [Figure 1(*c*)] containing 500 ml of a corrosion-inhibiting deicing salt or plain sodium chloride solution. Three-milliliter solution samples are being collected periodically from the jars (test cells) to examine the chemical changes of the test solutions.

RESULTS AND DISCUSSION OF RESULTS

Concrete Cracks and Rust Stains on Slab Surfaces by Corrosion-Inhibiting Deicing Salt and Salt Substitute Solutions

Figure 2 shows typical slab surfaces after 484-day ponding with 3% CMA and Domtar solutions. Numerous concrete cracks and rough concrete surfaces were found on the slabs tested with the solutions. A summary of the average crack length and the average area of yellow rust stains of three slabs caused by rebar corrosion

TABLE 1 Chemical Compositions of Corrosion-Inhibiting Deicing Salts Used in Cone-Shaped Concrete Test Cells

	Ca	Mg	Na	K	P	SO_4	Cl
Deicing Salt A	0.35	2.71	27.64	0.03	2.36	2.14	52.01
Deicing Salt B	0.40	0.01	36.95	0.01	2.14	2.35	56.25
NaCl	0.00	0.00	39.00	0.00	0.00	0.003	60.00
Deionized water	0.00	0.00	0.00	0.00	0.00	0.00	0.00

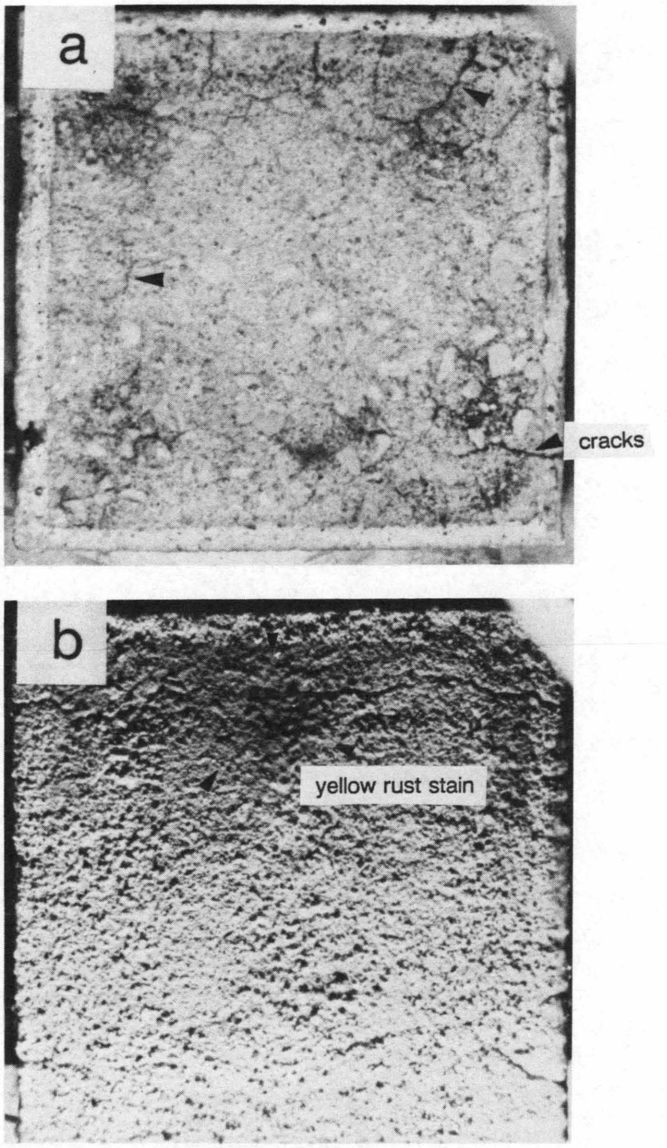

FIGURE 2 Top surfaces of concrete slabs showing (*a*) numerous concrete cracks and (*b*) yellow rust stain.

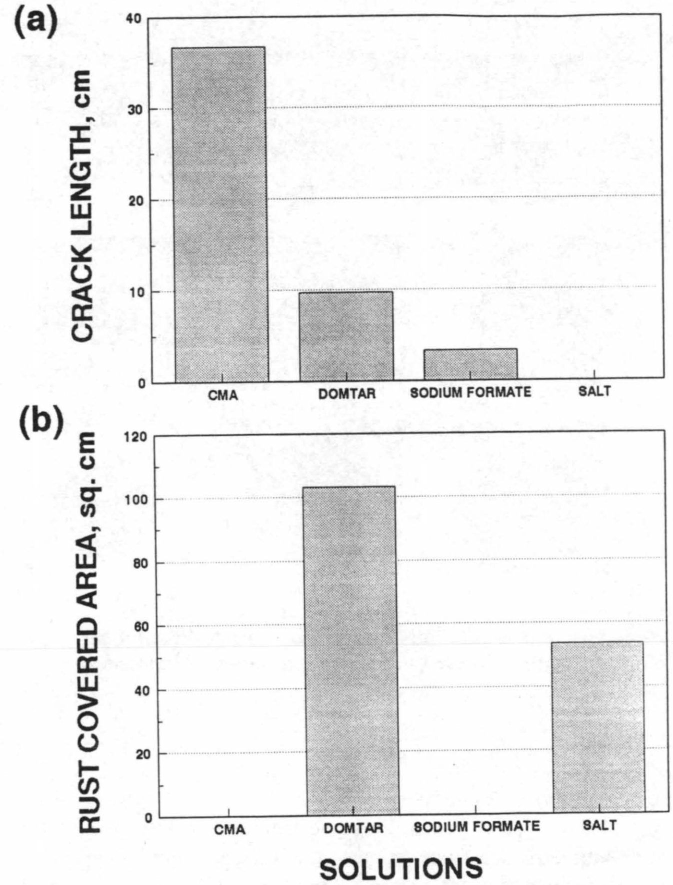

FIGURE 3 Average concrete crack length (*a*) and average area of yellow rust stains (*b*) caused by 3 percent corrosion-inhibiting deicing salts and salt substitute.

is shown in Figure 3. The slabs tested with CMA and sodium formate solutions contained cracks [Figure 3(*a*)] but did not show any yellow rust stains on their surfaces [Figure 3(*b*)]. On the other hand the slab surfaces tested with salt (NaCl) solutions showed signs of rebar corrosion in concrete, but no cracks were observed (Figure 3). Figures 2 and 3 indicate that the cracks in the concrete slabs were created by both rebar corrosion and chemical reactions between deicing chemicals and concrete. These results made it of interest to investigate the concrete degradation caused only by chemical reactions between the deicing media and concrete.

Precipitates in Corrosion-Inhibiting Deicing Salt Solutions

To evaluate the effects of the chemical reaction between corrosion-inhibiting deicing salts and concrete, the cone-shaped concrete samples were placed in the 3, 6, and 20 percent corrosion-inhibiting deicing salt and plain sodium chloride solutions. After leaving the concrete samples in the test solutions for a day various amounts of precipitates were found in the test cells depending on the type and the concentration of corrosion-inhibiting deicing salts. Figures 4 and 5 show that the precipitates were formed by chemical reactions between the concrete and the corrosion-inhibiting deicing salts. The precipitates were observed on the concrete sample surfaces, at the bottoms of the test cells, or both. However, no precipitates were found in the test cells containing NaCl (Figure 6) and deionized water (Figure 7). For Deicing Salt A (Figure 4) the amount of precipitates increased with an increased concentration of corrosion-inhibiting deicing salt. For Deicing Salt B (Figure 5) the amount of precipitates decreased with an increased concentration of corrosion-inhibiting deicing salt. It appears that the amount of precipitates increased with time in all cases.

The chemical changes in the test solutions as well as the physical changes in the concrete samples are being monitored as a function of time. The results collected so far provide clear evidence of chemical reactions between concrete and the corrosion-inhibiting deicing salts. The impact of the chemical reactions on concrete degradation can be understood by determining the chemical and mineralogical changes in the concrete caused by the corrosion-inhibiting deicing salts. The chemical changes in the solutions provide part of the necessary information on changes in

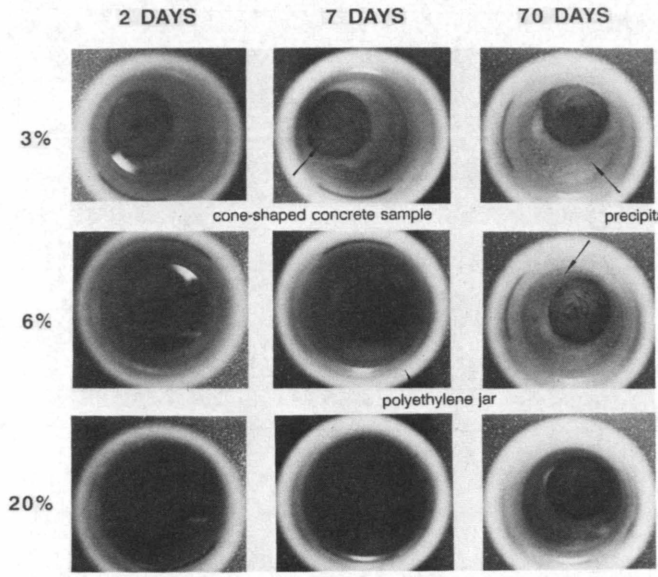

FIGURE 4 Top views of test cells containing 3, 6, and 20 percent corrosion-inhibiting Deicing Salt A. Precipitates increased with increased concentration and as a function of time.

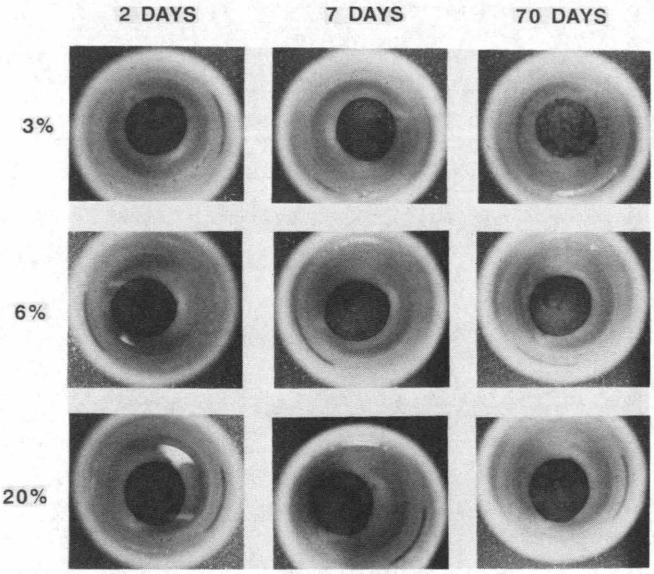

FIGURE 6 Top views of test cells containing 3, 6, and 20 percent NaCl. No precipitate found after 70 days.

the chemistry and bonding strength of concrete, but the mineralogy of the precipitates must also be determined. The precipitates are being collected from the test cells for quantitative and qualitative chemical and mineralogical analyses to define the chemical reactions.

The physical changes in the concrete samples after 400 days of reaction have not been significant enough (i.e., a minimum of 15 percent weight and dimension changes) to determine the extent of changes by the corrosion-inhibiting deicing salt solutions. The physical properties (volume changes on dry-wet and freeze-thaw cycling) of the precipitates will be determined at a later date.

CONCLUSIONS

The results collected to date provide clear evidence of the chemical reactions between concrete and the corrosion-inhibiting deicing salts. The different amounts of precipitates found in the test

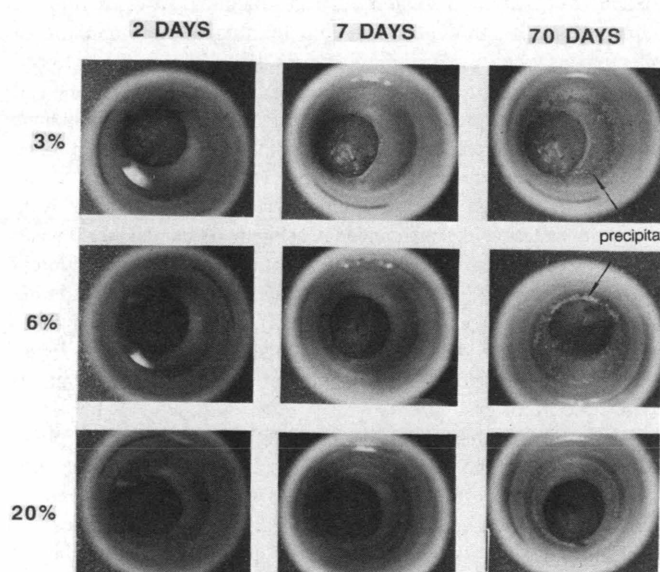

FIGURE 5 Top views of test cells containing 3, 6, and 20 percent corrosion-inhibiting Deicing Salt B. Precipitates decreased with increased concentration.

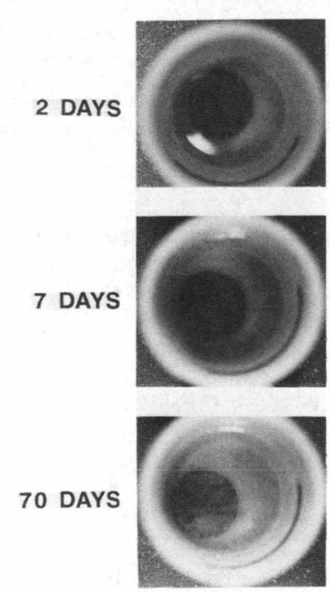

FIGURE 7 Top views of test cells containing deionized water. No precipitate found after 70 days.

However further research is necessary to understand the mechanisms of concrete degradation by corrosion-inhibiting deicing salts. The concrete in the cone-shaped concrete samples are expected to show significant physical changes resulting from the corrosion-inhibiting deicing salts with time. The precipitates must be collected from the test cells for quantitative and qualitative analyses to identify the degree of chemical reaction, the chemical elements involved in the reactions, and the mineralogy of the precipitates. The volume changes in the precipitates in dry-wet and freeze-thaw cycling will indicate the impacts of the precipitates on microcracks in concrete.

ACKNOWLEDGMENTS

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PART 4

Snow and Ice Control

Economic Impact of Winter Road Maintenance on Road Users

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State and local governments spend significant amounts of both human power and money annually on snow and ice control programs. On the basis of previous studies expenditures to counteract adverse road surface conditions during snow and icy conditions consume as much as 33 percent of some state highway maintenance budgets. These poor road surface conditions exist for only 4 to 8 percent of the total time that these roads are used. Expenditures of this magnitude have traditionally been justified through improved road user benefits, usually increased safety and decreased traffic delay. An important objective of evaluating the effectiveness of winter road maintenance operations (salting, plowing, and following a bare pavement policy) is determination of the economic impact of such operations on road users from a safety standpoint. The approach taken in evaluating the financial benefits of removing the adverse road surface conditions during snow and icy conditions is to compare the savings (benefits) to the costs expended in deriving them. It is concluded that winter road maintenance operations result in average direct savings to road users of 45 and 20 cents per vehicle kilometer of travel on two-lane highways and multilane freeways, respectively. The costs of winter road maintenance operations are offset by the benefits within the first 35 min after establishing bare pavement.

During winter, snow and ice on streets, roads, and highways cause hazardous driving conditions. Failure to act and deice roads effectively and as soon as possible creates general threats to the health and safety of the members of the community. Reducing the level of effort for snow removal and ice control has immediate consequences on traffic delay, traffic volumes, traffic congestion, and the public's image of the state department of transportation. Without close attention to the effective removal of snow and ice from streets, roads, and highways during periods of snow and icy conditions, local economies will suffer, traffic accidents will escalate, and most activities of individuals, industries, utilities, schools, and governments are handicapped in social and economic ways. This paper analyzes and evaluates the economic impacts of winter road maintenance operations (salting, plowing, and following a bare pavement policy) on road users in three states (New York, Illinois, and Wisconsin) and shows any cost savings per vehicle kilometer of travel (VKT).

BACKGROUND

A methodology was prepared for selecting the test sections and reviewing the exposure data, winter maintenance operations records, and traffic accidents on all of the selected test sections in the three states (*1,2*). The methodology covered the traffic accidents that were reported to have occurred during the research periods on two-lane highway test sections totaling 1,600 lane-km in New York (Wayne and Tompkins counties), Wisconsin (Walworth County), and Illinois (Ogle and Lee counties) and on multilane divided (freeway) test sections totaling 400 lane-km in New York (Cortland and Monroe counties), Wisconsin (Walworth County), and Illinois (Ogle County). In addition to the traffic accident data detailed information was also available on winter road maintenance operations, including the date, the operation route, times of salt applications, state of the weather, the time of day, and so on. A very time consuming before-and-after method was used to analyze these data. After determining the hour that the deicer (salt) application (just preceding the establishment of bare pavement) occurred (zero hour), hourly intervals were taken backwards and forwards (up to 12 hr in each direction) (*1,2*). The incident of each reported traffic accident was determined by time, severity, location, and traffic density. These data were then used to work out traffic accident rates and cost rates for every elapsed time (12, 11, . . . , 1) before establishing bare pavement and its conjugate after. An illustration of the compilation approach is presented in Figure 1. The reduction in traffic accident rates from before to after were found to be statistically significant (*2*). The methodology was presented in detail in an earlier paper (*2*).

ECONOMIC ANALYSIS

After confirming that the reduction in traffic accident rates from before to after were statistically significant (*1,2*) at the selected level of confidence (99 percent, 95 percent, or both), it is appropriate to conduct the economic analysis for winter road maintenance and its impact on traffic accident occurrence. Conclusions on costs and benefits documented in other studies and serving the purpose of the present analysis were reviewed.

Literature Review

A benefit-cost analysis study is one that identifies both the beneficial effects and the cost effects of winter road maintenance operations and that quantifies all effects in term of dollars. Comparisons can then be made to determine whether the economic benefits exceed the costs. The costs of winter road maintenance can be classified as follows:

1. Direct costs, that is, expenditures made by a government agency to remove snow and ice:
   –Labor,
   –Equipment,
   –Materials, and
   –Administration.

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STEP ONE:
- For each sub-event; two points of time need to be identified:
  - The approximate time of establishing the policy's goal on the testing section "**P**"; and
  - The hour that the deicer application (just proceeded the establishment of the policy's goal) occurred is called zero hour "**0**".
- Take hourly intervals backwards and forwards (up to 12 hours) from "**0**", and identify each interval's traffic volume and accidents.

HWY Section	Sub-event	Sub-events *(shaded cells below)* based on hour of the day																							
1	1	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	0	17	P	19	20	21	22	23	24
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1	2	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	0	P	23	24
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•
87	3979	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
		0	2	P	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

STEP TWO:
- Hourly intervals are now defined by relative hours (n; where n = -12, -11, ..., -1, +1, ..., +11, +12) before and after the "**0**";
- For all sub-events; categorize and total traffic accidents, and calculate and total VKmT per each relative hour (n); and
- Calculate the traffic accident rates for each elapsed time (Z Hours; where Z = 12, 11, ..., 1) before and its conjugate after.

HWY Section	Sub-event	Sub-events *(shaded cells below)* based on relative hour (n)																								
		Before (-)												0	After (+)											
1	1	12	11	10	9	8	7	6	5	4	3	2	1	0	1	2	3	4	5	6	7	8	9	10	11	12
1	2	12	11	10	9	8	7	6	5	4	3	2	1	0	1	2	3	4	5	6	7	8	9	10	11	12
•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•
87	3979	12	11	10	9	8	7	6	5	4	3	2	1	0	1	2	3	4	5	6	7	8	9	10	11	12
Hourly Total		Categorized traffic accidents & VKmT for each n												0	Categorized traffic accidents & VKmT for each n											
Accident Rates		For each elapsed time (Z hours) from the zero hour												0	For each elapsed time (Z hours) from the zero hour											

FIGURE 1 Compilation approach for two-lane test sections.

2. Indirect costs, that is, expenditures resulting from the procedures or conditions used for winter road maintenance:
   - –Damage to vehicles from deicing materials,
   - –Damage to pavements and structures from deicing materials,
   - –Property damage and deaths resulting from accidents between winter maintenance equipment and vehicles,
   - –Costs to the user to remove snow and ice from driveways, and
   - –Other indirect costs.

In all studies conducted so far the direct costs are of primary interest. It has also been found that it is an extremely expensive operation to get accurate estimates of indirect costs. Minimizing the direct costs while achieving the desired appropriate level of service is a major goal in most highway departments. In the Snow Belt states every highway agency has been trying to make winter road maintenance operations as cost-effective as possible. Safe winter driving conditions must be provided to the public; at the same time expenditures must be kept to a minimum, since winter road maintenance operations do not provide any lasting improvement to the highway system. Labor represents the largest class of expenditure in highway maintenance activities. The level of winter road maintenance can vary from one unit of government to another, as well as within a unit of government; the expenditures including all of the direct and indirect costs vary accordingly. Each county or state can calculate the range of expenditures for direct costs at the end of each winter season. This range depends mainly on the number of snowstorms, the actual depth of snow, the water content of the snow, the duration of the storms, temperature, prevailing wind, and other factors.

The benefits that accrue as a result of the open roads in winter can be classified as follows:

1. Direct benefits, that is, road user savings from using the road during snow and ice conditions:
   - –Vehicle operating cost reduction;
   - –Travel time reduction; and
   - –Avoidance of property damage, personal injury, or loss of life.
2. Indirect benefits, that is, those benefits derived from the community:
   - –Availability of community services (hospitals, postal services, schools, police, and fire protection), and

–Maintenance of business (industry, government, personal income).

Many of the benefits are intangible. The most commonly claimed road user benefits have been increased safety and decreased traffic delays, particularly in urban areas. These benefits are the most recognized and immediately apparent during snow conditions. Changing the level of service or the level of effort for snow and ice removal and control have immediate effects on road users' safety, travel time, traffic volumes (which can be altered), traffic congestion, and public acceptance.

Vehicle Operating Costs

In reality and as a result of an effective snow and ice control policy, there have been many savings in vehicular operating costs. For vehicles some savings are generally converted to time savings by driving at increased speeds, with a probable increase in fuel and tire costs (*3*).

Fuel Consumption

All fuel consumption economic estimates for vehicles on snow- and ice-covered highways were derived from Claffey's work (*4*).

1. Road surface. Claffey (*4*) considered the road surface condition to be an important variable affecting average vehicle fuel consumption. The average operating pavement speed during snow and icy conditions was converted to fuel consumption in liters per kilometer (gallons per mile). The excess fuel consumption per kilometer (mile) multiplied by highway length yielded the total number of liters (gallons) of fuel consumed in excess of normal fuel consumption. Fuel consumption varies with travel speed, and this has been accounted for in Claffey's work, as shown in Table 1 (*4*).
2. Type of vehicle. In Claffey's work (*4*) additional fuel consumption was applied only for passenger car vehicles, light trucks, and vans. For estimating additional fuel consumption it was assumed that large trucks did not contribute to traffic volumes during snow.
3. Level of service. The level of service would affect the fuel consumption rate of vehicles traveling through snow and ice conditions. In a given snowstorm the snow and ice control policy set for the level of service would form the basis for the best estimate of road surface conditions. Modification of the level of service affected the economics of fuel consumption by causing corresponding changes in pavement condition (*4*).

Tires, Oil, and Vehicle Maintenance

During snow and ice conditions vehicle operating expenses such as those for tires, oil, and vehicle maintenance have not been studied or included in any research to date. These expenses are usually a function of distance and are not time driven (*3*). As a result of changing the level of service, the changes in these expenses are considered to be insignificant.

Fixed Operating Costs

Fixed operating costs (insurance, depreciation, taxes) are yearly expenditures and are not a function of time or distance driven (*3*).

Travel Time

As a result of snow or ice on pavement the vehicle traveling time on a given highway segment increases. For a given highway trip the vehicle traveling time with snow and ice on the pavement must be compared with the normal vehicle traveling time under dry pavement conditions. Normal trip times have been related to the type of highway, traffic volume, capacity ratios, sight distance, time of day, and many other variables (*5*). The magnitude of the delay for any one vehicle is given by (*6*)

$$\text{Delay} = \text{trip length} \times \left(\frac{1}{\text{snow speed}} - \frac{1}{\text{normal speed}}\right) \tag{1}$$

The total delay for all vehicles traveling on a given roadway segment can be calculated from

$$\text{Total delay} = \frac{\text{mean delay} \times \text{AADT} \times M_i \times D_j \times H_k \times V_r \times L}{\text{average trip length}} \tag{2}$$

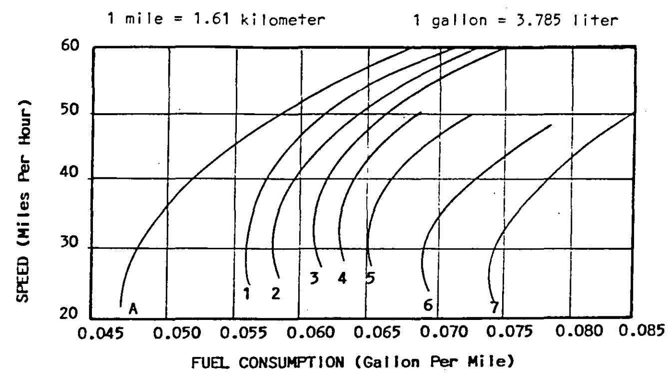
where

AADT = average annual daily traffic,
M_i = traffic monthly factor for month i of year,
D_j = traffic daily factor for day j of week,
H_k = hourly index for hour k of day,
V_r = volume reduction factor because of snow, and
L = roadway segment length.

It has been the custom in highway economics studies to price travel time in dollars, so that the figures could be used in the economic analysis. No publication so far has determined the value of travel time in a scientific manner. The basic work was done by Claffey (*4*) in 1960 and 1961. The result of Claffey's work (*4*) was the value of \$1.42/hr for a passenger car. Later two more studies were published by Thomas and Thompson (*7*) and Lisco (*8*). The study of Thomas and Thompson (*7*) followed the same procedure used by Claffey (*4*), but with a much improved mathematical model. The study of Lisco (*8*) was done in the Chicago Loop area, and it compared the cost of driving vehicles with riding the rapid transit rail system. The study of Lisco (*8*) used a mathematical model almost identical to that used in the study of Thomas and Thompson (*7*). The study of Thomas and Thompson (*7*) concluded a value of about \$2.80/person-hr, and the study of Lisco (*8*) concluded a value of \$2.50/person-hr. These two values in dollars per hour of travel time represent highly selective conditions (peak urban hours). The AASHTO *Manual of User Benefit Analysis of Highway and Bus-Transit Improvements* (*9*) presented a chart that can be used to obtain estimates of travel time value once the time savings per trip for a particular highway are known. However uncertainties over the exact time saved by cumulative improvements may be so great that it becomes a needless refinement to select the exact point on the curve that corresponds to the

TABLE 1 Passenger Car Fuel Consumption in Snow Belt States under Snow and Icy Conditions (*4*)

(a) Passenger Car Fuel Consumption in Snow Belt States under Snow & Icy conditions



A = Dry Pavement.
1 = Very Slippery Hard-Packed Snow and Ice.
2 = Hard-Packed Snow and Ice with irregular bumpy wippled surface.
3 = 12 mm. (1/2 inch) new snow on hard-packed snow.
4 = 19 mm. (3/4 inch) new snow on hard-packed snow.
5 = 25 mm. (1.0 inch) new snow on hard-packed snow.
6 = 38 mm. (1.5 inch) new snow on hard-packed snow.
7 = 50 mm. (2.0 inch) new snow on hard-packed snow.

(b) Correction Factors to Adjust Passenger Car Fuel Consumption for Snow & Icy Conditions

Speed (Km/Hr)	Dry Pavement	Very Slippery Hard-Packed Snow	Hard-Packed Snow & Ice with Bumpy Surface	New Snow on Hard-Packed Snow (mm)				
				12	19	25	38	50
32	1	1.23	1.30	1.36	1.43	1.47	1.51	1.60
48	1	1.16	1.20	1.28	1.32	1.35	1.45	1.54
64	1	1.11	1.14	1.20	1.23	1.28	1.40	1.48
80	1	1.06	1.10	1.12	1.18	1.24	1.34	1.45
96	1	1.04	1.08	1.10	----	----	----	----

Note: Correction factors are designed to be applied to values obtained from Claffey's charts and the above table (a).

estimated cumulative time savings for each curve. A conservative value of travel time per hour of $3.00 (1975 U.S. dollars) per passenger car was generally used throughout the AASHTO manual (*9*). This estimate was derived from a value of $2.40/person-hr and 1.25 persons/vehicle. For trucks the manual used a travel time per hour of $7 to $8/truck on the basis of 1975 truck driver wages and fringe benefits.

Accident Costs

Motor vehicle accident costs are an important component in benefit-cost evaluations of highway safety improvements. Among the attempts that were made in the last decade to provide comprehensive estimates of motor vehicle accident costs is a 1984 study by Miller et al. for FHWA (*10*). That study did not express accident costs in a form that can be directly used in benefit-cost analyses. In a paper published later accident costs from the cost data presented in the study by Miller et al. (*10*) and accident data from five states were developed by McFarland and Rollins for FHWA (*11*). These accident costs can be used directly with state accident data, thereby facilitating the use of state-of-the-art accident cost estimates in benefit-cost analyses of highway safety improvements.

Winter Maintenance Costs

The complexities of obtaining accurate cost records for municipal, county, and state governments are so great that an early decision was made to restrict the current study to cost records available from each testing area's department of transportation. The level of winter road maintenance can vary from one unit of government to another as well as within a governmental unit. Total cost information for winter road maintenance for each participating county was provided by the highway department authority in the respective state. These costs are based on the expenditures and direct costs of the winter road maintenance program during the winter season of each county's testing period. Figure 2 presents the winter road maintenance cost for each test area (the participating areas are not identifed in Figure 2) as the cost, in U.S. dollars per lane kilometer per average snowstorm (total snowfall divided by total number of snow and ice storms for each participating area).

METHODOLOGY

An assessment of the combined effects of winter road maintenance operations (salting, plowing, and following a bare pavement policy) on cost and traffic accident reduction was performed to determine whether the economic benefits exceed the costs and to show any cost savings per vehicle kilometers of travel.

Benefit-Cost Analysis

The computation of winter road maintenance benefits requires an estimate and calculation of the savings that would accrue to the highway users (direct benefits) and to the community (indirect benefits).

Winter Road Maintenance Benefits

In calculating the winter road maintenance benefits it is necessary to understand the following points:

1. The savings accruing from winter road maintenance are normally called benefits;
2. Winter road maintenance benefits are either difficult to measure in terms of dollars or border on being intangible; and
3. Almost all indirect winter road maintenance benefits are intangible; that is, the ability to assess a dollar value is impractical or cumbersome.

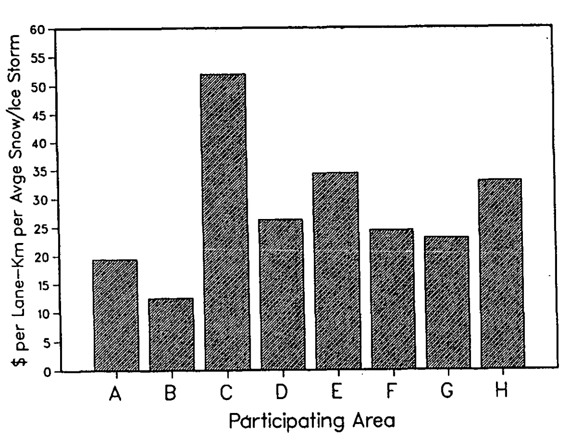


FIGURE 2 Average unit cost for winter road maintenance.

To strengthen the computation in this analysis all previous methods used in other research projects for computing road improvement benefits were studied. The road user savings considered for computing the direct benefits of winter road maintenance in this analysis were as follows:

1. Road user safety. As a result of winter road maintenance the primary benefit accrues from the reduction in the before traffic accident rates to its conjugate after. Two estimates of traffic accident costs (1980 and 1990) were presented earlier: the 1980 McFarland and Rollins (*11*) estimate for FHWA, which was updated to the year 1990, and the 1990 National Safety Council estimate (*12*). During the course of the present study a third 1991 FHWA estimate based on 1988 U.S. dollars and updated to the year 1990 was considered for accident cost calculations (*12*). The 1991 FHWA estimate based the costs of motor vehicle crashes on the allocation of scarce highway safety resources to maximize benefits and evaluated proposed safety regulations. The costs included medical expenses, emergency services, workplace costs, travel delay, property damage, and administrative and legal out-of-pocket expenses. It also included cost of wages and household production and cost of pain, suffering, and lost quality of life. On the other hand the other two estimates based their costs only on medical expenses, wage loss, insurance administration cost, and motor vehicle property damage. The 1991 FHWA estimate [$2,950,000/fatal accident, $75,400/injury accident, and $4,900/property damage only (PDO) accident] was used throughout the present study as the basis in traffic accident cost-related calculations. The road user safety benefit calculations are based on the traffic data discussed, analyzed, and evaluated in earlier papers (*1,2*). It should be noted that each winter road maintenance operation was studied very carefully. On the basis of winter road maintenance records provided by highway authorities, the approximate time of establishing their policy's goal (bare pavement) for each subevent (winter road maintenance operations on one test section to counteract snow and icy conditions during any one snowstorm) was determined. The deicer (salt) application that just preceded the establishment of the policy's goal was used for the analysis. For each subevent the hour that the deicer (salt) application occurred is called the zero hour. Traffic accident rates for each elapsed time (Z hr, where Z = 12, 11, 10, . . . , 1) before the zero hour and its conjugate after were calculated. Figure 1 illustrates the compilation approach for two-lane sections. To determine the effect of winter road maintenance on traffic safety, a comparison between each elapsed time's (Z hr) traffic accident rate before and its conjugate after was conducted. Figures 3 and 4 present a summary of the analysis results. The following example illustrates the procedure for the road user safety benefit calculations:

$$\text{Accident rate reduction}_{\text{categ},Z} = \text{rate}_{\text{before},Z} - \text{rate}_{\text{after},Z}$$

$$\begin{aligned}\text{Accident rate reduction}_{\text{inj},4} &= 4.99 - 0.58\\ &= 4.41 \text{ traffic accidents (injury)}/10^6 \text{ VKT}\end{aligned}$$

$$\begin{aligned}\text{Accident rate reduction}_{\text{PDO},4} &= 2.96 - 0.45\\ &= 2.51 \text{ traffic accidents (PDO)}/10^6 \text{ VKT}\end{aligned}$$

$$\begin{aligned}\text{Road user safety benefit}_4 &= 4.41 \times 75{,}400 + 2.51 \times 4{,}900\\ &= \$345{,}000/10^6 \text{ VKT}\end{aligned}$$

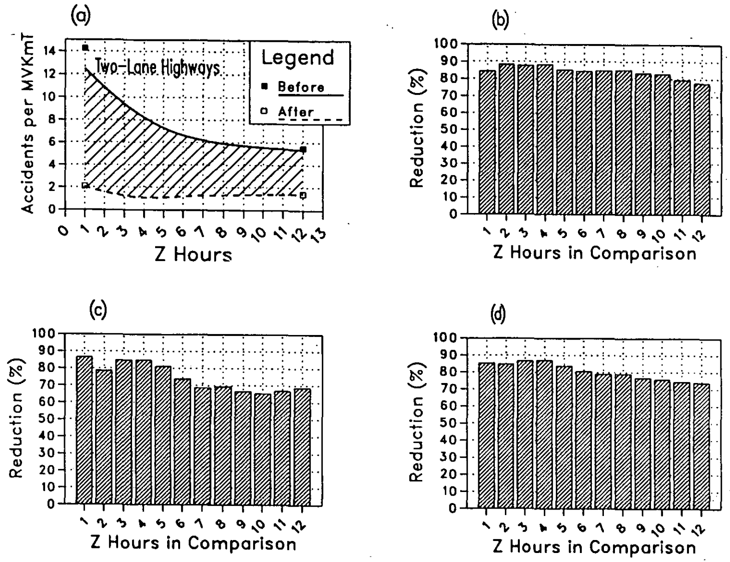


FIGURE 3 All traffic accident rates and their reductions from before to after on two-lane undivided testing sections: *a*, all traffic accident rates; *b*, injury rate reductions; *c*, PDO rate reductions; *d*, all accident rate reductions.

In addition to the road user safety benefit, AASHTO has summarized the benefits to road users in the form of reduced vehicle operating costs, decreased travel time, and increased comfort and convenience (*13*). The most important factor in determining these benefits as a result of winter road maintenance during snow and icy conditions is the running vehicle speed. Before predicting the speed of vehicles on the test sections during the winter period researched, it is necessary to know the conditions of the roadway before and after establishing bare pavement. As an example assume that present snow and ice maintenance operations were performed adequately during a snowstorm that lasted 6 hr and that the snowfall was about 50 mm (2 in.). Before the start of the storm state highway department trucks are loaded with a deicer (in the present study, salt) and are standing by. As soon as there is an indication that snow will stick, salt is applied to the roadways in the respective areas. During the first hour of the storm there

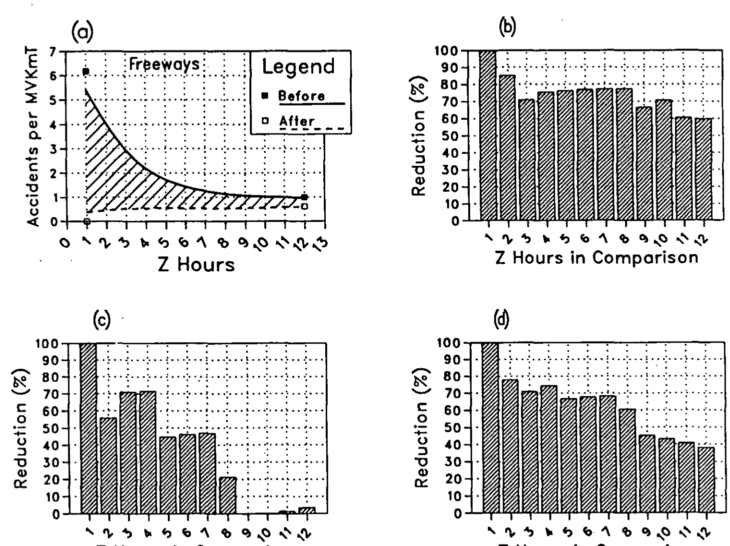


FIGURE 4 All traffic accident rates and their reductions from before to after on multilane divided testing sections: *a*, all traffic accident rates; *b*, injury rate reductions; *c*, PDO rate reductions; *d*, all accident rate reductions.

may be some accumulation of snow and this will tend to slow traffic. However the salt takes effect after this period and the pavement becomes wet and clear. Under light storm conditions [less than 50 mm (2 in.) of snow] the pavement generally remains clear throughout the entire storm after the first salt application. Traffic is able to maintain speed at all times except during the first hour.

Assume that nothing is done to the roadways. Once snow has started to fall on the test sections more would stick and accumulate. Under heavy traffic this would probably pack very fast and within 0.5 hr an ice crust would form. Traffic would slow considerably and the accumulated snow and ice would become quite rough. This would tend to slow traffic even more. Traffic would not be able to maintain normal speed during or after the storm. Traffic would be restricted to 32 to 48 km/hr (20 to 30 mph) on this icy road surface in comparison with a normal speed of 56 to 72 km/hr (35 to 45 mph) on two-lane rural highways and a normal speed of 40 to 56 to 90 to 105 km/hr (25 to 35 to 55 to 65 mph) on multilane rural freeways. A similar circumstance could be pictured for the various priority highways and different storm conditions. If temperatures remained low and nothing was done to remove ice from the pavement, the ice could remain on the highway for weeks.

A previous study (*6*) reported that the average ranges of vehicle speed reduction during snow and icy conditions were 18 to 42 percent and 13 to 22 percent on two-lane highways and freeways, respectively. In the present study, and to be conservative, the average vehicle speed reductions as a result of snow and icy conditions for two-lane rural highways and multilane rural freeways are 25 percent [16 km/hr (10 mph)] and 15 percent [15 km-hr (10 mph)], respectively, of their average normal speeds [65 and 100 km/hr (40 and 60 mph)]. Time savings are also generated from driving at normal speeds. Time savings or time losses associated with winter road maintenance can be valued within a marketable concept.

2. Vehicle operating cost. Snow and icy conditions restrict vehicle movement in a variety of ways, depending on the actual condition of the ice or snow on the pavement. The roadway surface condition has an effect on the vehicle operating cost. A slippery road surface or a road covered with snow and ice would tend to increase the vehicle operating cost. Snow, and particularly ice, increases fuel consumption by causing slippage of the traction wheels, which in turn produces engine revolutions without corresponding vehicle movement.

On the basis of the data presented in Table 1 the vehicle fuel consumption cost savings accruing from winter road maintenance operations on roadways were calculated as follows.

–By using Claffey's charts (*4*) presented in Table 1 the average operating vehicle speeds were converted to fuel consumption in liters per kilometer (0.435 gal/mi) as follows: [the conservative assumption is that the average snow accumulation is 12 mm (0.5 in.)]:

$$\begin{aligned}\text{Normal average vehicle speed, 40 mph} &= 0.053\ \text{gal/mi}\\ &= 0.023\ \text{L/km}\\ \text{Adjustment because of snow } (0.053 \times 1.2) &= 0.063\ \text{gal/mi}\\ &= 0.027\ \text{L/km}\\ \text{Normal average vehicle speed, 60 mph} &= 0.068\ \text{gal/mi}\\ &= 0.029\ \text{L/km}\\ \text{Adjustment because of snow } (0.068 \times 1.1) &= 0.075\ \text{gal/mi}\\ &= 0.032\ \text{L/km}\end{aligned}$$

–The operation cost savings per vehicle kilometer was calculated by multiplying the excess fuel consumption in liters per vehicle kilometer

$$\Delta\text{LPK} = \text{LPK}_{\text{snow}} - \text{LPK}_{\text{normal}}$$

by the average fuel cost per liter during the test period [on the basis of the data obtained from the consumer price index (CPI) reports (1989 to 1991) for the average prices of gasoline during the months of the test period] and as follows:

$$\text{CPL} = \frac{\sum_{M=1}^{3-8} (\text{gasoline price} \times \text{VKT})_{M,\text{area}}}{\sum_{R=1}^{R=2} \text{VKT}_R} = \$0.50/\text{L}$$

where

CPL = average cost per liter of fuel (\$/L),
R = northeast or north-central region, and
M = testing period month.

The following example illustrates the procedure for the vehicle operating cost calculations:

$$\Delta\text{LPK} = 0.027 - 0.023 = 0.004\ \text{L/km}$$

$$\begin{aligned}\text{Fuel consumption savings} &= 0.004 \times 0.5\\ &\approx \$0.01/\text{VKT (conservative)}\end{aligned}$$

3. Travel time. Severe weather conditions such as those resulting from snow- and ice-covered pavements increase the time of vehicle travel on a given highway section. Snow travel delay is the additional time required in an average trip beyond the normal travel time. The economic value of increased travel time on a highway section resulting from snow and ice is tempered by several variables, including type of vehicles, normal time delays, trip purpose, time of day, and increased fuel consumption. The procedure for determining the travel time savings that accrue from the winter road maintenance operations on the roadways is as follows:

–A conservative \$3.00 value of time savings per hour based on a 1975 estimate was obtained from the AASHTO manual (*9*). This value was updated to the year 1990 by using the gross national product (GNP) deflator:

$$\begin{aligned}\text{Time saving}_{1990} &= \$3.00 \times \frac{\text{cost of market basket}_{1990}}{\text{cost of market basket}_{1975}}\\ &= \$3.00 \times 131.40/59.30 = \$6.65/\text{hr (conservative)}\end{aligned}$$

and the CPI:

$$\begin{aligned}\text{Time saving}_{1990} &= \$3.00 \times \frac{\text{cost of budget}_{1990}}{\text{cost of budget}_{1975}}\\ &= \$3.00 \times \frac{130.70}{53.80} = \$7.28/\text{hr}\end{aligned}$$

Economists often consider the GNP deflator to be a better measure of overall inflation in the economy than the CPI. The CPI is based on the budget of a typical urban family. By contrast the GNP deflator is constructed from a market basket that includes every item in the GNP, that is, every final good and service produced by the economy. Thus in addition to prices of consumer goods, the GNP deflator includes the prices of airplanes, lathes, and other goods purchased by business. It also includes government services (*14*).

–On the basis of Equations 1 and 2 the following equation was generated and used to calculate the reduction in vehicle travel time (prevented delay) after the zero hour of salt spreading that was considered and used

$$\text{Average delay}\left(\frac{\text{hr}}{\text{VKT}}\right) = \frac{1}{\text{snow speed}_{\text{avg}}} - \frac{1}{\text{normal speed}_{\text{avg}}}$$

–The total savings value in travel time per vehicle kilometer of travel was calculated on the basis of the following equation:

$$\text{TTS} = \text{TSV}_{1990} \times \text{TS}$$

where

TTS = travel time saving value ($/VKT),
TSV_{1990} = updated time saving value ($/hr), and
TS = vehicle travel time saved per (hr/km).

The following example illustrates the procedure for the travel time saving calculations:

$$\text{Average delay} = 1/48 - 1/65 = 0.00545 \text{ hr/VKT}$$
$$= 20 \text{ sec/VKT (30 sec/VMT)}$$

$$\text{TTS}_{\text{GNP}} = 6.65 \times 0.00545$$
$$= \$0.036\text{/VKT (conservative)}$$

Winter Road Maintenance Costs

The computation of winter road maintenance costs requires an estimate and calculation of:

1. Expenditures made by a government agency to remove snow and ice from highways, roads, and streets (direct costs), and
2. Expenditures that result from the procedures or conditions utilized for winter road maintenance (indirect costs).

In calculating the winter road maintenance costs it is necessary to understand the following points:

1. The primary interest in most previous economic studies is the direct costs; and
2. It is a complicated and extremely costly operation to estimate accurate values for indirect costs.

The winter road maintenance direct cost used in the present analysis is the expenditures (labor, equipment, material, and administration) made by the government agency in each participating area to counteract snow and icy conditions on roads and highways. The direct cost was provided as a total cost rate [dollars/lane-kilometer (lane-mile)/average snowstorm] by the authorities in the highway department of each respective participating area. The average direct cost of winter road maintenance per lane-kilometer (lane-mile) per average event was calculated on the basis of the data presented in Figure 2, the length of the test sections, and the following equations:

$$\text{TDC}_p = \text{CR}_p \times \text{TL}_p \times \text{No. of Events}_p$$

$$\text{ADC (\$/lane-km/event)} = \frac{\sum_{p=1}^{4-5} \text{TDC}_p}{\sum_{p=1}^{4-5} (\text{TL}_p \times \text{No. of Events}_p)}$$

where

TDC_p = total direct costs during this study period in p area ($);
CR_p = cost rate of winter road maintenance in p area ($/lane-km/average snowstorm);
TL_p = total testing sections length in p area (lane-km);
ADC = average direct cost ($/lane-km/average event);
z = elapsed time after winter road maintenance (hr); and
p = for undivided highways, Wayne, Tompkins, Walworth, Ogle, or Lee county; for divided highways, Monroe, Cortland, Walworth, or Ogle county.

The following procedure illustrates the calculations (CR_{1-5} = cost rate not identified, but based on data presented in Figure 1) for the direct cost:

$$\text{TDC}_{\text{Walworth}} = \text{CR}_1 \times 116.3 \times 2 \times 14 = \$3{,}256.40\ \text{CR}_1$$

$$\text{TDC}_{\text{Wayne}} = \text{CR}_2 \times 101.8 \times 2 \times 46 = \$9{,}365.60\ \text{CR}_2$$

$$\text{TDC}_{\text{Tompkins}} = \text{CR}_3 \times 100.9 \times 2 \times 63 = \$12{,}713.40\ \text{CR}_3$$

$$\text{TDC}_{\text{Ogle}} = \text{CR}_4 \times 79.0 \times 2 \times 8 = \$1{,}264.00\ \text{CR}_4$$

$$\text{TDC}_{\text{Lee}} = \text{CR}_5 \times 19.3 \times 2 \times 6 = \$231.60\ \text{CR}_5$$

$$\text{TDC}_{\text{All}} = \$920 \times 10^3$$

$$\text{ADC (\$/lane-km/average event)}_{\text{two-lane}} = \frac{920 \times 10^3}{13{,}415.5 \times 2 \times 1.61}$$
$$= \$21.5\ (\$34.3)$$

$$\text{ADC (\$/lane-km/average event)}_{\text{freeway}} = \$37.0\ (\$59.3)$$

Road Users' Savings

On the basis of the preceding two sections the direct benefits and direct costs per vehicle kilometer of travel after winter road maintenance operations for both two-lane highways and freeways are presented in Figures 5 (*top*) and 6 (*top*), respectively. Accordingly direct road users' savings (traffic accident severity reduction, travel time, and fuel consumption reduction) on both types of highways were generated and are presented in Figures 5 (*bottom*) and 6 (*bottom*). The author believes that these curves can be useful for future benefit-cost analyses and in determining road users' direct savings for areas that use similar winter road maintenance operations.

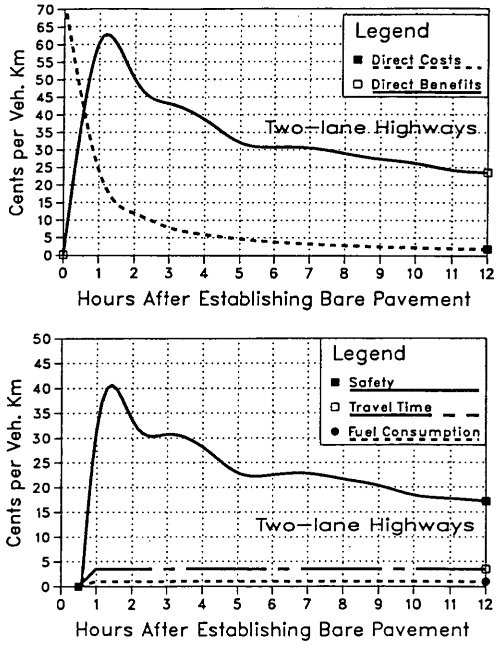


FIGURE 5 Direct benefits, costs *(top)*, and savings *(bottom)* of winter road maintenance operations on two-lane highways.

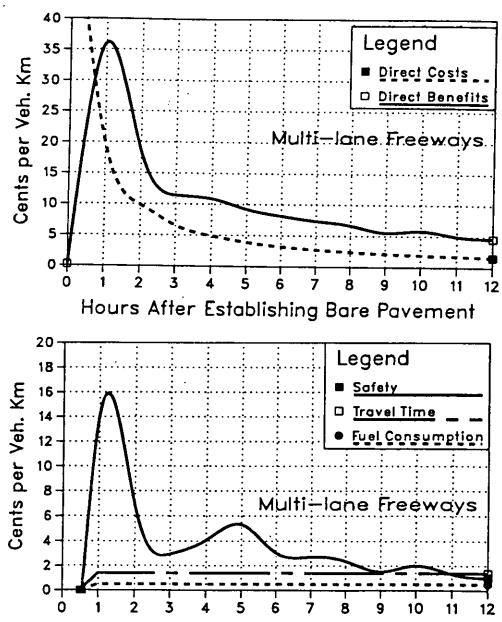


FIGURE 6 Direct benefits, costs *(top)*, and savings *(bottom)* of winter road maintenance operations on multilane divided freeways.

direct savings for areas that use similar winter road maintenance operations.

Benefit-Cost Ratio

On the basis of winter road maintenance benefits and costs that were calculated in the preceding three sections, the benefratio was calculated to determine how much the road users receive as savings (in dollars) for every dollar spent on winter road maintenance operations.

The benefit-cost ratio (*B*/*C*) of direct benefits and direct costs of winter road maintenance operations was calculated for each *Z* hr after establishing bare pavement (zero hour) on the basis of the following equation:

$$\left(\frac{B}{C}\right)_{\text{direct},Z} = \left(\frac{\text{\$ of winter road maintenance direct benefits}}{\text{\$ of winter road maintenance direct costs}}\right)_Z$$

The following example illustrates the procedure for (*B*/*C*) ratio calculations:

Road user safety savings = \$0.345/VKT

Fuel consumption savings = \$0.010/VKT

Travel time savings = \$0.036/VKT

Total VKT during the first 4 hr after = 15,682,250

Total winter road maintenance direct cost = $\$920 \times 10^3$

$$\left(\frac{B}{C}\right)_{\text{direct},4} = \left[\frac{(0.345 + 0.010 + 0.036) \times 15{,}682{,}250}{920 \times 10^3}\right]_4 = 6.50$$

and conclude that during the first 4 hr after establishing bare pavement (zero hour) the average road users' savings were \$6.50 for each \$1.00 spent on winter road maintenance operations.

RESULTS AND CONCLUSIONS

Before 1991 several extensive European studies (*15*) measured the impact of winter road maintenance operations (and its economic benefits) on road users' safety. In the United States no such study up to the date of the present research has documented how effective winter road maintenance operations are on road users' safety.

The maximum benefit of a policy for dealing with snow and ice problems would be to achieve a bare pavement condition as quickly as possible. The optimum snow and ice policy varied from one participating area to another, but they were all similar in establishing bare pavement as soon as possible.

The primary objective of the present study was to compare and evaluate the direct benefits and direct costs accrued through winter road maintenance operations (salting, plowing, and following a bare pavement policy) during snow and icy conditions on highways. It was possible to do a benefit-cost analysis on the basis of the direct benefits and the direct costs. The results and conclusions reached are as follows:

Two-Lane Undivided Highways

On the basis of a representative average case (Z = 4), the following conclusions were reached.

Conclusion One

Winter road maintenance operations on two-lane highways reduce traffic accident costs by 88 percent and the average traffic severity by 10 percent.

1. The traffic accident cost rate before was \$390,000/10^6 VKT (MVKT) [\$625,000/10^6 vehicle mi of travel (MVMT)];
2. The traffic accident cost rate after was \$46,000/MVKT (\$74,000/MVMT);
3. The average traffic accident severity before was \$49,000/accident; and
4. The average traffic accident severity after was \$44,000/accident.

Conclusion Two

Winter road maintenance operations on two-lane highways cause direct economic benefits to the users much more than direct economic costs:

1. During the first 4 hr after salt spreading (zero hour) the average road users' savings were \$6.50 for each \$1.00 spent on winter road maintenance operations in direct costs;
2. The average direct winter road maintenance costs are offset by the average direct winter road maintenance benefits as soon as 71 vehicles have driven over a deiced two-lane highway; and
3. The winter road maintenance service pays for itself within the first 25 min after establishing bare pavement.

Multilane Divided Freeways

On the basis of a representative average case ($Z = 2$) the following conclusions were reached.

Conclusion Three

Winter road maintenance operations on freeways reduce the traffic accidents cost by 85 percent and the average traffic severity by 30 percent.

1. The traffic accident cost rate before was \$196,000/MVKT (\$316,000/MVMT);
2. The traffic accident cost rate after was \$30,000 per MVKT (\$49,000/MVMT);
3. The average traffic accident severity before was \$57,500/accident; and
4. The average traffic accident severity after was \$40,500/accident.

Conclusion Four

Winter road maintenance operations on freeways cause direct economic benefits to the users two to three times the amount of the direct economic costs:

1. During the first 2 hr after salt spreading (zero hour) the average road users' savings were \$2.00 for each \$1.00 spent on winter road maintenance operations in direct costs;
2. The average direct winter maintenance costs are offset by the average direct winter road maintenance benefits as soon as 280 vehicles have driven over a deiced two-lane highway; and
3. The winter road maintenance service pays for itself within the first 35 min after establishing bare pavement.

Both Types of Highways

Conclusion Five

During snow and icy conditions the ratio of accident and cost rates on two-lane highways in comparison with those on multilane freeways are (a) approximately 2 times in the period prior to establishing bare pavement and (b) approximately 1.5 times in the period after establishing bare pavement.

Conclusion Six

Winter road maintenance operations reduce accident and cost rates from before to after establishing bare pavement by $\geq$80 percent.

Conclusion Seven

Traffic accidents severity is (a) greater on icy freeways than on icy two-lane highways by 17 percent and (b) lower on deiced freeways than on deiced two-lane highways by 8 percent.

Conclusion Eight

Winter road maintenance operations on highways cause direct savings to the users (Figures 5 and 6) in the after period. (a) The average direct savings to a two-lane highway user is 45 cents/vehicle-km of travel, and (b) the average direct savings to a multilane freeway user is 20 cents/vehicle-km of travel.

RECOMMENDATIONS

On the basis of the results and conclusions presented in this paper the author recommends the following:

1. The direct economic impact of different procedures and policies for winter road maintenance operations on road users should be studied and evaluated by using the methodology described in this paper.
2. More research on investigating the indirect economic impact of winter road maintenance operations on road users should be conducted.
3. More research on measuring the impact of different weather conditions on vehicle fuel consumption and air quality should be conducted.
4. A policy on proper computerization of winter road maintenance records, which will simplify and ease computations of the economic impacts, should be adapted.

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Comparison of Liquid and Solid Chemicals for Anti-Icing Applications on Pavements

R. G. ALGER, J. P. BECKWITH, AND E. E. ADAMS

Tests designed to assess the anti-icing properties of several chemicals that are potential candidates for winter highway maintenance were performed. Included in the test scenario were different application rates. Meteorological conditions were closely monitored to compare the efficacy of each chemical given a certain weather combination. The Saab friction tester was used to monitor friction levels on test sections to quantitatively monitor the effectiveness of each anti-icer. Statistical methods were used to correlate the collected data. The test also included observations of the handling properties of the chemicals, ease of application, and the effect of traffic on the sections, but these results are not provided. Overall anti-icing is effective, liquid chemicals were for the most part more effective than solids, and during this test, at low application rates, none of the chemicals appeared to be effective at temperatures below −6°C (21°F).

The use of chemicals to maintain safe operating conditions for vehicle operators and pedestrians is an important issue for winter maintenance crews worldwide. Cost and environmental concerns make it necessary to minimize the amount of chemical used and to look for alternatives to chemicals that may be harmful to the environment if used improperly.

In an attempt to optimize chemical usage a study was conducted at the Keweenaw Research Center in Houghton, Mich., under the Strategic Highway Research Program. The study was designed to assess the anti-icing properties of different liquid and solid chemicals in a field situation. To assess the efficacy of each chemical as an anti-icing agent, 12 chemicals were applied on 3.7-m (12-ft) by 30.5-m (100-ft) sections of asphalt prior to a forecast precipitation event. In several instances "different" chemicals implies that the same chemicals were applied at two application rates.

OBJECTIVES

The study was undertaken to determine the anti-icing properties of certain chemicals and to assess qualitatively some of the techniques used to successfully apply the different chemicals as anti-icers. This testing included observations of meteorological parameters before and during each test so that correlations to these variables could be made.

The concept behind anti-icing is that the chemical is applied to the pavement prior to the conditions for icing. This then inhibits the ice-to-pavement bond so that the overburden ice can be removed by mechanical means. Thus the chemical is not required to melt through the entire thickness of ice, as is the case for "deicing."

If anti-icing is successful a lesser quantity of a particular chemical can be applied to attain and sustain a safe operating condition. Since weather plays an important role in a successful anti-icing operation, significant weather parameters were monitored to correlate the parameters with chemical efficacy.

TEST SETUP

Thirteen scenarios were tested on a portion of asphalt airport runway located at the Houghton County Memorial Airport. Because of the heavy snowfall in the region this tarmac is not maintained for service during the winter months. All of the sections were asphalt. Twenty-seven successful tests were conducted during the winter of 1991 and 1992. The chemical applications are shown in Table 1.

Liquid chemicals were applied at a rate equivalent to 45 to 90 kg (100 or 200 lb) of dry (solid) chemical (calculated by using percent chemical per weight of solution) to 1.61 lane km (1 lane mi) (7.66 g/m^2 = 100 lb/lane mi).

The test sections were 3.6 m (12 ft) wide and 30.5 m (100 ft) long. Sections were separated by a 61-m (200-ft) buffer zone to help eliminate cross contamination between chemicals.

During a test the average friction value was used as the measure to indicate how well a treatment worked in achieving a safe level of tire-to-road surface friction. Friction measurements were taken every 15 min throughout the test with a Saab friction tester. This device consists of a fifth wheel on a car that measures friction by comparing torque and speed and recording it on an on-board computer. The data obtained give an average friction value over the

TABLE 1 Chemical Application Rates (100 #/l.m. = 100 lb/linear mi = 7.66 g/m^2)

Chemical	Rate
NaCl solid	100#/l.m. and 200#/l.m.
NaCl solid and $CaCl_2$ solid 5:1	100#/l.m.
NaCl solid and $CaCl_2$ liquid 10gal/ton	100#/l.m.
$MgCl_2$ liquid	100#/l.m.
CMA solid	100#/l.m. and 200#/l.m.
CMA liquid	100#/l.m. and 200#/l.m.
Potassium Acetate liquid	100#/l.m. and 200#/l.m.
Urea solid	100#/l.m.
Control	No chemicals

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30.5-m (100-ft) test section. Evaluations of whether a second application of chemicals or a snow removal operation was necessary were made at 2-hr intervals. This was accomplished by looking at the 15-min average Saab friction tester readings.

A statistical package was set up to analyze all of the measured data that were acquired during a test period. This package was developed to have each day's test results added to a large matrix as soon as a test was complete. For each day comparisons were made between treatments, taking into account the weather conditions for that day.

An even larger matrix was developed by adding the measured parameters from each test day to those from the previous day's tests. In this way a running comparison is made between the treatments for all previous tests, still taking into account the weather for each test day.

ANALYSIS

Perhaps the most important, quantifiable factor that can be used to determine how well each chemical performs is the friction value. If a test section (chemical) reaches some desired friction value during a test, the outcome of applying the chemical is at least partially successful. If the average friction over the test period remains above a given acceptable value, this is another indication that the chemical is performing with some success. In terms of maximum friction a chemical that brings the friction up to a value comparable to that for wet pavement is likely to be quite successful. Values of maximum friction less than that for wet pavement when compared with the average friction can also give some indication as to the performance of the chemical application.

To compare the relative efficacy of each of the chemicals, an analysis was performed by using the friction values obtained during each test and under varying environmental conditions. A Friedman analysis was used for each test by comparing the friction value at each 15-min time interval and ranking these values from 1 to 13. From these interval rankings a sum of ranks throughout a test period is made.

The Friedman test (*1*) is a nonparametric two-way analysis of variance test. It treats each test time within a given day as a block in which it ranks the friction coefficient values. The ranks of a treatment over that day are then summed and compared with those of the other treatments. The analysis is based on the hypothesis that there is no difference in the distributions, and it determines the probability that the hypothesis is correct. This test was repeated for each test day. All Friedman tests were performed to a level of significance of 0.05.

Once the sums have been obtained for each test individually, the ranking for the entire winter period can be obtained by simply ranking the results for each chemical for all of the tests performed.

During a single test many things can happen that designate how a chemical performed. The following are among the possible scenarios:

- Friction starts low and stays low.
- Friction starts low and increases throughout the test.
- Friction starts high and stays high.
- Friction starts high and decreases throughout the test.
- Friction starts low, increases, and then drops off toward the end of the test.
- Friction starts high, drops off, and then increases again.

Considering the many perturbations, the use of maximum or average friction may be deceiving. To examine the qualitative aspect of each test by some means beyond the average and maximum friction values, a method was devised to place a numerical value on the performance. This value, called *effect*, is designed to quantify subjectively the performance of a chemical throughout an entire test period. For instance a test section may have a friction value of 0.7 at the beginning of the test. Because the snow was falling or temperatures dropped considerably during the initial part of the test the friction may have dropped off rapidly. This section would have a high value for maximum and possibly average friction but may not have actually performed well overall. There is a possibility that any one section may rank either high or low with standards of friction, but the overall performance may not be reflected.

To give some quantitative basis to the perceived effectiveness, the performances of the chemicals as the combination of three independent contributions are considered. These are the time that it takes for the chemicals to start to work (A), the average friction obtained throughout a test (B), and how well each chemical sustained an acceptable value throughout the test period (C). All of the friction results were analyzed, and the final decision was made that 22 percent of the total ranking should be attributed to (A), 67 percent to (B) and 11 percent to (C). On the basis of that decision a set of equations was designed to determine an overall value for effectiveness. The range of minimum to maximum frictions that was used for the calculations was 0.1 to 0.8. The value of 0.1 is representative of what the Saab friction tester would produce on glare ice and 0.8 is representative of what it would produce on ice-free, wet pavement.

The chemicals were ranked for each test by the following equation:

$$\text{Effect} = A + B + C \tag{1}$$

whose coefficients are calculated as

$$A = \frac{30}{t_{0.5}} \tag{2}$$

$$B = (F_{avg} - 0.1) \cdot 8.57 \tag{3}$$

$$C = (F_{max} + F_{end}) \cdot 0.625 \tag{4}$$

where

$t_{0.5}$ = time at which Saab friction tester result becomes ≥ 0.5,
F_{avg} = average friction for test,
F_{max} = maximum friction attained during test, and
F_{end} = friction value at end of testing.

This method ranks the performance of the chemical over the duration of the test and results in a rating from 0 to 9, where 0 is very poor and 9 is excellent.

The coefficient A accounts for the time that it takes for the Saab friction tester to reach an acceptable value. A friction value in the range of 0.42 to 0.56 is considered "good" for verbal braking action when measured with the Saab friction tester (*2*). For the purposes of the tests described here the acceptable value was set at 0.5. All times are taken by assuming the elapsed time from the onset of precipitation if possible. Otherwise the test was started at the time that the chemicals were applied. The next Saab friction tester run after chemical application was at 15 min. If the friction value for a given test section has reached 0.5 at this point, then $t_{0.5}$ = 15 and A = 2. If the friction never reaches 0.5, then $t_{0.5}$ tends toward infinity and this value goes to 0. If the friction value does not reach 0.5 during a test, then A is set to 0.

The coefficient B accounts for the average friction throughout the test. Since maintaining the average friction over the period of the test at or above the desired value of 0.5 is considered to be the most important factor of chemical application, this portion of the ranking is given the most weight. For example if the average friction for the entire test was 0.8, which is an expected value for ice-free wet pavement, the value for this coefficient would be 6. If average friction is 0.1, which is representative of black ice, this portion goes to 0.

The coefficient C determines how well each chemical performed as far as attaining and maintaining higher friction throughout the test. In this case if the friction value increased to 0.8 and was still at 0.8 at the end of testing, the contribution to the effect would be 1. If both average and ending friction are 0.1, the contribution to the effect would be 0.1, nearly 0. All other combinations range between 0 and 1.

In summary the "effect" equation (Equation 1) is the sum of the three coefficients and has a maximum value of 9 and a minimum value of 0. These equations were developed by weighting the relative importance of each of the three friction considerations described and assigning a value to them. The effect was calculated for each chemical and each test and was then corroborated with the visual assessment of the friction plots to ensure that the outcome was reasonable in every case.

A simple example may clarify this ranking scheme. Consider the curves in Figure 1. These example curves are developed by the use of contrived datum points chosen to illustrate six possible combinations for friction plots. Table 2 gives the result of the effect calculation for each of these six curves by using Equation 1. The curves are designated C1, C2, C3, C4, C5, and C6 both in Figure 1 and in Table 2. From Table 2 it can be seen that effect can vary considerably depending on the changes in friction throughout a test.

TABLE 2 Effect Calculations

Data Set	$t_{0.5}$	F_{avg}	F_{max}	F_{end}	A	B	C	Effect
C1	30	0.50	0.80	0.10	1.0	3.4	0.6	5.0
C2	∞	0.30	0.30	0.30	0.0	1.7	0.4	2.1
C3	15	0.74	0.80	0.80	2.0	5.5	1.0	8.5
C4	15	0.31	0.80	0.10	2.0	1.8	0.6	4.4
C5	15	0.40	0.80	0.80	2.0	2.6	1.0	5.6
C6	75	0.40	0.80	0.80	0.4	2.6	1.0	4.0

To make use of the calculated effect, a value that was deemed acceptable was determined. To accomplish this a hypothetical test was chosen to estimate an outcome from chemical application that would be acceptable to highway users. In this test the friction starts out at a value of 0.1 and comes up to a value of 0.5 at a time of 45 min. The friction then remains at 0.5 throughout the rest of the test period. The calculated effect for this scenario is 4.0. This value has been chosen to depict the acceptable effect for a test.

RESULTS

The Friedman analysis was first performed on the average friction data for each test day. These rankings were then analyzed over the entire winter test period by use of a second Friedman analysis. Figure 2 is the result for this ranking for the 27 tests. The relative ranking of the 13 treatments is given on the top of Figure 2 along with the value for the sum of ranks. The larger the sum the better the overall performance of the chemical application. These values increase across the plots from left to right. The bars on the graph depict groups of chemicals that cannot be statistically distinguished from one another, that is, those chemicals whose performance does not appear to be significantly different from those of the others connected by the same bar.

Figure 3 contains the results for the effect calculations for all of the 27 test days.

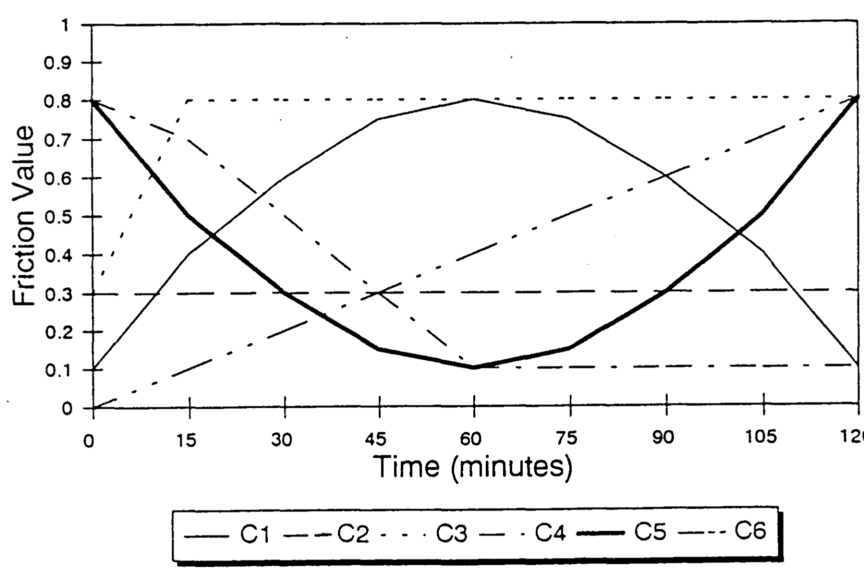

FIGURE 1 Example friction plots (time 0 = chemical application).

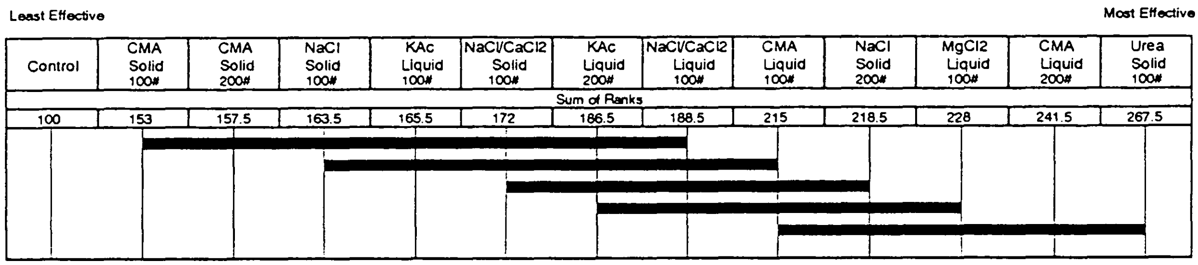

FIGURE 2 Friedman analysis, sum of ranks, All tests (27 cases) (# = lb = 0.45 kg).

After calculating the effect values for each chemical for all of the tests run during the winter period, the Friedman analysis was performed on these results. Figure 4 gives the results of these calculations for the 27 tests.

The data were further analyzed to identify any correlations between the weather parameters and the performance of the chemicals. The statistical significance of any correlations present was tested by using the Pearson and Bonferroni methods (*1*). From these tests it can be determined if the outcome of a test is statistically dependent on a given meteorological parameter.

Examination of all of these sets of data revealed that in no instance did the effect achieve an acceptable value of 4 (discussed earlier) when the pavement temperature was below −6°C (21°F). Realizing this the Friedman analysis was performed for the tests when the pavement temperature was above −6°C (21°F). This was done for both friction and effect, and the results are given in Figures 5 and 6. During 21 tests the pavement temperature was above −6°C (21°F).

Pavement temperature was the only meteorological parameter that showed a relationship to the outcome of the anti-icing procedure. Wind and ambient air temperature are two other parameters that have been shown to affect chemical efficacy (*3*). These two parameters were eliminated by the nature of testing, since no tests were conducted during periods of high wind and extremely cold temperatures.

The effect values were also analyzed in terms of the three components *A*, *B*, and *C* in an attempt to assess the chemicals in the three separate categories.

Figure 7 shows the result of the Friedman analysis for the *A* values, which should give a good indication of how fast each chemical begins to work. This result is for all of the 27 tests performed.

The same analysis was performed for cases in which the pavement temperature was at or above −6°C (21°F). The graph is not given to avoid redundancy. In both cases the order that each chemical performed in comparison with the others was identical. For the most part the liquids were faster than the solids when the *A* value was used as the indicator.

Figure 8 shows the result for the 27 cases for the *B* values.

The *B* value should give a good indication of how effective each chemical is at attaining an acceptable value of average friction. The analysis was also performed on the 21 cases. There were some subtle differences between the 27- and 21-test analyses. For the most part the liquids were again superior to the solids. The most important difference between the two results was that urea fell behind liquid CMA at 7.66 g/m^2 (100 lb/linear mi) and potassium acetate (KAc) at 15.32 g/m^2 (200 lb/linear mi) when the temperature was brought up to −6°C (21°F). This indicates that the urea may perform better than some of the other chemicals at the colder temperatures.

DATE	CONTROL	CMA 100	CMA 200	KAc 100	KAc 200	MgCl2 100	NaCl 200	NaCl/CaCl2 100 Liq	NaCl/CaCl2 100 Liq	Solid CMA 100	Solid CMA 200	Urea 100
12/12/91	3.8	9.2	9.0	9.1	9.0	9.1	8.7	8.5	9.1	8.7	8.9	8.9
12/13/91	4.9	5.2	5.0	5.0	5.3	5.1	5.0	4.4	4.9	4.9	4.5	4.8
12/20/91	2.3	2.4	2.7	2.2	2.3	2.9	2.0	2.0	1.9	2.3	2.3	2.0
01/02/92	3.3	8.7	8.8	8.6	8.7	8.8	6.0	6.7	5.6	6.2	5.7	6.2
01/06/92	6.8	8.0	6.6	6.2	5.6	5.9	5.9	5.4	5.0	4.9	4.9	4.1
01/08/92	4.2	4.7	5.7	4.6	5.6	5.4	4.6	4.7	4.5	4.7	4.6	4.5
01/09/92	4.5	2.6	2.6	2.7	2.8	3.4	2.6	2.6	2.7	2.6	2.6	2.1
01/10/92	5.5	4.7	4.9	2.1	1.8	4.6	2.3	2.2	2.2	2.5	2.1	2.4
01/11/92	3.2	6.9	8.7	8.3	6.2	8.7	5.1	6.7	5.4	5.9	4.9	6.0
01/22/92	2.2	4.6	4.7	2.6	3.0	3.4	2.6	3.6	2.4	2.7	2.5	2.6
01/23/92	1.6	2.6	4.2	3.1	4.5	4.6	2.5	2.6	2.6	2.8	2.7	2.6
01/27/92	2.1	3.4	3.7	3.4	3.2	3.5	3.1	3.9	2.9	2.4	1.9	2.8
01/28/92	1.5	2.7	2.5	2.1	2.4	2.5	2.1	2.3	2.1	2.2	2.3	2.4
01/30/92	1.7	5.1	5.3	4.1	4.8	4.9	4.4	6.7	5.6	6.4	6.1	6.4
01/31/92	1.7	2.4	3.1	2.4	3.2	3.7	2.6	3.2	2.6	2.9	2.8	3.2
02/06/92	2.0	2.4	2.4	2.7	2.7	2.4	2.7	2.9	2.2	2.2	2.1	2.2
02/10/92	1.8	2.0	2.8	2.3	2.4	2.5	2.4	2.5	2.5	2.1	1.7	1.7
02/11/92	2.1	1.8	2.0	2.2	2.1	2.1	2.4	2.2	2.2	2.1	2.1	1.9
02/12/92	1.9	2.1	2.4	2.1	2.0	2.3	2.0	2.3	2.2	2.3	2.3	2.5
02/13/92	1.5	2.6	4.3	1.9	3.0	4.4	2.3	2.3	2.2	2.0	2.1	2.2
02/14/92	1.7	7.5	8.7	4.2	8.3	8.8	8.6	8.8	7.1	5.2	6.4	8.6
02/20/92	1.7	4.7	4.4	4.4	4.0	3.6	1.8	4.3	4.3	1.6	1.7	3.7
02/24/92	2.1	6.2	7.1	5.4	6.5	7.4	5.1	5.6	4.8	5.2	4.7	5.0
02/25/92	3.4	7.5	8.5	6.5	7.5	8.5	5.6	6.2	6.2	6.2	5.8	6.7
02/27/92	6.6	6.6	7.6	6.6	7.5	8.9	7.9	8.0	8.8	8.0	8.8	8.7
03/09/92	1.8	2.4	1.9	2.4	2.1	2.7	2.5	2.2	2.6	2.6	2.7	2.3
04/01/92	4.2	2.5	4.0	2.5	5.6	5.7	2.4	5.0	2.6	2.8	2.7	2.6

FIGURE 3 Effect values for all tests (27 cases).

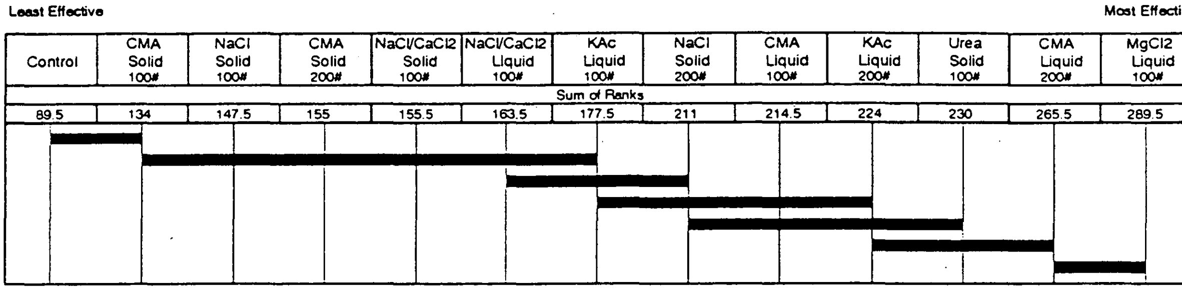

FIGURE 4 Friedman analysis, effect values, all tests (27 cases).

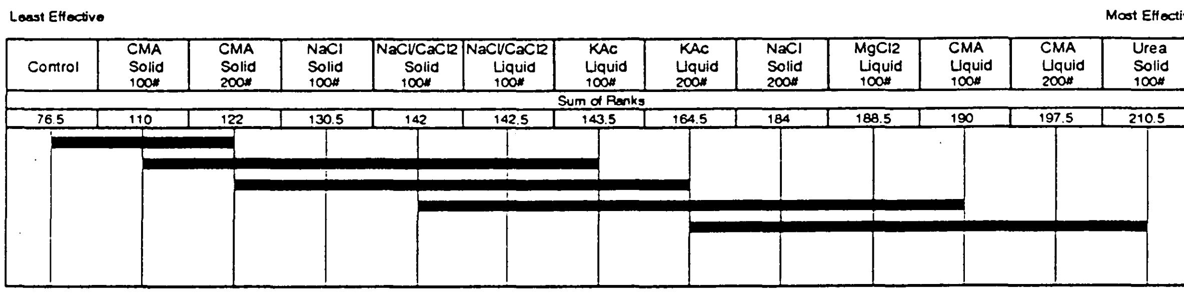

FIGURE 5 Friedman analysis, sum of ranks, pavement temperature above −6°C (27 cases).

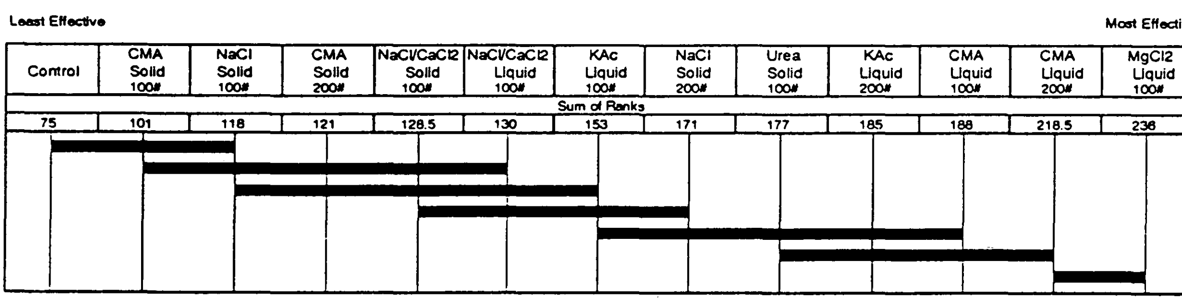

FIGURE 6 Friedman analysis, effect values, pavement temperature above −6°C (21 cases).

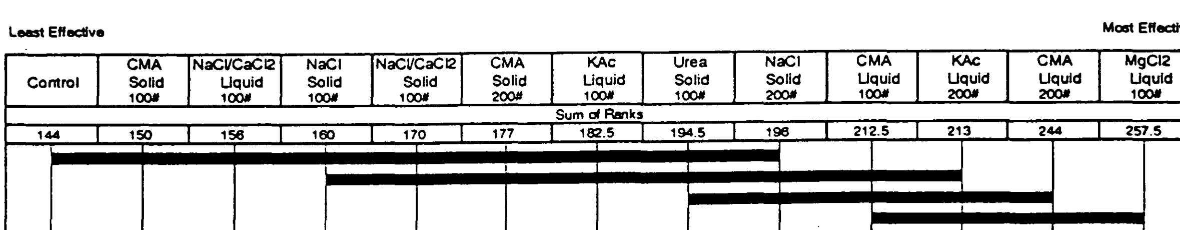

FIGURE 7 Friedman results, *A* values, all tests (27 cases).

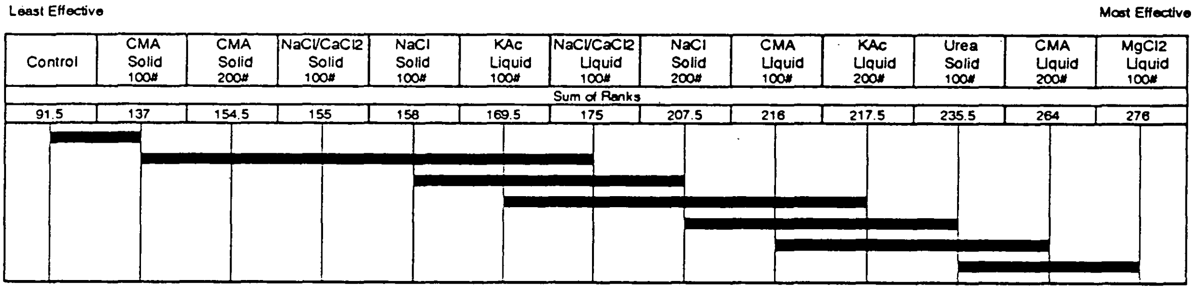

FIGURE 8 Friedman results, *B* values, all tests (27 cases).

Finally the result of examination of the *C* values by use of the Friedman analysis is shown in Figure 9 for the 27 cases. The *C* value should give an indication as to how well the chemicals can maintain a higher friction value with time.

The liquids, with the exception of the 15.32-g/m^2 (200 lb/linear mi) application of NaCl, were once again grouped as the better performers. This indicates, as is probably already well known, that salt tends to maintain quality on the pavement at least at the higher dosage.

The final analysis of the test data was performed on test days when two applications of chemical were deemed necessary to attempt to reach the desired level of service. The use of freezing point depressants for anti-icing is effective; however, in the present study the situation in which the snow that had fallen on the chemically treated asphalt pavement was simply peeled off to clear the pavement was not always observed. What did occur is that in many instances, when the chemicals were placed on the bare pavement, the majority of the snow was easily removed, but a thin film or layer of slush-like snow that could not be removed with a plow blade remained. The influence of the chemicals was visually apparent in these instances, in that the snow was much thinner than that on the untreated sections or the buffer zones and the treated sections were translucent and sometimes appeared to be bare. However in those instances when a low friction was recorded, close inspection revealed that a very thin ice layer was present. In those cases even power sweeping did not in general provide dramatic improvement, but depending on the pavement temperature another application of chemical was effective in removing the snow, particularly for the case of the liquids. Of the 27 tests run during the winter, 7 had a second application after 2 hr.

Since only seven test periods of data were analyzed care should be taken when drawing conclusions from these results.

Figure 10 shows the result of the Friedman analysis for the first 2 hr of the seven tests, or generally the anti-icing portion. From Figure 10, it can be seen that the relative order of effectiveness of the chemicals changed somewhat, but most importantly that it is difficult to separate the chemicals from each other, as indicated by the fact that only three bars exist and the overlap between them is substantial. This result should be obvious since the reason for applying chemicals a second time is that the first application was for the most part unsuccessful.

Figure 11 is the Friedman graph for the second application of chemicals. This result is not as expected because all of the liquids, with the exception of potassium acetate at 7.66 g/m^2 (100 lb/linear mi) performed better than the solids. This does not agree with the conception that liquids do not perform as well as solids as deicers. It is possible that the anti-icing effect of the liquids was adequate to get favorable results from a second application. Keep in mind, however, that only seven sets of data were available for this analysis.

OVERVIEW

As a general conclusion the use of liquids in an anti-icing program is superior to the use of dry chemicals (solids). This is certainly the case for the CMA liquid versus CMA solid, the only one of the chemicals for which there was a direct comparison. The one exception to this was the dry urea. This ranking of the urea is somewhat surprising since visually it did not appear to perform as well as the liquids.

Visually the liquids produced a more even pattern of anti-icing. This is likely due, at least in part, to the fact that they were not as easily displaced as the solids once they were applied to the bare pavement. When the liquids were placed on the thin film of

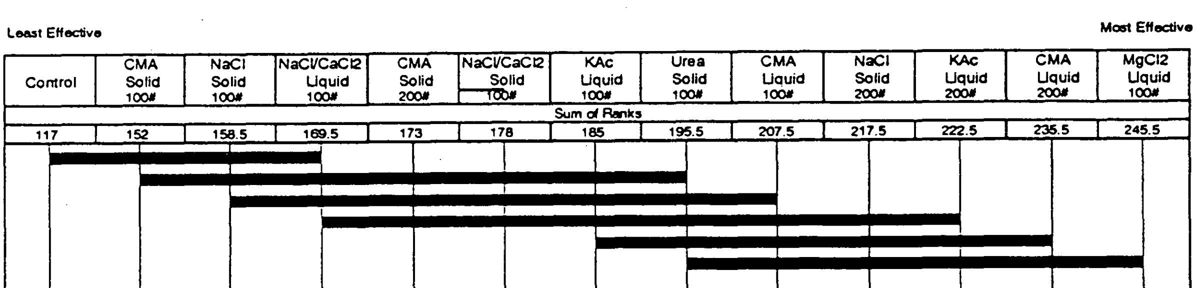

FIGURE 9 Friedman results, *C* values, all tests (27 cases).

Least Effective | Most Effective

CMA Solid 200#	KAc Liquid 100#	NaCl Solid 100#	NaCl/CaCl2 Solid 100#	Control	CMA Solid 100#	NaCl Solid 200#	KAc Liquid 200#	NaCl/CaCl2 Liquid 100#	CMA Liquid 100#	MgCl2 Liquid 100#	CMA Liquid 200#	Urea Solid 100#
Sum of Ranks												
21.5	39	40.5	41	43	45	48	51.5	53	57.5	62.5	64	72.5

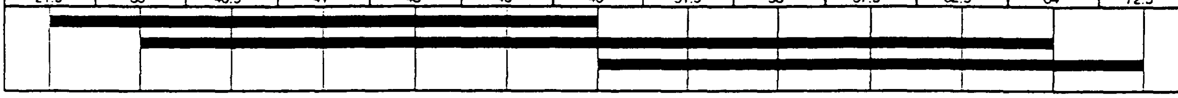

FIGURE 10 Friedman results, double application, first 2 hr (seven cases).

Least Effective | Most Effective

Control	CMA Solid 200#	NaCl/CaCl2 Solid 100#	CMA Solid 100#	KAc Liquid 100#	NaCl/CaCl2 Liquid 100#	NaCl Solid 100#	NaCl Solid 200#	Urea Solid 100#	CMA Liquid 100#	CMA Liquid 200#	KAc Liquid 200#	MgCl2 Liquid 100#
Sum of Ranks												
19.5	33	35	38	40.5	43	44.5	50	53.5	63	63	70.5	83.5

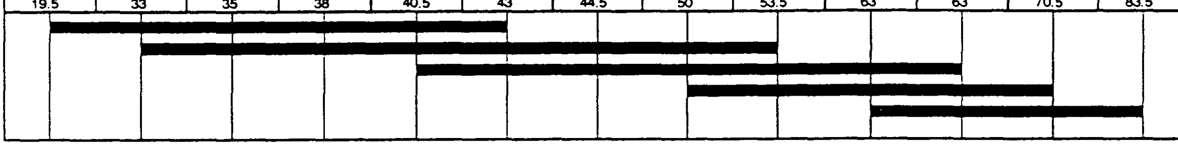

FIGURE 11 Friedman results, double application, second 2 hr (seven cases).

slush or ice as described above they were much more effective in removing this thin layer. When considering deicing such as this the liquids were much faster; however, if there is a substantial mat of snow the liquids were not necessarily as effective as the solid chemicals. The solid chemicals then appeared to have the advantage of burrowing and not becoming dilute as quickly, thus providing a better chance of exposure to the pavement, although at the rates at which they were applied, deicing under such conditions was generally ineffective.

In a review of all of the analyses performed on the data collected during 1991 and 1992 some interesting trends have evolved. Figure 12 is a tabulation of the Friedman results for several different scenarios. The shaded boxes around the chemical names are included to signify the liquid chemicals and to make it easier to differentiate them from the solids.

In summary:

- Liquids are generally more effective than solid chemicals for anti-icing.
- A thin layer often remains after snow that has accumulated on top of the treated pavement but that is effectively treated with

All Effects (27 Cases)	T1	T11	T7	T12	T9	T10	T4	T8	T2	T5	T13	T3	T6
Effects for 21 Cases, Pave. Temp.>+ -6C	T1	T11	T7	T12	T9	T10	T4	T8	T13	T5	T2	T3	T6
"A" Effect Values, 27 Cases	T1	T11	T10	T7	T9	T12	T4	T13	T8	T2	T5	T3	T6
"A" Effect Values, 21 Cases	T1	T11	T10	T7	T9	T12	T4	T13	T8	T2	T5	T3	T6
"B" Effect Values, 27 Cases	T1	T11	T12	T9	T7	T4	T10	T8	T2	T5	T13	T3	T6
"B" Effect Values, 21 Cases	T1	T11	T12	T9	T7	T10	T4	T8	T13	T5	T2	T3	T6
"C" Effect Values, 27 Cases	T1	T11	T7	T10	T12	T9	T4	T13	T2	T8	T5	T3	T6
"C" Effect Values, 21 Cases	T1	T11	T7	T12	T10	T9	T13	T4	T2	T5	T8	T6	T3
2-Test Days (7 Cases), Both Tests Included	T12	T1	T9	T4	T11	T7	T8	T10	T2	T5	T13	T3	T6
2-Test Days (7 Cases), 1st Test	T12	T4	T7	T9	T1	T11	T8	T5	T10	T2	T6	T3	T13
2-Test Days (7 Cases), 2nd Test	T1	T12	T9	T11	T4	T10	T7	T8	T13	T2	T3	T5	T6

Denotes Liquid Chemicals

T1 = CONTROL
T2 = CMA, LIQUID, 100#
T3 = CMA, LIQUID, 200#
T4 = KAc, LIQUID, 100#
T5 = KAc, LIQUID, 200#
T6 = MgCl2, LIQUID, 100#
T7 = NaCl, SOLID, 100#
T8 = NaCl, SOLID, 200#
T9 = NaCl/CaCl2, SOLID, 100#
T10 = NaCl/CaCl2, LIQUID, 100#
T11 = CMA, SOLID, 100#
T12 = CMA, SOLID, 200#
T13 = UREA, SOLID, 100#

FIGURE 12 Overall friedman results in increasing order of effectiveness for each set of tests.

another light application of chemicals has been plowed off. In this instance the liquids were superior.

- Solid chemicals are probably better for use in attempting to remove a thick mat but would require a larger amount of chemical.
- None of the chemicals tested was effective at a pavement temperature below −6°C (21°F).

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