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Maintenance Management, The Federal Role, Unionization, Pavement Maintenance, and Ice Control

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A State Government Views a National Problem

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Maintenance of a completely integrated mass transportation system includes work on highways, railroads, parking facilities, terminals, stations, and park-and-ride facilities. People, industries, and commercial establishments are moving from highly urbanized areas to suburbia and usually relocate along existing transportation corridors. Use of mass transportation facilities is encouraged to minimize the tremendous congestion on highways. Maintenance of related facilities to meet this requirement must be accomplished by using public funds in the form of federal financial aid. Without this assistance, high fares will force people to use the already overcrowded highways.

The federal government, state governments, local governments, public authorities, and other jurisdictions have spent large sums of money constructing a major highway network and providing large improved mass transit systems. The Interstate Highway Program alone was responsible for thousands of kilometers of major highways, and, while this was being accomplished, other agencies were building toll roads, freeways, and local feeder roads and were upgrading the existing highway network (by dualization, widening, and the like). This generally resulted in a large disbursement of people from the inner city to sprawling suburbs. At the same time, major commercial establishments and employers remained in the city or went to suburban areas along established transportation corridors. For instance, a business in the city that is not thriving or finds that the majority of its potential workers or customers or both are located in suburbia will most likely relocate to some place that is convenient for its workers and customers. This relocation then will be close to existing transportation facilities. Department stores have either moved or established branches in scattered geographical areas of suburbia, which has created a heavier dependence on the automobile. The result is the need for people movers from home to work that are both economical and efficient.

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Privately owned bus and rail facilities are now becoming the mainstay of transportation in metropolitan areas because the public has been encouraged by all levels of government and the communication media to use mass transit. Railroads were not making money on the passenger business but rather were dependent on freight tariffs. The need for passenger service actually placed a burden on the railroads, and government subsidies were necessary to keep the service available at an economical rate. The railroads and bus companies were caught in the nation's economic spiral of rising costs and the need for increased and extended service while keeping the fare down. Subsidies were needed to keep the mass transit system alive; otherwise, the highways would become overburdened to the point where they might become one big parking lot.

Mass transit can work only if people have access to the facilities, which requires adequate parking facilities strategically located and necessary feeder roads to them. There are many ways to provide this system, one of which is to use the existing roadway network and build parking areas convenient to both the neighborhood and the mass transit facility. Another method is to use existing (or about to be constructed) parking lots at commercial sites to serve the commuter. These spaces could be used during the normal workday when they are not really needed and can provide a side benefit to the commercial establishment—visibility to customers.

What does all of this have to do with transportation maintenance? This complete network now functioning to move people and products either through the individual vehicle or the mass transit unit needs to be kept operational 24 h/day, 365 days/year. Operations of all modes must be maintained. Operation and maintenance of the vehicles have been the responsibility of the operator even though some units are publicly owned and privately operated. Physical plant structures and vehicles can be maintained by the operator. Maintenance of parking lot, highway, and rail facilities can be accomplished by either the operator or a public agency.

Railroads already have the maintenance of rail facilities and component systems established and functioning. Training new personnel to perform this service would be both time consuming and costly; therefore, this maintenance is most likely to remain with the rail operators. Parking lot and highway maintenance are not familiar to the bus company or not a part of rail maintenance and therefore would be in another maintenance system. All states have the highway maintenance expertise and management systems to maintain highway and parking lot facilities. A situation arises when assigning priorities of maintenance, and generally parking lots would receive low priorities in both labor and fiscal matters. Many states are now experiencing financial problems and cannot afford to maintain their highways to normal standards and will probably be forced to reduce maintenance on parking lots to a bare minimum or to zero maintenance. A partial solution is to have the park-and-ride facilities in commercial lots that are maintained by merchants if possible. The rest can be maintained by a local municipality, authority, or rail owner through revenues collected from parking fees. It should be possible to accomplish this total maintenance activity by using the procedure just mentioned.

One ingredient is still missing—the financing to accomplish this maintenance activity and provide the necessary economical service. Federal funding was necessary to construct our current total transportation system, and, if the investment is to be preserved, federal assistance is essential.

Highway maintenance is a growing industry because of the last 2 decades of intense highway construction. Many states have explored, with very poor results, several avenues of funding for the continued construction of the Interstate and freeway systems. Voters have rejected bond issues consistently, and legislators have failed in their efforts to raise revenue through increased or new taxes. The result is a trend away from building highways on new alignments to upgrading existing highways through general improvements and widening. There is a dependence on high levels of highway maintenance service to maximize the effective flow of traffic on existing highways. This is becoming an impossibility as most states are forced to reduce their budget dollar per lane kilometer each year as a result of inadequate funding, and they are now accepting a lower level of maintenance service. Lower levels of highway maintenance service generally result in higher decay rates for bridge and pavement structures, which, in turn, because of premature failure, places these structures in the priority need category for reconstruction at an early date. An adequately funded preventive maintenance program could postpone the reconstruction for many years and result in a net savings to the government and ultimately the taxpayer. A total program based on a priority repair and reconstruction engineering analysis could maximize the life span of a highway while minimizing overall cost. This program can only be effective if funds are made available immediately. New Jersey has a good maintenance management system, as do most states, and the hope is that federal funding will be coming for highway maintenance operations that use reporting data accumulated from these maintenance management systems. Reimbursement for maintenance operations would be most effective if the federal government would accept the management system of the state involved for the program and reporting tool.

Mass transit facilities should be maintained by money (federal money, one hopes) directed to a particular activity, such as parking lot maintenance or rail maintenance. This funding could then be used to reduce or eliminate the parking fare, or the portion of the riding fare related to maintenance. An example would be a parking lot that costs approximately $100,000/year to maintain and charges 50 cents/car/day for its use. If this lot were to receive $50,000 in federal maintenance funds, it would allow the fare to be reduced to 25 cents/car/day. This reduction would then probably make the mass transit facility more attractive and remove some of the vehicles from the highway, indirectly saving maintenance dollars and some critical energy dollars.

Basically, the problem is viewed as being one of financial responsibility to maintain a complex network of transportation modes that was built or upgraded with federal assistance to meet a people and cargo movement demand. I feel that the federal government must provide this financial assistance now.

Federal Aid for Maintenance and an All-Modes Management System

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Recent years have seen suspension of commercial flights because of runway deterioration, restricted operating speeds because of deteriorating railroad tracks and roadbeds, questions about the effects of reduced service levels on structural investment and safety, and increased pressure to optimize use of limited resources. It has been suggested that federal aid for maintenance and all-modes management systems may hold some solutions. This paper deals with those issues. The federal aid for maintenance alternatives posed by some is supported but only if it comes in the form of a highway development and operations block grant to each state. Expansion of the categorical grant approach will result in the addition of cumbersome and costly procedures and provide no real benefit. After enumerating several approaches, the paper suggests that an all-modes maintenance management system would operate most effectively as a joint decision-making process by modal administrators with guidance and assistance from the U.S. Department of Transportation.

A few years ago, commercial flights into the Santa Fe Airport were suspended because of the deterioration of the runways. The clear inference was that maintenance was inadequate. We are hearing ever more frequently that the deteriorating condition of railroad tracks and roadbeds is resulting in restricted operating speeds and raising questions about safety. With the current financial restrictions on highway agencies, there are questions about how much levels of service can be reduced before safety and structural investment are critically sacrificed.

Transportation agencies must be interested in preserving essential elements and providing safety and efficiency for all modes of transportation for which they have responsibility. They must be interested also in planning and carrying out maintenance while optimally using the limited resources available to them. A logical question that has been posed is whether an all-modes maintenance management system should be developed by departments of transportation in the interests of attaining adequate across-the-board maintenance.

A number of years ago there was a drive for establishing "unit" management for local roads. This was aimed at overcoming the inefficiencies that presumably exist where a county is split into commissioner or supervisor districts or where there are township road authorities in addition to the county authority. Considerable progress was made in getting changes to unit management, but there was a long way to go and things seemed to stabilize about 20 years ago. I had occasion then to review the local road management situation in Kansas, which was a state in which there were still many county and township management authorities. One would presume after analyzing the situation that I could develop information that would provide stimulus to consolidation of management responsibility. The state highway department helped me select two sets of side-by-side counties. One county in each set had unit management and the other had county and township management. I made inspections of a large sample of roads in all counties and reviewed the inventory and financial record data. I fully expected to develop a strong case for unit management.

As it turned out, either the case was not to be made or my two-set sample was not representative. One set of adjacent counties had little comparability in terms of terrain and farming economy. It provided only a demonstration of disproportionately great needs for improvement in one county created by terrain, drainage problems, and soil conditions. The other set of counties appeared to be a good comparison. Both had prosperous agricultural economies. Both had good professional county engineers. In one county, of course, the engineer was responsible only for county (not township) roads. In the other county, the engineer had responsibility for all local roads. The overall road expenditures for construction and maintenance in the two counties were comparable. Inventory records for the two counties, supported by field inspections, showed surprising differences. The unit management had provided an almost uniform level of improvement and maintenance throughout the total road system. The roads were gravel surfaced and well maintained. The county-township-management county had a range in types of roads and levels of maintenance. The principal county arterials were hard-surfaced with asphalt concrete. The remainder of the county arterials had well-maintained gravel surfaces comparable to the roads in the unit management county. The extensive

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length of hard-surfaced arterials represented the significant difference between the county arterials in the two counties.

There was another big, and probably just as important, difference. The township roads, in the county-township management county, included some improved gravel roads but, what was most significant, a large proportion of the township roads were unimproved and apparently received little or no maintenance. These were roads that served no essential year-round service. They either provided alternates to other improved routes or simply served as field access roads for farming operations. Many of them were simply a pair of wheel tracks (classified as primitive roads in inventory records).

Although I recognize the limited nature of my study, the road users and the county economy appeared to be better served in the county-township than in the unit management county. Major routes provided better service. Much less money was spent on roads of little or no importance.

Three things were demonstrated in this study.

1. Unit management does not guarantee the most economic development and maintenance of road facilities.
2. There may be more incentive for discriminating decisions where local taxes and conscientious local managers are responsible for local roads.
3. Levels of maintenance should be established to fit service needs and not be applied uniformly to a road system in which there is great variability in traffic service.

FEDERAL AID FOR MAINTENANCE

It has been my long- and strongly held conviction that providing federal aid for maintenance would be a tragedy, that it would relegate state agencies to branch offices of the federal government and stultify innovative efforts by state agencies. In looking at the situation today, I am not so positive about the results of federal aid for maintenance. It really depends on whether the historic "strings-attached" federal aid for highways procedure is followed or whether the objective of providing financial assistance for maintenance is effected by converting federal aid for highways into a block grant to states to be used for highway development and operations. This would represent a drastic change from the existing categorical grant approach, but there are indications that the climate may be right for such a change. The Comptroller General, in his report to congress on August 19, 1975, pointed out the need for fundamental changes in federal assistance to state and local governments.

Highway officials are very much aware of the problem associated with effective management of federal-aid programs split into many categories. The executive arm of the federal government has recommended a sharp reduction in federal highway aid categories. The general public is concerned that the proliferation of federal programs has resulted in cumbersome and costly procedures and administrative practices.

Insofar as federal aid to state and local governments for transportation is concerned, why should we not have a block grant? The advocates of categorical grants have implied that such grants are necessary to direct resources to urgently needed and not adequately recognized programs. The bridge programs and numerous safety programs are examples. The fallacy in this approach, however, is the assumption that decisions on needed emphasis can be effectively made on a national basis. There are just too many variations from state to state and community to community. Furthermore, and of greatest importance, the controlling nature of the categorical program grants discourages states and local governments from pursuing more rational and beneficial programs.

I suggest that the aims of current federal-aid programs can be obtained under a block grant system with requirements for establishment of cost-effective planning and programming techniques by grantee agencies receiving grants. I think such techniques are the only answer to the effecting of real economies in government, and, at the same time, to meeting the development and operating requirements for safe and efficient transportation. The federal responsibility will then be to review and approve planning and programming processes and to foster research and development directed toward improved management. They will not be responsible for reviewing programs and projects whether they be for transportation development or operations.

ALL-MODES MAINTENANCE MANAGEMENT

There are some instances now where maintenance organizations have multimodal responsibilities. The Highway Division of the Oregon Department of Transportation maintains state airport facilities. The Wayne County Road Commission in Michigan operates and maintains the Metropolitan Detroit Airport. It is significant, however, that, because of the specialized nature of much of the airport facilities, the greater part of the maintenance is done by personnel always assigned to the airport. In other words, even though there is an agencywide maintenance management system for the county, only a limited amount of work activities is done by crews that work both on roads and airport. It is worth noting further that the Wayne County maintenance management system covers county parks, sewers, and buildings as well as the airport and roads.

All-modes maintenance should be considered in two parts: (a) the planning function and (b) the operating (organizing, staffing, and directing) function. In carrying out the planning function (setting levels of service, establishing work methods and staffing patterns, and allocating resources through the budget process) there should be a consistency between programs for the different modes. It should be possible to evaluate the programs to ensure that cost-effectiveness achievements are comparable from mode to mode. This does not mean that the U.S. Department of Transportation (DOT) should do the planning. If we are to have modal administrations, the individual modal administrations should perform the planning but in ways consistent from mode to mode.

Should the transportation agency, as distinguished from the modal administrations, decide how the department will be organized, staffed, and directed to accomplish the planned objectives? In other words, should we establish maintenance operations as a department function, or should the resources be allocated to the modal administrations and the carrying out of programs be left totally as their responsibility? Or shall there be joint decision making by modal administrators with guidance and assistance from DOT? This strikes me as the logical approach. There are some maintenance activities that are common to two or more modes. And, in some instances, these might be performed more efficiently by the same crews. Some work might be effectively contracted out for all administrations.

The important thing is that there be joint decision making on organizations, staffing, and directing. A task force might be established to accomplish this under a steering committee with representatives from all modes and DOT.

Federal Role in Supporting Research and Development for Reducing Transportation Maintenance Costs

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The 1975 policy statement of the Secretary of Transportation reflects that, for the most part, the nation's transportation infrastructure is in place. What is needed now is the modernization, repair, and more effective use of existing capacity. The cost of maintaining the transportation system is of concern, and much is being done in all sectors to cope with the situation. Inefficient maintenance practices, the need for more reliable and maintainable equipment, the need to eliminate or reduce manual labor, and the need for better information systems are being addressed. Solutions take time to implement and are expensive in most cases. Although the findings in this report do not show that a crisis situation exists or is imminent, they do suggest opportunities for the U.S. Department of Transportation to expedite the changes needed to significantly reduce future maintenance costs. Four major areas have been identified in which common needs exist for all transportation modes, in government as well as the private sector. Only visible action in the form of federal leadership, coordination, and dissemination of technical knowledge can help achieve the needed changes more rapidly.

In 1975, the Secretary of Transportation issued a statement on national transportation policy (1). In this statement, he pointed out that, for the most part, the nation's required transportation infrastructure is in place. What is needed is modernization, repair, and more effective use of existing capacity.

Need for more effective maintenance has been expounded by Clary (2). Railroad track neglect, the need for upgrading resurfacing and bridge replacement of the highway system, needed dredging of waterways, and maintenance of air traffic control systems were described in terms of spiraling maintenance costs and the threat this presents to the country's transportation systems. The railroad's track problems are perhaps the most dramatic. Because of decreasing revenues and other reasons, needed upgrading and repairs were deferred year after year. The results of these actions (and, obviously, other factors) have led to the current situation of bankrupt corporations and an enormous capital investment requirement preceding any recovery efforts.

Highways, on the other hand, have enjoyed a period of stimulated growth with the building of the Interstate Highway System. Now that this effort is almost complete, the task of maintaining the massive network (in light of increased costs, inflation, and competition for available dollars) is most formidable. Fears of the aforementioned railroad dilemma are understandable.

While airlines are coping with maintenance burdens, the Federal Aviation Administration is contending with increased maintenance requirements resulting in the need for bigger budgets and more personnel—all this in a climate of fiscal restraint and competing national priorities. In light of this situation, it is natural for the U.S. Department of Transportation (DOT) to be concerned. The Office of the Assistant Secretary for Systems Development and Technology initiated a project in September 1975 to increase the overall perspective of maintenance problems and to determine what actions might be taken to minimize the probabilities of future crises.

OBJECTIVES

National transportation policy is established to serve the nation by helping to guide the development, financing, and maintenance of the transportation system (1). In addition, the policy sets precedents on the use of federal subsidies, regulation, maintaining diverse transportation modes, and ensuring fairness toward competition of the modes.

It was within this framework that a basic objective of this study—to determine the DOT's role in reducing the maintenance burden of transportation systems—was established. Two questions that underlie the basic objective were also addressed.

1. How significant are maintenance costs with respect to total system costs?
2. What research and development (R&D) opportunities exist to effectively reduce current and future maintenance costs?

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FINDINGS

Highways

The Federal Highway Administration (FHWA) estimates that the annual cost of highway maintenance is nearly $6 billion and is increasing at a rate of about $300 million/year. This represents about one-third of all current highway expenditures (3, pp. 1-30). If the trend shown in Figure 1 continues, maintenance could account for one-half of all highway expenditures by 1985 (4). Even with this expenditure, there are currently 1.127 million km (700 000 miles) of streets and highways in need of improvement of which 322 000 km (200 000 miles) are rated as critical. In addition, 90 000 bridges need replacement (5, 6). Road building, on the other hand, has decreased somewhat but obviously will not cease. Figure 2, which shows this, is based on FHWA data. Over the next 20 years, the amount of road building and upgrading that will be required is most formidable (7). There also exists a large backlog of secondary highway maintenance that has been estimated by FHWA to average about $2.25 billion/year over a 20-year period (1972 to 1992) (3). Figure 3 shows a more detailed examination of maintenance costs. Labor and traffic services each account for almost 50 percent of maintenance cost dollars. (Traffic maintenance includes snow and ice control and rest area and sign maintenance.) Physical maintenance, however, still consumes most of the volume of work performed (8).

Federal, state, and local highway departments are well aware of the challenge of maintaining roads and highways and have been for many years. The Transportation Research Board (TRB) and FHWA, together with the entire highway industry, are working to meet this challenge. Searches for better materials, equipment, and processes have been supported by both federal and state funds. More recently, causes for maintenance inefficiencies were identified, and improved maintenance management systems were encouraged. The following are the primary causes of highway maintenance inefficiencies:

1. Patronage system,
2. Lack of maintenance management research,
3. Inadequate maintenance data,
4. Lack of uniform standards,
5. Ineffective procedures for planning and scheduling, and
6. Overstaffing to meet emergencies.

Case studies have shown that maintenance planning, including performance standards and a good information system, can save significant amounts (11). For example, Crawford (10) found in 1971 that Louisiana maintenance costs were increasing by $4 million to $5 million/year. Through improved maintenance management practices, routine maintenance costs dropped $1.5 million. An attrition plan reduced the number of workers from 5150 to 3596 persons and increased total productivity. It has been estimated that 20 to 30 percent of current maintenance costs could be eliminated by efficiency improvements (5), especially those that reduce maintenance work force requirements. The 1974 National Transportation Report (20) stated that "there is no doubt that labor cost is the major cost factor, and the one that can have the greatest impact on total cost. This finding suggests that measures to control costs should focus primarily on labor costs."

Still the challenge is growing. As the Interstate Highway System was built, the primary and secondary roads were neglected (7). Demands on shipping produced heavier loads and greater traffic volumes, which tends to shorten the service life of pavements and structures (6). This suggests that the cost of maintaining roads could get worse, especially if antiquated maintenance practices continue (4). It is not surprising then that many states feel that increased labor efficiency is the first step in easing the budgetary and staffing strains (9;21, p. 53).

After effort for improved efficiency, a stepped-up program of research is considered the most promising avenue for reducing maintenance costs. FHWA, in conjunction with TRB, has identified highway maintenance research needs and has estimated that an investment of $9 million could return $750 million over a 5-year period (4). Some areas suggested for more research and development include

1. Diagnostic and condition measurement equipment;
2. Standardization of equipment;
3. Better composite materials;
4. Improved criteria for design and use of highways and bridges;
5. Better cleaning and litter pickup equipment;
6. Improved bridge design;
7. Better methods and equipment for pavement removal, replacement, and patching;
8. Methods for recycling materials;
9. Compact tools;
10. Equipment and maintenance data systems;
11. Equipment management system; and
12. Life-cycle costing.

The overall objective is to eliminate or at least minimize the need for maintenance. This can be accomplished through the integration of maintenance needs and requirements into design procedures and construction practices to obtain maximum benefit. A large portion of the maintenance cost burden can be avoided with better initial construction (12). (To this end, DOT has initiated a program to improve construction methods; improved tunneling methods are being demonstrated.)

A need exists for more standardization. The standardization of basic highway maintenance equipment would reduce acquisition costs, provide assets for more spare parts, and simplify the maintenance tasks. A recurring criticism is that there are no industry design standards or solid research experience for effective design (3). Adequate field data are also not readily available to provide the necessary basis for this type of endeavor.

Much of the maintenance costs can be reduced only by the mechanization of heretofore basically human labor processes and by great improvement of the efficiency of current mechanical processes. In addition, new equipments and processes must also be designed for operation and maintenance by semiskilled persons (13). There is some doubt about whether these things will be done. There is, however, a large program directed to improve highway maintenance capabilities and reduce costs (3). This year, FHWA has established a new R&D category, improved technology for highway maintenance, with the following research programs (22):

1. Use of waste material in road repair,
2. Improved inspection techniques for bridges and drainage structures,
3. Development of new test procedures and improved testing programs,
4. Improved technology for highway maintenance, and
5. Rehabilitation of existing pavements and development of premium pavements requiring no maintenance.

In addition, much complementary work is being conducted under the direction of state highway departments and the industry associations.

Railroads

The annual maintenance expenditure for railroads is currently more than $5 billion and is increasing at a rate of more than $280 million/year (Figure 4). This represents approximately 39 percent of all railroad expenditures (14, p. 16). These figures, however, do not take into account the amount of maintenance that is deferred as a matter of practice by the railroad industry. Deferred maintenance is more difficult to quantify. The most exhaustive study (16) estimated the total deferred maintenance of way and structure at approximately $6 billion for the nation's railroads. A special report on railroads concluded that the single largest deterrent to railroad profitability has been the result of a generation of deferred maintenance (17).

Harris (15) contends that "too much of the railroad's limited resources is required for maintenance." It can similarly be stated that, if the railroads are to increase their profitability, the cost of maintenance (39 percent) is a prime candidate for reduction. Examination of maintenance procedures indicates that there is considerable scope for reducing costs and increasing availability. [This has been confirmed also in foreign railroads (23).] Federal Railroad Administration personnel have related that, because of the downward trend of the railroad industry, maintenance policy, for the most part, is to repair as necessary. Preventive maintenance is practiced, however, on locomotives.

Improvements in maintenance management for some railroads could significantly reduce maintenance costs. It appears that systems management, operations research methods, and industrial management techniques are not being used to the extent that they should be in complex operations. Work standards, automated data systems, advanced operations, and optimization models are tools that are normally associated with modern management practices. In this regard, the American Association of Railroads and other associations are working on maintenance standards and minimum design standards. Lack of standardization in car components such as heating and cooling controls has led to many maintenance problems (25).

Government regulation may also be contributing to inefficient operations. Safety measures, for example, that result in maintenance actions such as inspection or immediate repair are based primarily on personal judgment or experience. Accurate data verifying these policies are believed rare. Better data are needed to correct these and other deficiencies.

In many operations, modern automated data systems have been installed because no maintenance policy can succeed without a continuous feedback of accurate field data. There are also emerging examples of expanded use of computer models that can assist in determining the best maintenance policies for various conditions (24, Appendix 4). Traditional ways of doing business and outmoded labor agreements are the biggest obstacles in introducing these changes.

Research and development in the railroad industry has been almost exclusively hardware oriented (15). Designs have been made to better accommodate specific services, such as trailers and containers as well as automobiles. Research and development is continuing on rails, trucks, controls, fire protection, and tunnel construction. Designs for track renewal machines and diagnostic and measurement equipment are also appearing for use. In addition, there is also some research, such as that on wheel-rail dynamics, that is evolving to improve the much needed base of technical knowledge. More of this type of research is believed necessary.

Urban Mass Transit

Data on buses, subways, and trolleys were found for the most part in Transit Operation Reports published by the American Public Transit Association (APTA). These data had to be extrapolated to estimate the national costs associated with maintenance. The annual maintenance expenditure for buses, subways, and trolleys is estimated to be $900 million/year at present. This is increasing primarily for trolley operations; bus and subways have remained somewhat constant. [Trolley maintenance costs are increasing primarily because of the aging fleet of vehicles. Only one trolley has been reported purchased since 1955 (26, p. 17, Table 13).] Maintenance expenditures account for 15 to 30 percent of operating expenses reported by the transit properties (Figure 5). These figures obviously do not include maintenance that is deferred for various reasons.

A closer look at available statistics shows that trolley and subway maintenance costs are higher than those for buses primarily because of the maintenance of way and structures expense. The following gives maintenance as a percentage of operating expense for trolleys, subways, and buses for 1973 and 1974:

Maintenance Expense Item	Trolleys	Subways	Buses
Way and structures	17.4	15.6	0
Equipment	9.9	13.4	15.7
Total	27.3	29.0	15.7

A large portion of the bus expense is the daily servicing required to keep equipment clean and operable. Figure 6 shows that servicing expenses approach those of actual repairs. Together, they account for 15 percent of the total operating expenses, which is second only to drivers' wages. The cost of maintenance is probably higher, for, although deferred maintenance cannot be quantified, it is known to exist. This policy is considered the lesser of many evils, which seems paradoxical in light of the fact that deferred maintenance is one of the suggested contributors to declining ridership.

The Urban Mass Transportation Administration (UMTA), together with industry groups such as APTA, is engaged in efforts to improve an industry that for the most part has been degrading since the late 1940s. Research and development expenditures for new buses, subways, and paratransit vehicles are increasing. Although bus engine performance has improved significantly since the 1930s, much needs to be done to improve components such as air conditioners, brakes, transmissions, and starters. Air conditioners for buses have caused the most significant maintenance problems. Regulations requiring procurement by the low bidder have caused an apparent setback in the improvement of these units as less reliable units have appeared in these buses.

Much effort is being directed toward improved maintenance management. Maintenance philosophies vary greatly from company to company. The key problems, however, appear to be the lack of good, readily available maintenance data; inadequate maintenance consideration during design; and the need to employ modern systems management methods, including ones for equipment and work standards, life-cycle costing, and modeling for systems optimization. The recognition of these needs is the basis for UMTA's current transit management program. In the research and development programs, TRANSBUS appears to contain many of the activities

Figure 1. Maintenance cost as a percentage of total highway expenditures.

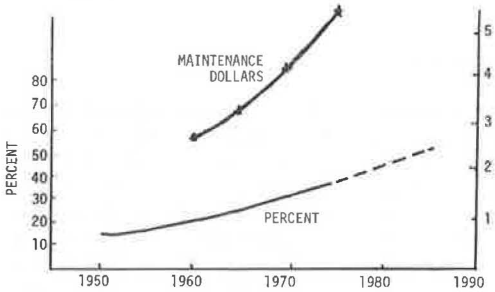


Figure 2. Highway building trend, 1964 to 1973.

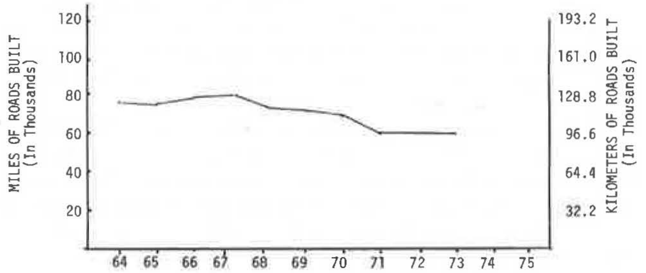
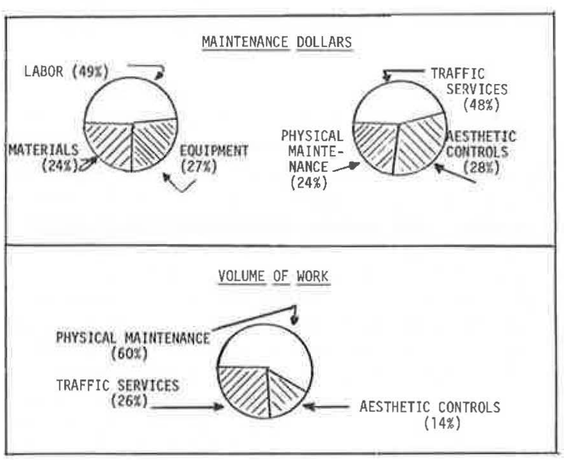
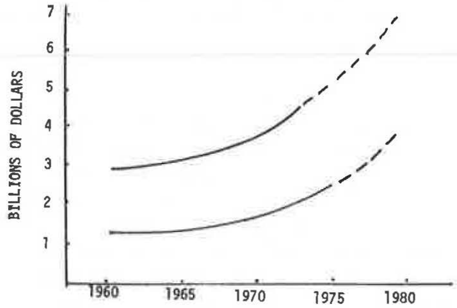


Figure 3. Distribution of maintenance dollars and volume.



NOTE: Traffic services include snow and ice control, rest areas and sign maintenance. Aesthetic controls include litter, mowing, vegetation, etc. Physical maintenance includes repairs to surface and base, structures, shoulders, etc.

Figure 4. Railroad maintenance costs from 1960 to 1974.



needed to overcome the aforementioned deficiencies. Test and evaluation programs for light rail transit continue this trend by including assessments of maintenance costs early in the life cycle. Some development work has also been done toward effective measurement and automatic diagnostic equipment.

Federal Aviation Administration

The cost of maintaining the nation's airway facilities is now approaching $350 million/year and is increasing at a significant rate as shown in Figure 7, which is based on data obtained from the Office of Budget of the Federal Aviation Administration (FAA). This is primarily due to recent inflation, expansion of the national airways system, and the obsolescence of a large portion of field equipment.

The FAA is aware of and has been attacking the problem of increasing maintenance costs. Existing equipment is being upgraded with the replacement of electronic tubes with solid state components. Current research and development is on automatic remote control and

Figure 5. Mass transit maintenance costs as a percentage of total operating expenses.

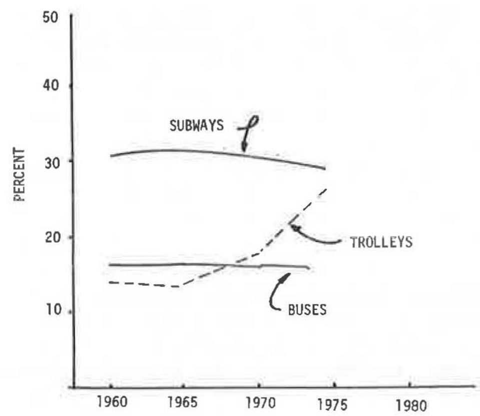


Figure 6. 1973 average bus operating expenses.

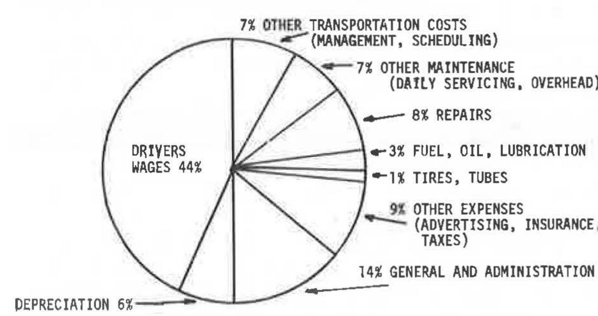
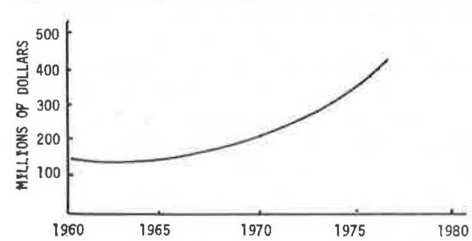


Figure 7. Airway facility systems maintenance costs.



monitoring systems (diagnostics) for newer equipment. In addition, a program called "zero maintenance" has been initiated to pull together the various activities engaged in reducing maintenance costs. This program includes the development of a new staffing (work) standard that will be used as a basis for updating the maintenance instruction manuals. A maintenance data system (MARS-1) is being implemented; information relating operation requirements to maintenance costs has been compiled; and systems modeling has been initiated to review maintenance concepts.

A significant deficiency appears to be in the current procedure to determine the maintenance concept after the procurement of equipment. This should be analyzed fully before purchasing decisions; that is, life-cycle costing should be done. Improvements in equipment specifications on maintainability, maintenance analysis, and test requirements should also be investigated more thoroughly.

U.S. Coast Guard

The maintenance expenses required to keep the U.S. Coast Guard (USCG) operational are as varied as the equipment used: buoys, rescue boats, larger ships, aircraft, traffic systems, bases, piers, communications and surveillance electronics, and the like. For this reason in particular, meaningful cost information could not be obtained without a level of effort inconsistent with the scope of the overall study.

The USCG is very much aware of the increasing costs of operations and maintenance and, although there is no overall program directed at reducing maintenance costs, specific efforts are always going on. In electronics, preventive maintenance is on the way out with the elimination of vacuum tube equipments. Computerized maintenance systems (diagnostics) are being developed to help optimize maintenance planning. New failure and maintenance data collection systems are being implemented and trend analyses are being performed.

New maintenance concepts are being tried. For example, experiments with downgraded equipment have been conducted in which a series of power sources are allowed to operate in a degraded mode with acknowledged failure modes. No preventive maintenance is performed and redundant equipment is used. Current R&D efforts include development of self-maintaining buoys.

DISCUSSION OF RESULTS

Significance of Maintenance Costs for the Various Transportation Modes

The cost of performing maintenance is of such magnitude that it should not be ignored. Highways, railroads, and mass transit are consuming almost $12 billion/year, and the burden is increasing at more than $600 million/year, which is somewhat consistent with the rate of inflation. More elusive is the cost of not performing maintenance. Estimates in both the highway and railroad industries indicate that deferred maintenance currently accumulated is greater than expenditure for maintenance of the current year. Although it cannot be proved conclusively, deferred maintenance is strongly believed to be a primary contributor to the degrading market share of both the railroads and mass transit. Some alarm was detected in the possibility that a similar degradation could occur with the national highway and road system.

The data compiled do not depict an existing crisis, nor is there any indication of an imminent catastrophe. Maintenance is still an operational option. However, in the current dollar competition situation, not to recognize the opportunity to reduce costs that will make available resources for more choices is unwise. The potential for reducing maintenance costs is considered very high and should be pursued more vigorously.

A Role for the Federal Government

It is the policy of DOT (through cooperative efforts with industry) to improve the efficiency and productivity of transportation systems (1). The information obtained in the course of this study shows that maintenance costs can be reduced both through improved maintenance management and through an expanded research and development program.

With respect to other opportunities, such as improved designs for new equipment or construction, an investment to reduce maintenance cost offers an attractive alternative characterized by a lower investment, a shorter implementation time, and a wider area of application. Where the unit rate of return might appear smaller when compared to the development of new equipment, the range of application of improved maintenance methods could more than offset the unit difference. In other words, improved maintenance techniques and methods would have more universal application in that they could be applied to old equipment as well as new equipment.

At an aggregate level, maintenance (regardless of application) has common principles, functions, needs, problems, and shortcomings. Some selected cases have shown a real savings of 20 to 30 percent to be possible. If a 10 percent savings were more likely, the savings on $13 billion/year is $1.3 billion/year (obviously an oversimplification but believed useful for sizing the opportunity).

The variation found between and even within the modes indicates that a need exists to accumulate the latest knowledge concerning maintenance, to organize it in the context of overall objectives, demonstrate selective aspects, disseminate the information, and assist in implementation. To date, research activities in maintenance-related areas have been somewhat fragmented and, perhaps with the exception of highways, lack a well-organized national program. It is also more than likely that these activities have been underfunded.

There evidently exists in some sectors a reluctance to consider maintenance as a bona fide candidate for R&D especially in light of "high risk" or "high technology" criteria. However, it must be seriously considered consistent with the stated DOT R&D goal to find less expensive ways to maintain the existing infrastructure and reduce capital costs and maintenance for new infrastructure. In addition, R&D by the federal government is warranted as a means to obtain factual data for making sound regulatory and other policies (19, pp. 4-8).

Not only the opportunity but also the role of the federal government should be made more clear. At the federal level, one can see into many areas of maintenance activity in the various departments and administrations, industry, and foreign nations. One also can see more easily the results of billions of dollars spent in research and development. The federal government has shown greater competence as a developer than perhaps in any other role as demonstrated by its participation in the highway program.

A more active role by the Department of Transportation, particularly in the area of R&D, is suggested by the foregoing. The extent of this activity will have to be determined after more in-depth studies, including cost-benefit analysis, are made and are compared with other R&D candidate projects.

Targets of Opportunity

This study revealed areas where common needs exist in efforts to reduce maintenance costs. These areas should be examined in more detail to determine the specific actions that should be taken (programs), the needed funds (investment), the time to implement, and the potential monetary payoff.

Maintenance Data Systems

The most important need in all areas is that of reliable and comparable data. The data needed falls into three major categories: (a) reliability (or failure) data, (b) availability (or time) data, and (c) cost data. There are numerous efforts going on in this general area. However, there is a need for better guidance (perhaps from more experienced sectors such as the airline and aerospace industries) on collection, storage, real-time access, usage, and standardization for higher level aggregation. (Because this is a costly area and requires 5 years or more to reach payoff proportions, government support will be needed.) Discussion of the value of good data is academic; the front end costs are the deterrent.

Related to the subject of data systems, one of the most important and often forgotten aspects is use. When one reviews the maintenance problems of the various transportation modes, one notices that not enough is known about the equipment and systems in operation. This is particularly critical with respect to technical characteristics, such as time to wear out, maximum stresses, aging, and the like. Centers of knowledge are not apparent. DOT should explore the potential of establishing this function in existing organizations such as the Transportation Systems Center of DOT. This activity could then be directly integrated into the ongoing efforts of technology transfer.

Standards and Standardization

Labor is by far the most expensive maintenance cost item (20). For maintenance management to be efficient, there must be some baseline from which to plan and determine efficiency. Case studies of state highway departments have shown 20 to 30 percent savings from employing these fundamentals. Development of work standards is one of the basic tools. All modes are in some stage of developing work standards independently. The potential of joint efforts should be explored. For example, taking advantage of what has been done in highways could speed the process of implementation in other areas.

The need for better design specifications and standards is almost universal. In some sectors, where performance specifications are considered as more desirable because of the flexibility allowed in response to the requirements, effective requirements for reliability, maintainability, and adequate testing have eroded because of their inherent cost. Lack of effective requirements and programs in these areas is the cause of many operating problems and maintenance costs being experienced today. Better methods for achieving higher reliability, better maintainability, and more effective testing at reasonable cost must be developed. The challenge is too large for any one sector. A joint effort under the leadership of the federal government should be evaluated.

In the private sector where no company commands the market place or where no strong common association has developed, the advent of nonstandard components adds significantly to maintenance and operating costs. In some modes of transportation (mass transit and railroads) this situation is openly acknowledged. A more in-depth study in the area should provide specific steps that the federal government might take in conjunction with the operators and trade associations to improve this aspect.

Verified Maintenance Requirements

There is a strong indication that many unnecessary maintenance policies and tasks are the result of regulations. [The maintenance task performed to satisfy the regulation and not the regulation itself is being challenged.] These maintenance requirements, in many cases, are formulated from individual judgment, perception of cause, or expected result. All too often, there is no verification of the effectiveness of the maintenance action. Causal data are also rare. It is believed that many of the maintenance requirements are overly costly in terms of person-hours, nonavailability of equipment, or noneffectiveness from a safety viewpoint.

Too often, the penalties associated with lowest cost procurement have hurt operations. Life-cycle costing is believed to be much better. Government regulations should be brought up to date on this issue. One major problem that is evident with life-cycle costing is that of good data, and better data would doubtlessly improve the situation.

Major questions on the effect of maintenance on transportation operations cannot be answered because of the difficulty in describing the operational environment that currently exists. Operations research, or more specifically simulation modeling, offers a tool to better cope with this problem. Where these models are limited in providing discrete answers, they have been proved most valuable in understanding "sensitivities" of changes. Changes in maintenance investment (up or down), efficiency (up or down), or policy could be better understood by using these tools. Operational policies would also be better served. A major problem here, once again, is the need for good data. The cost is also prohibitive for small operations. Federal leadership is needed to hasten widespread use of these capabilities.

Monitoring and Testing

The need to determine when maintenance must be performed, what level of performance is desired, or what the existing conditions were was found in all transportation modes. The basic problem appeared to be the inability to accurately describe conditions and what they mean in terms of imminent consequences.

Most inspections are made to identify hazardous conditions or impending failure. It is suspected that most inspection criteria are based on individual experience. A significant void is believed to exist in terms of data that accurately describe what conditions precede certain failures and by how long. If this is true, a large investment of maintenance dollars is wasted. (The airlines have recently imposed a design requirement that insists that these characteristics be known or that redundancy be built into the equipment.) A more detailed study into inspection costs, criteria, and effectiveness will provide insight into those areas (such as bridges, tunnels, and tires) where better inspection criteria might best be developed.

Much information is needed on the condition of equipment and structures. The decisions on when to repair or replace are becoming more critical because of the increasing dollar competition. The entire area of nondestructive testing promises enormous payoffs. An organized effort in this area will hasten the development of needed techniques and information. The front-end cost of this endeavor and the associated risks suggest that federal government support is needed.

A relatively new technology is making its way into the transportation field—automated diagnostics and monitoring. Electronics and computers and applications in the aerospace and aviation industries have provided the impetus. The cost and complexity of equipment and the increasing value of time (availability) are making this expensive alternative more attractive. Much has to be done in this area to make it more acceptable to the other transportation modes. Government support in this area has already demonstrated its effectiveness with track monitoring equipment and automated transit systems.

An in-depth evaluation of the potential cost and benefits will help determine its proper place in the R&D hierarchy.

ACKNOWLEDGMENT

I conducted this study at the request of R. L. Maxwell, Assistant Director for the Office of Systems Engineering, U.S. Department of Transportation. This study was my principal task during my assignment to the Office of Systems Engineering as part of the Federal Executive Development Program.

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Pros and Cons of Federal Aid for Highway Maintenance

Louis G. O'Brien, Pennsylvania Department of Transportation

The pros and cons of federal aid for highway maintenance are discussed. Data on Pennsylvania's maintenance effort are given. A proposal to return to the states a small portion of the federal tax for highway maintenance and safety is made.

In recent years there has been some interest in federal aid for maintenance among a few hard-pressed highway administrators. I would like to look at the definition of maintenance, past practice in the federal-aid highway program, and the reason why some departments of transportation are looking for federal aid for maintenance before looking at the pros and cons of the idea. The American Association of State Highway and Transportation Officials defines maintenance as

> the act of preserving and keeping of the right of way and each type of roadway, roadside, structure and facility as near as possible in its original condition as constructed or as subsequently improved and the operation of highway facilities and services to provide satisfactory and safe highway transportation.

Maintenance does not include construction or betterment. Construction is related to a new capital improvement; betterment is the improvement, adjustment, or addition to a highway that more than restores it to its former good condition and that results in better traffic serviceability without major changes in its original construction. Table 1 gives the type of work that comes under this definition of maintenance in Pennsylvania and represents its major maintenance effort.

This federal-aid highway program started in 1916 when World War I created the need for improved roads and was aimed at rural areas. The Federal Highway Act of 1921 placed the responsibility for maintenance of federally aided routes on individual states, and, in the event of unsatisfactory maintenance, the federal government might perform the required maintenance measures and charge the cost to the state concerned (1). Many state highway departments started with county aid for construction and reconstruction of county roads, and the counties were responsible for maintenance. In fact, in at least two states, some or all of the maintenance is performed by forces other than those of the state, and the state pays for the cost. From the very beginning, then, there has been a tradition of aid for construction with maintenance not included. Over the years, the need for new and improved highways has generally far outpaced the funds available for construction.

Through the years, the federal highway program has developed a large and diversified group of supporters who have direct and indirect interest in the federal-aid construction program and are usually referred to by journalists as the "highway lobby". It should be recognized that, before any federal aid for maintenance is authorized, 60 years of past experience will have to be reckoned with. Why are some states even looking for or talking about federal aid for maintenance when there is still a backlog in estimated highway needs? Some of the following could, by themselves or in combination, lead to a desire for federal aid.

1. The costs of providing motorist services such as rest areas, lighting, snow and ice control, and the like are generally much higher per kilometer on Interstate roads than on non-Interstate roads.
2. Inflation has raised the cost of maintenance.
3. Lack of reconstruction on the older primary and secondary roads has required a greater maintenance effort.
4. The energy crisis, with its accompanying slowdown in annual growth of fuel taxes from 5 to 2.5 percent, is causing lower than expected revenue.
5. Debt service on bonds to finance the 10 percent and 30 percent state share of the federal-aid program is great; in Pennsylvania, debt service has gone from \$37.4 million in 1969 to \$164 million in 1976.

With this in mind, let us look at some of the pros and cons of federal aid to maintenance.

Publication of this paper sponsored by Committee on Maintenance and Operations Systems.

Table 1. Maintenance in Pennsylvania.

Rank	Activity Description	Expenditures Dollars	Percent
1	Snow and ice control, antiskid chemical	29 938 718	18.5
2	Patching, manual	18 653 530	11.5
3	Surface treating, liquid bituminous material	13 112 205	8.1
4	Patching, mechanized	9 829 223	6.1
5	Line painting	6 723 526	4.2
6	Surface treatment, plant mix	6 100 228	3.8
7	Snow and ice control, plow	5 459 194	3.4
8	Grading, shoulders	4 580 229	2.8
9	Ditch and drain cleaning	4 454 849	2.7
10	Signs, repair and replacement	3 126 971	1.9
11	Shoulder restabilization	3 032 390	1.9
12	Shoulder cutting	2 924 997	1.8
13	Brush, selective trimming and thinning	2 851 555	1.8
14	Haul antiskid material to stockpile	2 462 818	1.5
15	Snow and ice control with plowing and spreading	2 039 948	1.3
16	Inlet and end wall cleaning	1 983 037	1.2
17	Guardrail repair	1 738 826	1.1
18	Pipe repair and replacement under 91.4 cm	1 645 948	1.0
19	Litter pickup	1 562 285	1.0
20	Shoulders, unpaved patch	1 449 085	0.9
21	Shoulders, surface treatment	1 445 625	0.9
22	Mowing	1 419 743	0.9
23	Snow fence erection	1 358 258	0.8
24	Bridge deck repair	1 333 579	0.8
			79.9

Note: 1 cm = 0.394 in.

PROS

Federal aid to maintenance would help meet increased costs assumed by states. For example, Pennsylvania spent $1 236 515 just to maintain and operate Interstate roadside rests in fiscal year 1974-75.

Federal aid to maintenance would help ease the revenue problem at the state level because maintenance is financed out of cash from current income.

Federal aid to maintenance could free cash revenue normally used for road maintenance for use in reconstruction or resurfacing of non-federal-aid systems.

Federal aid to maintenance could be a way to develop and enforce minimum levels of maintenance on all federal-aid routes.

CONS

If the normal federal-aid partnership type of program is used in maintenance, the red tape would be staggering. For example, the restoration of 8000 flood-damaged sites will require 120 five-drawer file cabinets in the Pennsylvania Department of Transportation district and central offices just for the project documentation.

Maintenance of highways and provision of satisfactory service are dependent on a large number of factors, which makes development of a national standard, and its objective imposition on the states, difficult.

When one thinks of the additional red tape and added bureaucracy at both federal and state levels, one concludes that it is most likely better for states to just raise taxes themselves and get all the money.

Program scope and priority are best developed at the level closest to execution. Therefore, there are bound to be serious conflicts between line managers and the federal program administrators with regard to what should be done and when.

In the political sphere the highway lobby has fought hard and long to ensure the integrity of the Highway Trust Fund. There are some who would no doubt look at federal aid to maintenance as a bust of the Highway Trust Fund, something similar to a mass transit "green grab".

CONCLUSION

I believe that any aid to the states for maintenance will have to be given without strings. Perhaps the best solution would be to give back to the states 0.264 cent/liter (1 cent/gal), now a part of the federal tax for highway maintenance and safety, without any programming and thus allow the states to use this in addition to their other highway revenues. A normal state accounting procedure could then serve to document that at least that value of maintenance work had been performed on the federal-aid highway system and, in fact, can be certified to each year.

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Public Transportation Employee Collective Bargaining

Committee on Maintenance and Operations Personnel

The rapid increase of collective bargaining strength of public transportation employees was explored at a conference session presided over by Charles T. Edson, New Jersey Department of Transportation. This paper contains summaries of presentations by Joseph Adler, American Federation of State, County and Municipal Employees, AFL-CIO; James F. Kelley, Massachusetts Department of Public Works; William P. Hobgood, Federal Mediation and Conciliation Service; and Alfred L. Miller, Federal Highway Administration. The paper also contains an edited transcription of the question and answer session.

The image of security and prestige associated with employment in the public sector before and for a decade and a half after World War II has faded. Public employees have, in large numbers, lost faith that they will be fairly treated by their employers. These changes have contributed to rapid growth in public employee union membership. John Hoerr stated in the August 30, 1976, issue of Business Week that more than half of the 14 million state and local government workers are now represented by unions. Such a rapid increase in collective bargaining strength has seemed menacing and has led to the widespread belief that management is helpless to resist raids on public treasuries by a strong voting bloc. There is evidence that this power does exist to some extent and that abuses have probably occurred. On the other hand, city and state public employee layoffs during the recent recession and the collapse of strikes in some cities demonstrate that management can mobilize public support and that the results of collective bargaining need not be one-sided.

The Committee on Maintenance and Operations Personnel believes that the presentations and discussions at this session support the view that unions have a more cohesive and well-directed program to achieve their aims than public bodies have and substantiate John Hoerr's statement, "The dilemma, briefly put, is that the nation has yet to develop a cohesive policy for dealing with the rapidly growing power of public unions, particularly at the municipal and state levels."

JOSEPH ADLER ON COLLECTIVE BARGAINING: A UNION POINT OF VIEW

Public employees have as much right to collective bargaining as private sector employees have. Public employees are being used as scapegoats by political executives in their effort to place the blame on someone for the problems of the economy. The economic situation must improve to bring about changes in the public sector including improving the financing of state and local government services, productivity of employees, impasse procedures and arbitration, and quality of public management. The ultimate reform, of course, is to institute collective bargaining as a federal law and thus guarantee public workers their right to bargain with unions and employers.

JAMES F. KELLEY ON UNIONISM IN THE MASSACHUSETTS STATE HIGHWAY DEPARTMENT

Unionism as it exists in Massachusetts is effective in resolving labor-management problems. Collective bargaining, described as a method of determining employment conditions through negotiations between employer and employees, was not easily acquired in Massachusetts. In 1969, legislation was finally enacted that strengthened collective bargaining. The pertinent sections of this legislation have been set forth in detail. Management must be fully versed in collective bargaining procedures to deal with the unions. Employees receive many benefits from membership in the union and have the collective voice needed to make their needs known. Employers benefit from unionism in that they can require standards of performance and can eliminate grievances at different management levels. Unionism has progressed slowly in the public sector, but it is now growing fast and is being accepted by both employers and employees.

Publication of this paper sponsored by Committee on Maintenance and Operations Personnel.

WILLIAM P. HOBGOOD ON MEDIATION IN COLLECTIVE BARGAINING

Mediation is described as a voluntary process in which the mediator operates through persuasion and suggestion to get both parties into a common area of agreement. In the private sector, this process has generally been successful. It is questionable whether, in the public sector, this service has jurisdiction under the Taft-Hartley Act. In conjunction with the increased growth of organization in the public sector, mediation has increased. Differences between private and public sector bargaining involve the question of recognition and the procedures used for dispute resolution; the rule that a mediator must deal strictly with those at the bargaining table; the fact that the mediator must play a more aggressive role with the media as a result of the high visibility of public disputes; and the stance that the mediator should take in representing the public interest in a dispute. In the private sector, the mediator's concern is not with the quality of the settlement, but simply with the settlement itself. One of the most difficult problems is dealing with the open bargaining laws of many states and their impact on the mediator's effectiveness. Questions such as these need much more discussion as the role of mediation services in the public sector expands.

ALFRED L. MILLER ON COLLECTIVE BARGAINING: A TRAINER'S POINT OF VIEW

Statistics show that unionism in the public sector is on the rise. From a trainer's point of view, management must be presented with a plan of action that will minimize problems in the transition into collective bargaining. For an ideal management-employee relationship to exist, courses must be designed to effect an understanding of the collective bargaining process. A training program in collective bargaining procedures must be established to ensure good labor-management relations.

QUESTION AND ANSWER SESSION

Question

How does one ensure legislative support for collective bargaining agreements?

Answer

This problem can occur through failure of the legislature to delegate enough authority to bargainers. It can be overcome by including legislators on the bargaining team, by including on the bargaining team representatives from the state treasurer's office who are aware of budget limitations, or by obtaining advice from leaders in the legislature on an acceptable area of settlement that they are willing to firmly support.

Question

Is arbitration a fairer approach than collective bargaining in the settlement of labor disputes?

Answer

Arbitration should not replace collective bargaining, but it may be used as a last resort. A need exists to develop a greater number of arbitrators with sufficient expertise to resolve public sector disputes. Such individuals must be sensitive to public interests and reaction. A private sector arbitrator often applies private sector concepts too glibly in public sector problems. They must be aware of and sensitive to the differences.

Arbitration should be an evolutionary process in any bargaining relationship. Until that relationship matures and the parties understand the process, an indoctrination period must take place. This period need not be so long for the public sector as for the private sector. Voluntary participation is the key to the success of the bargaining process. Australia went directly into arbitration. Most commentators there would probably say that they wish that they had followed the American pattern even though it is more chaotic and disruptive in certain situations because it is a more maturing process.

Question

What about the arbitration of disputes involving emergency personnel such as police and fire fighters?

Answer

Voluntary arbitration for police, fire fighters, and other emergency personnel should come only as a last resort. The collective bargaining process works, and arbitration should not replace that process.

Question

What are the conflicts between merit systems and collective bargaining in the public sector? How can they coexist?

Answer

All cities and states do not have equitable merit systems, and collective bargaining provides equity to the worker. Good merit systems should be preserved, not eliminated.

In Massachusetts, civil service is not subject to collective bargaining. Therefore, promotions, where civil service exists, will still be covered by civil service. In the labor service, however, promotion is based mostly on seniority, and that is the area where the local union gets involved in the development of a contract.

Question

Is collective bargaining one-sided? Does most of the strength reside on the union side?

Answer

The pressure is not as one-sided as you might think. Discussions, held in closed sessions, do not afford opportunities for elected officials to bargain away management prerogatives in the hope of gaining votes by public statements of support for the union position.

Legislatures and mayors have short memories, and union support does not mean much after the elections. Pressures from the bottom may work where union grievances are concerned, but the pressures from the top are just not there.

Question

Why have public agencies seemed to be so powerless to resist union pressures?

Answer

The most serious defect in the early stages of public sector bargaining was lack of expertise at the table. I think that as expertise increases management will be less vulnerable to that type of pressure. People have been found who would negotiate a political agreement that was not really in anyone's interests but that looked pretty good on paper. Many management prerogatives and work rules changes were hidden in agreements that are going to come back to haunt both parties in the future.

Question

What about "sunshine bargaining"?

Answer

After some serious analyses and talking to many mediators who have just experienced it, in Florida particularly, one finds that it does not help at all. There should be full disclosure, there should be no under-the-table deal, there should be no memoranda of agreement that are hidden, but the process itself is just too sensitive and too emotional in many cases, and too politically oriented, to respond constructively to sunshine law procedures.

Maintenance Management System for Asphalt Pavements

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A Markov decision model was developed to define optimal overlay maintenance strategies for in-service asphalt concrete pavements. This model was initially quantified by use of the subjective opinions of experienced highway engineers, and provisions were made to update these initial estimates with field data by means of Bayesian statistics. Optimal maintenance strategies were determined by minimizing the expected present value of total costs associated with a pavement and were considered to consist of highway department maintenance costs and excess user costs. The pavement maintenance management system developed provides a systematic and rational means for defining overlay maintenance policies and developing long-range plans for future maintenance together with a means for determining optimal use of available funds. Overlay maintenance alternatives considered were thin, medium, and thick overlays. Results indicated that, when overlay maintenance was required, medium or thick overlays representative of 10- and 20-year design periods respectively were optimal.

Improved methods for planning maintenance of pavements have assumed increased significance in recent years because of increasing age and distance covered by existing highways, increased levels of traffic, and the necessity for economical, comfortable, and safe roadways. To provide some assistance in the solution of this important problem, a pavement maintenance management system (PMMS) has been developed that defines and evaluates optimal overlay maintenance strategies for in-service asphalt concrete pavements.

Maintenance is used herein in the context of structural rehabilitation, and the purpose of the PMMS is to examine available maintenance alternatives in terms of when and what type of overlay maintenance should be applied so that optimal utilization of available funds can be achieved. In addition, the PMMS assists in the long-range planning for maintenance needs and budgets. A schematic diagram of the system is shown in Figure 1 and has been implemented in the University of California maintenance strategy computer program (CALMS 1).

This paper describes the essential features of the PMMS and is based on materials presented in detail elsewhere (1). In this study, a system of pavements was subdivided into 32 categories defined by the volume and composition of traffic, thickness of the asphalt-bound layer, subgrade quality, and degree of uniformity of pavement as measured by variation in surface deflection. To illustrate the methodology, some of the results for one pavement category are presented. (This model was designed for the use of lane-miles; therefore, the values are not given in kilometers in the figures and tables.) We hope that this will suffice to illustrate the potential of the approach.

SCOPE

The pavement maintenance management system has been developed by using a Markov decision model and consists of three basic components (Figure 1):

1. Maintenance alternatives,
2. Stochastic model of pavement behavior (performance), and
3. Cost model.

In its development, four overlay maintenance alternatives have been considered:

1. No overlay,
2. Thin overlay,
3. Medium overlay, and
4. Thick overlay.

For each of the four alternatives, the stochastic aspects of pavement behavior have been modeled by using a discrete, homogeneous Markov process, the states for which were defined in terms of pavement condition as measured by present serviceability index (PSI) and a cracking index (CI). The Markov model was initially quantified by using the subjective opinions of pavement engineers knowledgeable in the field performance of pavements. Provisions have been incorporated in the system to permit continual updating of this information by using Bayesian statistics as field data are obtained.

The cost model used considers the total costs associated with a pavement; these costs consist of both high-

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way department costs and excess user costs. Highway department costs consisted of routine and major maintenance costs. Excess user costs, measured in terms of vehicle operating costs and time value for trucks and automobiles, were assumed to result from the performance of routine and major maintenance and from deterioration in pavement condition.

The optimal maintenance strategy defined by the PMMS is the alternative that minimizes the expected present value of total costs for each pavement condition state. Parametric linear programming was used to determine the optimal strategy as a function of an excess user cost scale factor (EXUCSF), which is a measure of the proportion of the actual excess costs incorporated in the total costs experienced by highway users. As will be seen, EXUCSF permits the optimal maintenance strategy to be defined when a highway department budget constraint is imposed and provides an objective means to evaluate whether a maintenance budget should be increased or decreased. EXUCSF also permits an objective and consistent means of establishing uniform maintenance policies for a total system of pavements.

Although 32 pavement categories, defined by the data given in Tables 1 and 2, were examined in the original investigation, only the results for pavement category 2 are given in the subsequent sections of this paper. The pavement in category 2 is defined by the following average values of category parameters:

1. Traffic—design traffic number (DTN) = 450 [traffic index (TI) = 10.4, and average daily traffic (ADT) = 7000];
2. Asphalt concrete thickness (t_{AC}) = 100 mm;
3. Subgrade quality—resilient modulus (M_r) = 31 000 kN/m^2 (4500 lbf/in^2) [California bearing ratio (CBR) = 3, exudation value (R) = 16.5, and soil support value (S) = 3.0]; and
4. Degree of uniformity—coefficient of variation (v) in deflection measurements = 12.5 (v = standard deviation/mean).

The presentation that follows, which illustrates the methodology of the PMMS, has been developed according to the format of Figure 1.

MAINTENANCE ALTERNATIVES

The purpose of a pavement is considered to be the provision of safe, comfortable, convenient, and economic transportation. Because badly deteriorated pavements do not serve this purpose well, maintenance has to be performed to prevent progressive deterioration. During the gradual deterioration in pavement condition, applying some level of routine maintenance to all pavements is normal in an attempt to correct minor deficiencies. This routine maintenance, which consists of limited patching, filling or sealing cracks, and filling potholes, may slow the rate of deterioration or even temporarily upgrade the condition of a pavement. However, even if routine maintenance is applied, the condition of a pavement will continue to deteriorate. Therefore, at some point in the deterioration process, a level of maintenance greater than routine will become necessary.

This more extensive maintenance is termed major and is considered to be the placement of an overlay resulting in an upgrading of pavement condition. For convenience, the level of major maintenance has been defined by the thickness of overlay, and the four alternatives previously defined.

Alternative 1 consisted of performing only routine maintenance. A 25-mm (1-in) thickness was assumed for alternative 2. Overlay thicknesses for alternatives 3 and 4 were determined by using the overlay design procedure of the Asphalt Institute (2). Alternative 3 was considered to be an overlay typical of staged construction and adequate for a 10-year design period. Alternative 4 was designed for a 20-year design period. For pavement category 2, the resulting overlay thicknesses were 50 mm (2.0 in) and 75 mm (3.0 in) for alternatives 3 and 4 respectively.

STOCHASTIC MODEL OF PAVEMENT BEHAVIOR

When a pavement is first constructed, its condition normally will be excellent. After it is opened to traffic, the condition will gradually deteriorate. This process of gradual deterioration can be thought of as a series of transitions of pavement condition states from the initially excellent condition to less desirable conditions. Such a concept of a transition of pavement condition states can be used as a basis to model the gradual deterioration of pavement condition. To accurately define the condition state of a pavement, parameters that measure pavement performance (condition) must be defined. In this study, pavement condition is defined by a pavement maintenance index, PMI, a two-component vector consisting of ride quality as measured by PSI, and the amount of load-associated cracking as measured by CI. The CI was defined as a percentage of a lane-mile that exhibited visible load-associated cracks. For each pavement category (Table 2), 21 pavement condition states were defined and are given in Table 3.

The stochastic or probabilistic nature of pavement performance was considered by use of a discrete Markov process representation of pavement behavior. The theory of Markov processes can be found elsewhere (13, 14). The states in the Markov process were defined by the value of the PMI. Thus changes in pavement condition were measured in terms of transitions between levels of the PMI that were made discrete. The Markov process was used to predict the behavior of a lane-mile of pavement in each possible pavement condition state as a function of time. Time was made discrete in a series of 1-year steps, and the process was assumed to be homogeneous in time.

The key assumption when a Markov process is used to predict pavement behavior is that the probability of transition to a future condition state is only dependent on the current condition state. Thus the current condition state of the pavement, not how the pavement arrived at its current condition state, is the only relevant information with regard to future behavior. This assumption is believed to be reasonable and justifiable for a given category of pavements.

A homogeneous Markov model can be completely defined by a transition probability matrix and an initial state vector. The transition probability matrix describes the likelihood of transition between every pair of pavement condition states over a 1-year time period. Thus, if a pavement begins the year in a given state, the chances that the pavement will end the year in other states are quantified in the transition probability matrix. The initial state vector is the means to describe the starting condition of the process and simply assigns probabilities to each starting state. For a system of pavements, the initial state vector would simply represent the percentage of the total lane-miles of pavement currently in each pavement condition state.

To use the Markov model to predict pavement behavior, we had to quantify each element of the transition probability matrix. Three methods of obtaining this information were considered:

1. An analytical model of the behavior of pavement to define the transition probability elements could be used.

Figure 1. Pavement maintenance management system for a category of pavements.

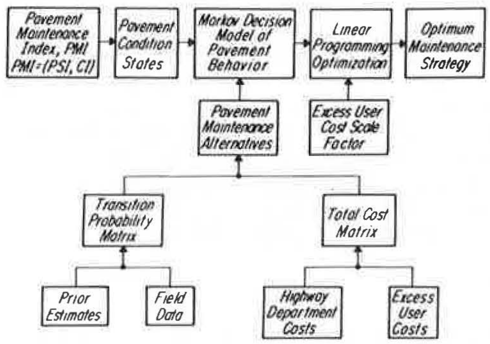
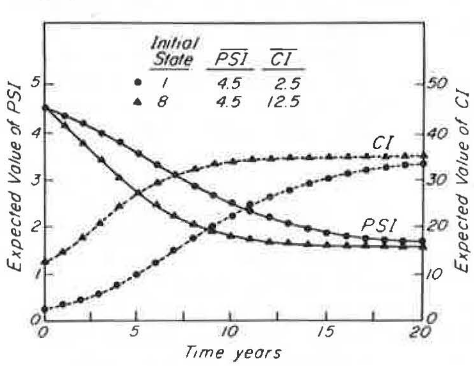


Table 1. Flexible pavement category parameters.

Item	Range	Mean
Traffic		
DTN	15 to 150	82.5
	150 to 750	450
	750 to 3000	1875
	3000 to 7000	5000
TI	6.9 to 9.2	8.5
	9.2 to 11.1	10.4
	11.1 to 13.0	12.3
	13.0 to 14.2	13.8
ADT[a]	600 to 4500	3000
	4500 to 11 000	7000
	11 000 to 24 000	18 000
	24 000 to 42 000	30 000
t_{AC}, mm	50 to 152	100
	152 to 305	229
Subgrade quality		
M_r, kN/m^2	10.4 to 51.8	31.0
	51.8 to 103.5	77.6
CBR[b]	1 to 5	3
	5 to 10	7.5
R	4 to 29	16.5
	29 to 43	36
S	1.4 to 4.6	3.0
	4.6 to 5.4	5.0
v, %	5 to 20	12.5
	20 to 40	30

Notes: 1 kN/m^2 = 0.145 lbf/in^2.
Coefficient of variation of series of surface deflection measurements is for 1 lane-mile of roadway. A high degree of uniformity corresponds to a low coefficient of variation.
[a]In one direction.
[b]Estimated from modulus values by relationship E in kilonewtons per meter^2 = 1000(CBR).

Figure 2. Expected values of PSI and CI as a function of time for maintenance alternate 1, pavement category 2.



The model must be capable of accurately predicting pavement behavior in terms of PSI and CI for specific values of traffic volume, asphalt concrete thickness, subgrade quality, and degree of uniformity. Furthermore, the model must predict future behavior in a probabilistic manner and be applicable to both new and deteriorated pavements. Although much progress has been made in recent years toward developing such an analytical model of pavement behavior, no pavement model with all these capabilities is yet available. Therefore, although an analytical model with the capabilities mentioned may be developed in the near future and could be used to obtain the necessary transition probability matrices, this method currently is beyond the technological development of pavement models.

Table 2. Flexible pavement categories.

Category	Traffic DTN	TI	ADT[a]	t_{AC} (mm)	Subgrade Quality M_r (kN/m^2)	CBR	R	S	v (%)
1	82.5	8.5	3 000	100	31 000	3.0	16.5	3.0	12.5
2	450	10.4	7 000	100	31 000	3.0	16.5	3.0	12.5
3	1875	12.3	18 000	100	31 000	3.0	16.5	3.0	12.5
4	5000	13.8	30 000	100	31 000	3.0	16.5	3.0	12.5
5	82.5	8.5	3 000	229	31 000	3.0	16.5	3.0	12.5
6	450	10.4	7 000	229	31 000	3.0	16.5	3.0	12.5
7	1875	12.3	18 000	229	31 000	3.0	16.5	3.0	12.5
8	5000	13.8	30 000	229	31 000	3.0	16.5	3.0	12.5
9	82.5	8.5	3 000	100	77 600	7.5	36	5.0	12.5
10	450	10.4	7 000	100	77 600	7.5	36	5.0	12.5
11	1875	12.3	18 000	100	77 600	7.5	36	5.0	12.5
12	5000	13.8	30 000	100	77 600	7.5	36	5.0	12.5
13	82.5	8.5	3 000	229	77 600	7.5	36	5.0	12.5
14	450	10.4	7 000	229	77 600	7.5	36	5.0	12.5
15	1875	12.3	18 000	229	77 600	7.5	36	5.0	12.5
16	5000	13.8	30 000	229	77 600	7.5	36	5.0	12.5
17	82.5	8.5	3 000	100	31 000	3.0	16.5	3.0	30.0
18	450	10.4	7 000	100	31 000	3.0	16.5	3.0	30.0
19	1875	12.3	18 000	100	31 000	3.0	16.5	3.0	30.0
20	5000	13.8	30 000	100	31 000	3.0	16.5	3.0	30.0
21	82.5	8.5	3 000	229	31 000	3.0	16.5	3.0	30.0
22	450	10.4	7 000	229	31 000	3.0	16.5	3.0	30.0
23	1875	12.3	18 000	229	31 000	3.0	16.5	3.0	30.0
24	5000	13.8	30 000	229	31 000	3.0	16.5	3.0	30.0
25	82.5	8.5	3 000	100	77 600	7.5	36	5.0	30.0
26	450	10.4	7 000	100	77 600	7.5	36	5.0	30.0
27	1875	12.3	18 000	100	77 600	7.5	36	5.0	30.0
28	5000	13.8	30 000	100	77 600	7.5	36	5.0	30.0
29	82.5	8.5	3 000	229	77 600	7.5	36	5.0	30.0
30	450	10.4	7 000	229	77 600	7.5	36	5.0	30.0
31	1875	12.3	18 000	229	77 600	7.5	36	5.0	30.0
32	5000	13.8	30 000	229	77 600	7.5	36	5.0	30.0

Notes: 1 kN/m^2 = 0.145 lbf/in^2.
Values given are mean values for each flexible pavement category parameter.
[a]In one direction.

Table 3. Definition of pavement states.

State Number	PSI Range	PSI Mean	CI (%) Range	CI (%) Mean	PMI
1	4.75 to 4.26	4.5	0 to 4.9	2.5	(4.5, 2.5)
2	4.25 to 3.76	4.0	0 to 4.9	2.5	(4.0, 2.5)
3	3.75 to 3.26	3.5	0 to 4.9	2.5	(3.5, 2.5)
4	3.25 to 2.76	3.0	0 to 4.9	2.5	(3.0, 2.5)
5	2.75 to 2.26	2.5	0 to 4.9	2.5	(2.5, 2.5)
6	2.25 to 1.76	2.0	0 to 4.9	2.5	(2.0, 2.5)
7	1.75 to 1.26	1.5	0 to 4.9	2.5	(1.5, 2.5)
8	4.75 to 4.26	4.5	5.0 to 19.9	12.5	(4.5, 12.5)
9	4.25 to 3.76	4.0	5.0 to 19.9	12.5	(4.5, 12.5)
10	3.75 to 3.26	3.5	5.0 to 19.9	12.5	(3.5, 12.5)
11	3.25 to 2.76	3.0	5.0 to 19.9	12.5	(3.0, 12.5)
12	2.75 to 2.26	2.5	5.0 to 19.9	12.5	(2.5, 12.5)
13	2.25 to 1.76	2.0	5.0 to 19.9	12.5	(2.0, 12.5)
14	1.75 to 1.26	1.5	5.0 to 19.9	12.5	(1.5, 12.5)
15	4.75 to 4.26	4.5	20.0 to 50.0	35.0	(4.5, 35.0)
16	4.25 to 3.76	4.0	20.0 to 50.0	35.0	(4.0, 35.0)
17	3.75 to 3.26	3.5	20.0 to 50.0	35.0	(3.5, 35.0)
18	3.25 to 2.76	3.0	20.0 to 50.0	35.0	(3.0, 35.0)
19	2.75 to 2.26	2.5	20.0 to 50.0	35.0	(2.5, 35.0)
20	2.25 to 1.76	2.0	20.0 to 50.0	35.0	(2.0, 35.0)
21	1.75 to 1.26	1.5	20.0 to 50.0	35.0	(1.5, 35.0)

2. Actual field data collected to monitor the field performance of pavements could be used. In recent years, continuous and systematic field performance measurement programs have been instituted by many state highway departments. Therefore, data collected in the format used in this study would make it possible to define the transition matrix. Unfortunately, the necessary field data were not available for use in this study.

3. Bayesian statistics, which permit the consideration of both subjective and objective data, could be used. This approach permits subjective or personal opinions of experienced personnel to be incorporated with actual objective data obtained from field measurement programs. This was the method adopted by us.

In a Bayesian sense, probability is interpreted as the degree of belief that can be assigned to the occurrence of an event. Thus, a Bayesian interpretation of probability allows the subjective opinions of pavement engineers who have observed the performance of field movements for many years to be considered and directly incorporated in the definition of the transition matrices. This provides a means, in the quantification of the Markov model of pavement behavior, of directly and advantageously using the many years of experience accumulated by pavement engineers.

For this investigation, the Bayesian or subjective estimates of transition probability elements were obtained from a group of experienced pavement engineers through the use of a questionnaire that was implemented by conducting personal interviews. The engineers who completed the questionnaire had a total of more than 100 years of experience in observing the field performance of pavements. The questionnaire allowed this background of experience to be incorporated in the initial quantification of the Markov model. Estimates of transition probability elements so obtained were used in this study, which permitted an initial and rapid quantification of the Markov process model of pavement behavior. As stated previously, Bayesian statistics allow both subjective and objective data to be incorporated in the estimates of probability. A procedure for incorporating objective estimates of transition probability elements obtained from field measurements was developed so that the subjective estimates could be continually updated as field data are collected. This capability was incorporated as a part of CALMS 1.

An example of the pavement performance predicted by means of the Markov model quantified by using only subjective information is shown in Figure 2. These results show the expected pavement performance if alternative 1 was continually followed over a 20-year time period for pavement category 2. Only 2 of the 21 possible pavement condition states are shown in the figure. The expected values of PSI and CI are illustrated for pavements with initial condition states of 1 and 8.

Pavements with these initial states have an average PSI value of 4.5 when time equals zero. However, the initial state CI values are 2.5 and 12.5, respectively. For both pavements, the expected value of PSI decreases with time. However, a pavement that initially exhibits more cracking (initial state 8) will show a much more rapid drop in the expected value of PSI. For long periods of time, both pavements tended toward an expected PSI value of 1.5, and this was the lowest average value for PSI used in this study.

The expected value of CI increases with time for both pavements. Pavements that begin with more cracking (initial state 8) show a more rapid rate of increase in cracking initially. For long periods of time, both pavements tended toward an expected CI value of 35.0, and this was the highest average value for CI used in this study.

The time histories of expected PSI and CI shown for two initial states in Figure 2 can be used to assess the applicability of the homogeneous Markov process model of pavement behavior used in this study. Although making a definitive assessment of how well the time histories shown in Figure 2 model actual field behavior of pavements is difficult until similar plots determined by large amounts of field data can be obtained for the conditions used in this study, the time histories shown appear to be in general agreement with observed pavement behavior and therefore provide a reasonable initial model.

COST MODEL

The total cost associated with a pavement was considered to consist of two components:

1. Highway department costs, which are funds spent to actually maintain a pavement, and
2. Highway user costs, which are costs incurred by the users of the pavement and are dependent on the condition state of the pavement and maintenance activities conducted by the highway department.

Highway department costs for the maintenance of trafficked pavement surfaces were divided into two types: (a) major maintenance costs, which are those costs associated with the placement of a thin, medium, or thick overlay, and (b) routine maintenance costs, which are those costs associated with such maintenance activities as filling and sealing cracks, limited full-depth repair or replacement, filling potholes, and resurfacing less than 19 mm (¾ in) in thickness for less than 153 m (500 ft). Material, equipment, and personnel costs necessary to actually perform the maintenance activity make up the highway department costs.

Highway user costs consist of vehicle operating costs and costs resulting from a value of time assigned to highway users. Users will always incur these costs regardless of the pavement condition or maintenance activities performed by the highway department. Conditions that occur that induce greater highway user costs than those associated with a pavement in excellent condition have been termed excess user costs and are the user costs of primary interest in this study. These excess user costs were considered to arise from: major maintenance operations, routine maintenance operations, and deterioration in pavement conditions. Although highway engineers generally agree that user costs should be considered as part of the total cost of a pavement, these costs are not now as well-defined as highway department costs.

In the past, user costs have been indirectly considered when determining maintenance policies in that the motivation for maintenance was based on providing the user with safe, convenient, and economical highway transportation. However, only recently has attention been directed to quantifying user costs so that they can be formally treated in the analysis of the total cost of a highway. At this time, sufficient quantitative information concerning user costs appears to be available to provide reasonable estimates for them. Furthermore, use of these estimates of user costs indicates that such costs are not small, and often exceed the actual maintenance costs for the pavement. Therefore, it is appropriate to make reasonable estimates of user costs by using available information rather than to neglect them because they are difficult to accurately quantify.

User costs considered were (a) vehicle operating costs, which are those costs associated with fuel, tires, oil, maintenance, and depreciation required to operate a vehicle, and (b) time costs, which are those costs resulting from an economic value of time assigned to the highway users. Vehicle operating costs and time value for trucks and automobiles were considered separately.

The concepts for a user cost model used in this study are those developed elsewhere (3, 4, 5, 6). Data to quantify the model also were obtained elsewhere (7, 8, 9, 10, 11, 12). All aspects of the cost model were incorporated in CALMS 1, and computations were performed from basic input values defined by the user.

MARKOV DECISION PROCESS

The theoretical basis of the PMMS is that of a Markov decision process. Introduction of the four maintenance alternatives to a Markov process defines a Markov decision process, the objective of which in this case is to select the optimum maintenance strategy for a pavement.

The four maintenance alternatives are represented in the Markov decision process by a transition probability matrix and a cost matrix for each maintenance alternative. The transition probability matrix representing each maintenance alternative was quantified by using the subjective opinions of experienced highway engineers. The elements of the corresponding cost matrix are calculated by CALMS 1 by using the cost model. The Markov decision process combines the transition probability matrix and associated cost matrix for each maintenance alternative and defines the economic trade-offs that exist between highway and department costs and excess user costs. In other words, is the money required to perform pavement maintenance justified by the expected savings to highway users?

The criteria used to define an optimal maintenance strategy for a pavement were based on minimizing the expected present value of total costs over an infinite time horizon for each initial pavement condition state. Use of an infinite time horizon implies that pavements in existence today will remain in existence forever. However, the expected future costs were discounted to their present value through the use of a discount (interest) rate that measures the time value of money. Therefore, the present value of a given cost decreases as the time from the present increases.

LINEAR PROGRAMMING OPTIMIZATION

Linear programming was used to define the optimal maintenance strategy for the discounted infinite-horizon Markov decision process. The optimal maintenance strategy is defined by the optimal maintenance alternative for each of the 21 initial pavement condition states. Therefore, after the current state of the pavement is known, the optimal maintenance alternative to apply is defined.

Each element of the total cost matrix for each maintenance alternative was expressed as the sum of highway department costs plus a prescribed percentage of the excess user costs. This can be represented as

$$TC = THYDC + \theta(TUC) \qquad (1)$$

where

TC = total costs,
THYDC = total highway department costs,
TUC = total excess user costs, and
θ = constant (percentage) and is termed EXUCSF.

EXUCSF allows the effect of scaling excess user costs up or down from their best estimate values (which are input to CALMS 1) to be examined. Use of parametric linear programming allowed the optimal maintenance strategies to be defined for all values of EXUCSF ranging from zero to a specified maximum value. Thus the optimum maintenance strategies were defined as a function of EXUCSF. EXUCSF will be shown to be a very useful tool when interpreting optimal maintenance strategies from the PMMS.

PRESENTATION AND INTERPRETATION OF TYPICAL RESULTS

The results of the PMMS define the optimal maintenance strategy as a function of EXUCSF. In this section, the significance of these results will be discussed and interpreted. In addition, the expected yearly cost to the highway department to allow an optimal maintenance strategy can be found from the results of the PMMS. These results should prove extremely useful in planning and budgeting highway department maintenance expenditures. Furthermore, the results of the PMMS can be used to determine the optimal maintenance strategy to follow when faced with a budget constraint. After the optimal maintenance strategy is defined, the consequences and implications of following an optimal maintenance strategy can be examined by using the Markov processes. For example, the long-term or limiting behavior of the pavement system if an optimal maintenance strategy is continually followed can be found. Determination of these results with the computer program CALMS 1 will be described in this section.

Optimal Maintenance Strategies

The definition of optimal maintenance strategies is the primary result obtained from the PMMS. An optimal maintenance strategy defines the optimal maintenance alternative to apply to a pavement as a function of its current condition state. Optimal maintenance strategies for pavement category 2 are summarized in Table 4. This table shows the optimal strategy as a function of the value placed on excess user costs, as measured by EXUCSF. It can be seen that, as the value placed on excess user costs increases (EXUCSF increases), the amount of major maintenance (maintenance alternatives 2, 3, and 4) in an optimal maintenance strategy increases.

The values used to determine excess user costs were input to CALMS 1 by using the best available estimates for user costs. We hope that the excess user costs determined by CALMS 1 represent realistic and reasonable values. If so, then the optimal maintenance strategy that should be followed corresponds to the optimal maintenance strategy for an EXUCSF of 1.000. This optimal maintenance strategy is the one that results when the best estimates of highway department and excess user costs are used to define total costs. For pavement category 2, this maintenance strategy is optimal for an EXUCSF anywhere from 0.885 to 1.131. Therefore, this maintenance strategy would remain optimal if the value placed on excess user costs was anywhere between 0.885 to 1.131 times the best estimate of actual excess user costs that were input to CALMS 1.

The optimal maintenance strategy for pavement category 2 was (a) do nothing for pavement states 1, 2, 3, 8, 9, and 15; (b) use medium overlay for pavement states 4, 5, 10, 11, 16, and 17; and (c) use thick overlay for pavement states 6, 7, 12, 13, 14, 18, 19, 20, and 21. The average values of the PMI, in terms of average PSI and CI, corresponding to each pavement state were defined in Table 3. It can be seen that the optimal maintenance strategy for pavement category 2 indicates that major maintenance should be performed when the average PSI value drops below 3.0 to 3.5 depending on the average CI value. Medium overlays, representative of staged construction, were optimal if the pavement was moderately deteriorated while thick overlays were required if the pavements were badly deteriorated. Note that thin overlays were not optimal for any of the pavement states. In other words, if the pavement required major maintenance, a medium or thick overlay was optimal.

Consider now the optimal maintenance strategies as the value of EXUCSF increases. If no value is placed on excess user costs, maintenance alternative 1 is optimal for all pavement states, and the maintenance cost only to the highway department is minimized. This maintenance strategy remains optimal until the value placed on excess user costs reaches 0.074 times the best estimate of excess user costs. At this value, the optimal maintenance strategy changes, and thin overlays become optimal for pavements in state 6. Note that this maintenance strategy requires more highway department expenditures. The optimal maintenance strategy remains unchanged until EXUCSF reaches a value of 0.075, where thin overlays for pavements in state 7 become optimal. The optimal maintenance strategy now indicates that thin overlays should be placed on pavements in both states 6 and 7, and the resulting highway department costs will therefore be greater than for the previous optimal maintenance strategy. This process continues until EXUCSF exceeds 1.0, which was specified as part of the input to CALMS 1. The order in which major maintenance enters the optimal policy defines the order of priority for performing maintenance. As the value placed on excess user costs increases, the amount of major maintenance in an optimal maintenance strategy increases and the resulting costs to the highway department increase.

Note that the optimal maintenance strategy changes rapidly at first as the value of excess user costs increases above zero. However, with further increase in the value of excess user costs, the optimal maintenance strategy changes more slowly for proportionate changes in EXUCSF. For example, the optimal maintenance strategy changes for thirteen pavement states when EXUCSF increases from 0 to 0.2. However, optimal maintenance changes for only four pavement states when EXUCSF increases from 0.8 to 1.0. This behavior is a direct result of the fact that maintenance of pavements in poor condition can reduce excess user costs more significantly than can maintenance of pavements in better condition. After the level of maintenance has reached the state where increased highway department costs to perform maintenance are only justified by large increases in excess user costs, the optimal maintenance strategies change more slowly for given changes in EXUCSF.

Expected Highway Department Costs

On each run of CALMS 1, the expected 1-year highway department costs for each pavement state are defined. A particular maintenance strategy defines the appropriate highway department costs for each pavement state, and the initial probability distribution (state vector) defines the proportion of pavements initially in each pavement state. The expected 1-year highway department costs for each lane-mile of pavement in pavement category 2 can be found from this information by simply taking the expectation of highway department costs.

Table 5 gives the expected first-year highway department costs per lane-mile per year for four different maintenance strategies. The four maintenance strategies given were optimal for EXUCSFs of 0.25, 0.50, 0.75, and 1.00 as noted in Table 4. The expected values of first-year highway department costs for each lane-mile of pavement in pavement category 2, given at the bottom of Table 5, were: \$3999, \$6405, \$7579, and \$7992 for the optimal maintenance strategies corresponding to EXUCSFs of 0.25, 0.50, 0.75, and 1.00 respectively.

The expected first-year highway department costs increase as the value placed on excess user costs increases. As previously discussed, the optimum maintenance strategy for an EXUCSF of 1.00 is the maintenance policy that should be followed. If this maintenance policy is followed, the expected first-year highway department cost for all category 2 pavements can be found by multiplying the cost per lane-mile by the number of lane-miles of pavement in category 2. For example, if 100 lane-miles of pavement are in category 2, then the total expected first-year highway department maintenance costs would be \$799 200. Although this is the optimal maintenance strategy that should be followed, the budget of the highway department for maintenance of pavements in category 2 may not be adequate to accomplish this high level of maintenance.

Optimal Maintenance Strategies for a Constrained Budget

Consider the situation in which the budget of the highway department for maintenance of all pavements in category 2 is not large enough to allow an optimal maintenance strategy for an EXUCSF of 1.00 to be followed. For this situation, the maintenance strategy that would be followed is the one for which the expected highway department costs are within the budget and the value of EXUCSF is as close to 1.00 as possible. For EXUCSFs lower than 1.00, the expected 1-year highway department costs to follow an optimal maintenance strategy are always equal to or less than that for an EXUCSF of 1.00. Therefore, the expected highway department costs can be reduced by examining optimal maintenance strategies for EXUCSFs lower than 1.00. For example, assume that the budget for maintenance of 100 lane-miles of pavement in category 2 is \$640 500. For this situation, the maintenance strategy that should be followed is the optimal maintenance strategy for an EXUCSF of 0.50. This is as close to the true optimal maintenance strategy that the budget will allow. It must be emphasized, however, that all maintenance strategies followed for which EXUCSF is not 1.00 are not optimal when all costs are considered. Therefore, if the budget is not sufficient to perform the maintenance indicated at an EXUCSF of 1.00, additional funds should be requested. On the other hand, if the budget is larger than the costs necessary to follow optimal maintenance strategy when EXUCSF equals 1.00, these funds should be diverted to other activities. Thus the value of the EXUCSF for which the performance of an optimal maintenance strategy is possible defines whether the budget is too large or too small and provides a firm basis on which this judgment can be made.

In this paper, only the results for pavement category 2 have been presented. However, definition of optimal maintenance strategies within a constrained maintenance budget considering all pavement categories can be defined by using the same techniques. The value of EXUCSF, which is the same for all pavement categories and allows an optimal maintenance strategy to be followed within the total budget, defines the best maintenance strategy to follow for each category of pavements. Furthermore, this guarantees that the same value will be placed on excess user costs for all pavement categories. Thus an objective procedure for determining the maintenance policy for all pavements without the bias of

Table 4. Optimal maintenance strategies for pavement category 2.

	Initial Pavement Condition State																				
EXUCSF	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
0 to 0.074	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
0.074 to 0.075	.	.	.	.	.	2	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.075 to 0.093	.	.	.	.	.	.	2	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.093 to 0.098	.	.	.	.	.	.	.	.	.	.	.	.	.	3	.	.	.	.	.	.	.
0.098 to 0.099	.	.	.	.	.	.	.	.	.	.	.	.	3	.	.	.	.	.	.	.	.
0.099 to 0.105	.	.	.	.	2	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.105 to 0.113	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	3
0.113 to 0.115	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	3	.
0.115 to 0.146	.	.	.	.	.	.	.	.	.	.	.	3	.	.	.	.	.	.	.	.	.
0.146 to 0.163	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	3	.	.
0.163 to 0.187	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	4
0.187 to 0.193	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	3	.	.	.
0.193 to 0.222	.	.	.	.	.	.	3	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.222 to 0.244	.	.	.	.	.	.	.	.	.	.	3	.	.	.	.	.	.	.	.	.	.
0.244 to 0.286	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	4	.
0.286 to 0.339	.	.	.	.	.	3	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.339 to 0.340	.	.	.	.	3	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.340 to 0.342	.	.	.	.	.	.	.	.	.	.	.	.	.	4	.	.	.	.	.	.	.
0.342 to 0.351	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	3	.	.	.	.
0.351 to 0.364	.	.	.	2	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.364 to 0.370	.	.	.	3	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.370 to 0.447	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	4	.	.
0.447 to 0.512	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	3	.	.	.	.	.
0.512 to 0.514	.	.	.	.	.	.	.	.	.	3	.	.	.	.	.	.	.	.	.	.	.
0.514 to 0.604	.	.	.	.	.	.	.	.	.	.	.	.	4	.	.	.	.	.	.	.	.
0.604 to 0.806	.	.	.	.	.	.	4	.	.	.	.	.	.	.	.	.	.	.	.	.	.
0.806 to 0.849	.	.	.	.	.	.	.	.	.	.	.	4	.	.	.	.	.	.	.	.	.
0.849 to 0.885	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	4	.	.	.
0.885 to 1.131	.	.	.	.	.	4	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.

Note: A dot indicates that the optimum maintenance remains unchanged from the last number entered above the dot. Current pavement condition state is 1.

Table 5. Optimal maintenance strategies and corresponding expected 1-year highway department costs for selected values of excess user cost scale factor for pavement category 2.

Initial Pavement State	Initial Probability Distribution	Optimal Maintenance Strategy for EXUCSF				Expected 1-Year Highway Department Costs for Each Pavement State ($/lane-mile/year)			
		θ = 0.25	θ = 0.50	θ = 0.75	θ = 1.00	θ = 0.25	θ = 0.50	θ = 0.75	θ = 1.00
1	0.045	1	1	1	1	55	55	55	55
2	0.100	1	1	1	1	72	72	72	72
3	0.200	1	1	1	1	109	109	109	109
4	0.150	1	3	3	3	170	11 501	11 500	11 500
5	0.100	2	3	3	3	6 341	11 634	11 634	11 635
6	0.004	2	3	3	4	7 340	11 806	11 806	16 337
7	0.001	3	3	4	4	12 015	12 015	16 892	16 392
8	0.005	1	1	1	1	127	127	127	127
9	0.040	1	1	1	1	148	148	148	148
10	0.100	1	1	3	3	196	196	11 689	11 689
11	0.100	3	3	3	3	11 784	11 784	11 784	11 784
12	0.050	3	3	3	4	11 919	11 919	11 919	11 947
13	0.004	3	3	4	4	12 090	12 090	16 992	16 992
14	0.001	3	4	4	4	12 300	17 047	17 047	17 047
15	0.001	1	1	1	1	312	312	312	312
16	0.002	1	3	3	3	330	12 471	12 471	12 471
17	0.003	1	3	3	3	369	12 532	12 532	12 532
18	0.030	3	3	3	4	12 628	12 628	12 628	17 377
19	0.030	3	4	4	4	12 763	17 412	17 412	17 412
20	0.030	4	4	4	4	17 457	17 457	17 457	17 457
21	0.004	4	4	4	4	17 512	17 512	17 512	17 512
Total for pavement category 2						3 999	6 405	7 579	7 992

Table 6. Limiting state probabilities if an optimal maintenance strategy (θ = 1.0) is followed for pavement category 2.

State	Average PMI (PSI, CI)	Limiting State Probabilities	State	Average PMI (PSI, CI)	Limiting State Probabilities
1	(4.5, 2.5)	0	12	(2.5, 12.5)	0.002
2	(4.0, 2.5)	0.478	13	(2.0, 12.5)	0.001
3	(3.5, 2.5)	0.276	14	(1.5, 12.5)	0.001
4	(3.0, 2.5)	0.047	15	(4.5, 35.0)	0
5	(2.5, 2.5)	0.013	16	(4.0, 35.0)	0.009
6	(2.0, 2.5)	0.009	17	(3.5, 35.0)	0.012
7	(1.5, 2.5)	0.009	18	(3.0, 35.0)	0.006
8	(4.5, 12.5)	0	19	(2.5, 35.0)	0
9	(4.0, 12.5)	0.064	20	(2.0, 35.0)	0
10	(3.5, 12.5)	0.049	21	(1.5, 35.0)	0
11	(3.0, 12.5)	0.024			

geographical or political influences is possible.

Limiting Behavior

The limiting behavior of a Markov process is the behavior that is expected after a long period of time. As previously noted, the Markov decision model defines optimal maintenance strategies as a function of the value placed on excess user costs. Consideration of the budget available to accomplish maintenance defines which of the optimum maintenance strategies can be accomplished. After the optimum maintenance strategy is defined, the maintenance alternative to follow for each initial state is known. Thus the optimal maintenance strategy defines the transition probability matrix applicable to the Markov process. After the 1-year transition probability matrix is defined, the theory of Markov processes can be used to define the limiting behavior of the pavement system in terms of the limiting state probabilities.

These limiting state probabilities define the probability of being in each state of the Markov process as time increases without bound. They can also be interpreted as the average fraction of time that the process is in each state. Thus the consequences of continually following the optimum maintenance strategy over a long period of time can be found. If the eventual condition state distribution is desirable, then strong motivation is provided to continually follow the optimum maintenance strategy. If the eventual condition state distribution is not acceptable, following an optimal maintenance strategy is pointless and additional maintenance alternatives should be considered.

The limiting state probabilities for an excess cost factor of 1.0 for pavement category 2 are given in Table 6. Seventy-five percent of the pavements in category 2 will eventually be in either states 2 or 3, which are highly desirable condition states. Furthermore, more than 96 percent of the pavements will have an average PSI rating of 3.0 or above. Only about 3 percent of the pavements will be in the largest average level of CI and those that are will possess average PSI values of 3.0 or more. For all lane-miles of pavements in pavement category 2, the expected values of PSI and CI are 3.7 and 4.8 respectively. Therefore, the eventual results of following an optimal maintenance strategy are highly desirable.

The expected 1-year highway department costs to follow an optimal maintenance strategy, after the pavements in pavement category 2 have achieved the limiting state probabilities, can be found in the same manner as when the initial probability distribution function described the pavement state probability distribution. The expected 1-year highway department costs at this time will be $2330/lane-mile. Recall that the expected first-year highway department costs to follow this same optimal maintenance strategy initially were $7992/lane-mile. Therefore, following an optimum maintenance strategy not only will minimize the expected present value of total future costs of an infinite time horizon and result in a pavement system that is in a highly desirable condition state but also will significantly reduce the expected 1-year highway department costs to maintain pavement category 2. For the initial probability distribution function assumed for pavement category 2, the expected 1-year highway department cost savings for maintenance would eventually be $5662/lane-mile. For 100 lane-miles of pavement, the expected savings would be in excess of $0.5 million.

SUMMARY AND CONCLUSIONS

This paper has demonstrated the applicability of a Markov decision model to the determination of optimal overlay maintenance strategies for flexible highway pavements. The pavement maintenance management system developed considers the stochastic nature of pavement behavior and defines optimal maintenance strategies based on minimization of the expected present value of total costs associated with a pavement. The management system also defines the implications of following an optimal maintenance strategy in terms of pavement condition and expected costs.

The Markov process representation of pavement behavior provides a meaningful and useful framework in which field performance measurements on pavements can be used. Furthermore, the field performance of thin, medium, and thick overlays can be evaluated to determine the degree to which they beneficially upgrade pavement condition, which can, in turn, be used to update the Markov process transition probability matrices through the use of Bayesian statistics.

The transition probability matrices representing each maintenance alternative in this study were quantified by using a Bayesian interpretation of probability. This allowed the experience of pavement engineers in observing the field performance of pavements to be directly incorporated into the Markov decision model. The subjective opinions of the pavement engineers were quantified through the personal implementation of a questionnaire that led to a rapid quantification of the Markov process model and provided an initial model.

From the results of this investigation, four conclusions appear warranted.

1. Optimal maintenance strategy was dependent on the value placed on excess user costs as measured by the excess user cost scale factor.
2. Level of maintenance in an optimal maintenance strategy increased as the value placed on excess costs increased.
3. For a given excess user cost scale factor, the level of maintenance that was optimal increased as the condition of a pavement deteriorated. For excess user cost scale factors greater than approximately 0.36, either medium or thick overlays were optimal if major maintenance was required.
4. The excess user cost scale factor is an important decision parameter that has the following significance: (a) It defines the optimal maintenance strategy that results when specific values are placed on excess user costs; (b) the value of the factor that corresponds to the optimal maintenance strategy followed defines the proportion of the true value of excess user costs that are implied by the maintenance strategy; (c) as the factor increased, the order of priority for performing maintenance was defined, and this priority order indicates that thin overlays are seldom optimal major maintenance strategies and only occur at very low excess user cost scale factors; and (d) the optimal maintenance strategies defined for specific factors provide a means to obtain the optimal maintenance strategy that will meet a constrained budget for the performance of maintenance.

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Further Refinement of Louisiana's Maintenance Cost Formulas

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This paper deals with the development of a procedure for reliably predicting highway maintenance requirements. Such a procedure would materially aid highway maintenance management in planning maintenance activities. The least squares analysis technique applied to a data base derived from the historical records maintained by the Louisiana Department of Highways yielded a series of models that adequately estimated maintenance requirements for planning purposes.

Since 1930, the Transportation Research Board (formerly the Highway Research Board) has spent a considerable amount of time and money in analyzing maintenance costs and relating the costs to causal factors. Large-scale research projects have been undertaken in which considerable amounts of historical and field data were collected and subsequently analyzed. The federal government, most state governments, and numerous local agencies have been continually working to find better methods of estimating maintenance costs. Those efforts made by the states of Louisiana, Arizona, Ohio, Idaho, and Virginia are noteworthy. Although a considerable amount of subjective and analytical study at federal, state, and local levels has been done in this area, little effort to date has been made to model different categories by grouping maintenance costs with respect to estimating their future requirements. In 1956, the Louisiana Department of Highways attempted to analyze maintenance costs by a quantitative method. A prediction method was developed that considered the age and later the roadway surface condition, traffic volume, subgrade classification, width of roadway surface, and right-of-way width.

Radzikowski (1, 2, 3, 4, 5, 6) wrote several reports on maintenance costs. Initial reports discussed the analysis of various maintenance cost data and suggested measures to be taken to reduce maintenance costs. In 1956, Radzikowski published a report describing a technique almost identical to the Louisiana Department of Highway's method, but he considered different variables in defining the base mile.

Sutarwala and Mann (7, p. 20) were the first to develop a conceptual mathematical model in the form of an equation that could predict the yearly maintenance cost of a given mile of roadway section. The equation developed was applicable only to the concrete pavements within the state of Louisiana. Two important conclusions were derived from this research effort. First, not only were the maintenance costs found to be independently related to the influencing variables themselves but also the interrelationship among the variables made an important contribution toward the estimation of maintenance requirements. Second, an assumption of only linear relationships between the maintenance costs and related variables was proved wrong, and cross product and nonlinear terms were found to be necessary to explain more fully the variation in maintenance requirements.

Mann (8) continued to work in this area. The initial model was modified in a way so that the adequacy of maintenance could be ensured. This was achieved by first evaluating the competence of the involved maintenance engineers in predicting maintenance costs first by using hypothetical sections for estimating maintenance expenditures and then by asking the maintenance engineers to estimate adequate maintenance expenditures on some selected physical sections in their jurisdictions. Adjustments that were necessary to correct deviations from uniform adequacy standards were made on these estimates. Thus reliance on what was spent as a measure of adequate maintenance was eliminated.

Betz (9), in a 1965 publication, reviewed all the research done in this field and pointed out the importance of such research work to developing countries. He concluded that the interractions and relationships among the influencing variables and maintenance requirements were complex in nature and any forced attempt to simplify them by trying to relate them independently would distort the validity of the models.

DATA PREPARATION

The data collected for this investigation were derived from different maintenance data files of the Louisiana

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Department of Highways. Some of the design data were taken from research reports published by the Research and Development Section of the Louisiana Department of Highways. Field observations were found to be unnecessary.

A preliminary investigation was made to ensure that all the data required to develop the models would be available. In certain cases, information could not be taken directly from the existing files and had to be mathematically derived from the raw data. Apart from the maintenance cost data by different categories, most of the other data were easily available and reliable. Certain assumptions were made concerning the available data.

1. Since the institution of recommendations made by a consultant's study, uniformity in adequate maintenance work has been achieved.
2. As a result of item 1, most of the maintenance cost data and performance data recorded during the previous 15 years are adequate and, to a great extent, reliable.
3. As a consequence of the initial assumption, it is assumed that control sections on which maintenance costs have assumed a regular pattern during the previous 5 years were adequately maintained. This assumption is justified because this research is primarily concerned with preventive maintenance.
4. Whenever the necessary data based on control section were not available (as was the case for all categories except surface maintenance and shoulder and approach maintenance data), the average of parishwide data for each of the highway functional classes was used.
5. Where conflict arose between data independently collected by the Maintenance Section and other sections of the Louisiana Department of Highways, the data recorded by the Maintenance Section were used.

DEVELOPMENT OF MAINTENANCE MODELS

Seven statistical models were developed by use of the least squares method. These models estimate maintenance costs requirements in dollars for the various categories as will be explained. The first five models are applicable to any control section; the models for (a) river-crossing operations and (b) maintenance overhead and administration are applicable at the parish and district levels respectively. The specific form of each model is given in the following sections. (These models are designed for U.S. customary units only; therefore, values are not given in SI units.)

Surface Maintenance

Surface maintenance cost per centerline mile = 19.6 + 177.9 (percentage of asphalt pavement) + 0.06 (percentage of concrete pavement) (average daily traffic) + 4.3 $(\mathrm{ADT})^{1/2}$ - 0.01 (ADT) (structural number). Coefficient of determination (r^2) = 0.86.

Shoulder and Approach Maintenance

Shoulder and approach maintenance cost per centerline mile = 17.5 + 284.7 (percentage of paved shoulder) + 13.8 (percentage of nonpaved shoulder) (soil support value) - 15.6 (percentage of paved shoulder) (soil support value) + 7.2 (age) - 55.3 (soil support value) + 1.9 $(\mathrm{ADT})^{1/2}$. $r^2 = 0.81$.

Roadside and Drainage Maintenance

Roadside and drainage maintenance cost per centerline mile = 50.8 + 909.5 (Interstate) + 151.0 (primary) + 3.9 (annual rainfall) + 22.5 (acres mowed). $r^2 = 0.70$.

Structure Maintenance

Structure maintenance cost per centerline mile = 42.9 + 2257.5 (number of steel structures) + 9.3 (length of steel structures) - 0.3 (deck area concrete and steel structures) + 0.3 (deck area of other structures). $r^2 = 0.50$.

Traffic Surface Maintenance

Traffic surface maintenance cost per centerline mile = 81.2 + 572.7 (Interstate) + 90.5 (primary) + 0.03 (urban traffic factor) + 0.04 (rural traffic factor). $r^2 = 0.86$.

River-Crossing Operations Maintenance

River-crossing operations maintenance cost per parish = 3608.9 + 5155.6 (number of bridge tenders) (salary increment factor) + 4516.4 (number of ferry tenders) (salary increment factor) + 8.1 (annual bridge openings) + 2957.5 (total ferry capacity). $r^2 = 0.98$.

Maintenance Overhead and Administration

Maintenance overhead and administration cost per district = 149 566.6 + 846.4 (total number of employees in district) (salary increment factor) + 84 251.4 (district centerline mileage per nonadministrative employee). $r^2 = 0.71$.

CONCLUSION

The development of adequate mathematical models for predicting various categories of maintenance cost requirements creates an extremely useful tool because intuitive and subjective estimates of needs by individual maintenance engineers may be verified. Although use of such models will not eliminate the problems of overmaintenance and undermaintenance, estimated maintenance requirements should be more consistent by being correlated to the causal factors that generate maintenance activities, which in turn will require more accountability in maintenance expenditures. It is important to note that generally these models were developed from parishwide or districtwide data and their application to any single short section must be done with caution. The effect of yearly inflation, although included as a salary increment factor in two of the models, must be considered in applying the models presented over time.

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Strategic Planning for Pavement Rehabilitation and Maintenance Management System

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A strategic planning scheme has been developed for a pavement rehabilitation and maintenance management system. Four usual phases compose the analysis framework: (a) problem analysis and data collection, (b) formulation of the mathematical model, (c) optimization, and (d) analysis of the solution. Matrices of roadway inventory, current pavement condition rating, gain of rating, pavement survival rate, minimum rating requirement, resource requirement, and resource availability are assembled in phase 1. In phase 2, the resource allocation problem is formulated in a zero-one integer linear programming model that maximizes the overall effectiveness of all proposed maintenance and rehabilitation activities subject to the following constraints: decision variable constraints; available supplies, equipment, work force, overhead, and state betterment budget constraints; and minimum distress rating and pavement rating constraints. The formulation of several other constraints, such as social, political, and geometric constraints, is also discussed. In phase 3, the mathematical model is used to determine the optimal maintenance strategy for each highway segment included in the analysis. Solutions are analyzed in phase 4. The formulation of this resource allocation method for strategic planning of maintenance and rehabilitation provides a basic framework within which management decisions can be made and altered while the effect that those decisions will have on the quality of pavements within a highway network can be fully recognized.

Funding for highway maintenance operations can be expected to become more stringently controlled in the future. In addition, highway management decisions will be greatly affected by new social attitudes toward the use of scarce natural resources; environmental impact; and human responses, values, and preferences. Strategic planning for the optimal allocation of limited resources will result in a significant amount of economic saving. Thus a systematic methodology is urgently needed to establish priorities for the optimal investment of available resources while satisfying the demands of the public for quality highway pavements. Moreover, the use of analytical techniques for the determination of optimal resource allocation policies for a given highway system can identify maintenance practices that can potentially save money by using money more effectively.

Management scientists have developed many mathematical models for resource allocation optimization. In operations research, resource allocation problems can usually be formulated in two alternative optimization schemes: (a) maximize the overall effectiveness subject to limited resources or (b) minimize the use of resources subject to minimum requirements of effectiveness. The former scheme is adapted herein for the system development because the current maintenance budget systems seem more consistent with maximizing effectiveness than with minimizing the use of resources. However, conversion from one formulation to the other can easily be accomplished. In addition, the methodology based on zero-one integer linear programming techniques may be readily applicable for the resource allocation of a highway maintenance system.

The purpose of this paper is to present basic concepts required for the development of a comprehensive pavement rehabilitation resource allocation system. A conceptual model based on the zero-one integer linear programming algorithm is presented. Special emphasis has been placed on (a) evaluation and rating of the condition of current pavement distress; (b) demands for pavement performance and service life; (c) effectiveness of different maintenance strategies and pavement maintenance survival rates; (d) requirements and availability of materials, supplies, equipment, work force, and overhead costs for pavement maintenance and rehabilitation; and (e) various budgetary constraints.

The terms used in the paper are defined below.

GLOSSARY

1. Analysis period is a period selected to be longer than any maintenance or rehabilitation method, including reconstruction, and expected to last without requiring additional maintenance work.
2. Types of distress are manifestations of pavement distress categorized by visual rating and other forms of evaluating pavements and include cracking, patching, rutting, roughness, raveling, flushing, and corrugations.
3. Environmental factors are soil or climatic variables that affect maintenance effectiveness. Soil variables include expansive clays and shales and frost heave Climatic variables include temperature and moisture

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cycling and poor drainage.

4. Highway network is the overall highway system included in the analysis.

5. Highway section is the conventional division of a highway system.

6. Highway segment is a portion of a highway section or a combination of several sections such that a segment can be treated as a unity in the analysis.

7. Maintenance activity is a general term for all work done to restore quality to a given pavement condition. In a more restricted sense, maintenance activity refers to the less extensive work done to upgrade the condition of a pavement.

8. Maintenance effectiveness is the sum of probabilities of retaining the overall gains of pavement rating due to maintenance activity on all highway segments over the analysis period.

9. Maintenance strategies are different activities to be selected for each highway segment in the analysis to increase the pavement rating above specified minimum requirement.

10. Pavement condition rating is the score of pavement quality for each type of distress of each highway segment.

11. Pavement rating score is a sum of all of the pavement condition ratings for each form of distress.

12. Pavement survival is the retention of a pavement condition rating higher than the pavement had when maintenance was applied.

13. Potential gain of rating is the expected increase of pavement rating score when a given type of maintenance or rehabilitation is applied.

14. Rehabilitation activity is a term reserved for the more extensive repair or reconstruction work done to a pavement to return it very nearly to its original structural capacity or safety condition.

15. Survival probability is the likelihood that a given maintenance strategy will retain its gain in pavement rating score over the analysis period when it is applied to a pavement with a given type of distress.

16. Traffic condition is the volume and weight of the traffic loading applied to a pavement.

MANAGEMENT PLANNING AND CONTROL SYSTEMS

In pavement maintenance management, strategic planning for resource allocation at the district highway department level is based on the policies and guidelines prescribed by federal and state transportation administrations. The objective at the district level is to maximize the total effectiveness of all maintenance and rehabilitation activities scheduled for the next year. Strategic planning is essentially built around a financial structure to provide the most cost-effective decisions on maintenance strategies so that all highway segments within the district can be maintained above a specified level of serviceability for normal driving. Guidelines set by strategic planning will eventually be carried out by district engineers for management control and construction supervisors for operational control.

A strategic planning scheme for pavement maintenance and management has been developed in four phases: (a) problem analysis and data collection, (b) formulation of the mathematical model, (c) optimization, and (d) analysis of the solution.

PHASE 1: PROBLEM ANALYSIS AND DATA COLLECTION

Phase 1 of the strategic planning for a pavement maintenance management system is the task of problem analysis and data collection that is categorized into four subtasks: (a) management decision, (b) roadway description, (c) pavement condition, and (d) resource information.

Management Decision

Management decisions determine the number of highway segments that will be considered in a highway network, the number of maintenance strategies that will be employed, the number of types of distress to be included in determining the current condition of all highway segments, and the analysis period for planning and control.

Highway Segment

One highway segment can be a portion of a highway section or a combination of several sections such that a segment can be treated as a unity in the study. The traffic condition and environmental factors that affect the effectiveness of maintenance and rehabilitation activities within the unity must be very similar if not identical. Then the strategic planning system will select an optimal maintenance strategy for each unity, that is, each highway segment specified by the decision maker. Highway sections that are expected to provide acceptable serviceability and require no maintenance during the next year need not be included in the scope of the study.

Maintenance Strategies

Undoubtedly, numerous practical applications of maintenance strategies can be listed. However, the more strategies included in a given analysis, the more effort is required in assembling maintenance effectiveness data and in the mathematical programming of the problem. As a consequence, the current list has been restricted to 11 rehabilitation and maintenance strategies, from strip seal to reconstruction: strip seal, fog seal, chip seal, light patching and chip seal, extensive patching and chip seal, chip seal and planned thin overlay, plant mix seal or open-graded friction course, thin overlay [less than 5.08 cm (2 in) of asphalt concrete], moderately heavy overlay [5.08 to 7.62 cm (2 to 3 in) of asphalt concrete], heavy overlay [7.62 to 15.24 cm (3 to 6 in) of asphalt concrete], and reconstruction. These strategies are listed in order of increasing unit cost. Usually, the first 4 strategies are funded from the state maintenance budget. Funding for the next 4 strategies is either from state maintenance budget or from the betterment budget. The last 3 strategies are funded from the betterment budget as contract work.

Pavement Distress

Usually, pavement distress manifestations can be categorized into the following 9 types: rutting, raveling, flushing, corrugations, roughness, alligator cracking, longitudinal cracking, transverse cracking, and patching. This classification has been used in several visual rating systems for evaluating pavements (1, 2, 3, 4).

Analysis Period

A heavy overlay will, undoubtedly, last longer than seal coats when applied to the same highway pavement. To calculate the overall effectiveness of all maintenance activities, one must analyze the pavement survival rates over a specified time period. An analysis period should be selected to be longer than any maintenance or rehabilitation method, including reconstruction, is expected to last without requiring additional maintenance work. A period of 10 years is recommended for analysis. This does not mean that maintenance decisions and budgeting for the next 10 years will be studied. Instead, only the next year's maintenance strategies and budgeting will be determined, but their choice will be based on the effectiveness of each maintenance strategy within the given analysis period.

Roadway Description

After the number of highway segments to be considered in a resource allocation scheme is determined by management decisions, the type of pavement, length, width, traffic, and environmental conditions of each segment can be established. The roadway data collected on each highway segment can be organized into a roadway inventory matrix. Traffic and environmental multiplying factors can be used to increase with traffic and climatic conditions that accelerate the appearance of various forms of distress. The formulation of these two factors will be discussed subsequently.

Pavement Condition

The pavement condition can be analyzed in the following aspects:

1. Current pavement condition rating of each segment for each type of distress,
2. Potential gains of rating of each segment for each maintenance strategy and type of distress,
3. Pavement survival rate of each maintenance strategy for each type of distress and time period on each type of pavement,
4. Minimum rating requirement of each segment for each type of distress and time period, and
5. Rating requirement of each segment and time period.

Current Pavement Condition Rating

Several pavement condition rating systems currently in use (1, 2, 3, 4) are readily applicable to the resource allocation model developed herein. By using such a rating system, one can fill out a current pavement condition rating matrix (Figure 1) based on the rating of each highway segment and each type of distress.

Potential Gain of Rating

Potential gain of rating is defined as the net expected increase of pavement rating of each segment for each type of distress and maintenance strategy. The potential gain of rating for a given kind of distress cannot exceed the amount of rating that it lost by that form of distress. A gain-of-rating matrix (Figure 2) is devised for each highway segment. When the number of segments gets large, the task of composing this collection of matrices can be done most efficiently by computer. Some maintenance strategies might not improve but rather might reduce the pavement ratings of certain types of distress. As an example, seal coating does not improve rutting, and a fog seal may accentuate flushing. In these cases, a zero or negative gain of rating will be required.

Pavement Survival Rate

Figure 3 shows a pavement survival matrix that contains the survival probability of each highway segment for each type of distress and maintenance strategy over the analysis period. Where maintenance and rehabilitation are concerned, the term "survival" indicates that the pavement condition is still expected to be rated high enough not to require additional maintenance or rehabilitation work. For instance, for a specific highway segment i, maintenance strategy j, and type of distress k at t years after maintenance work has been performed, a typical survival rate P_{ijkt} may be as follows:

$$P_{ijkt} = \begin{cases} 1.00 & \text{if } t = 0 \text{ year} \\ 0.90 & \text{if } t = 1 \text{ year} \\ 0.70 & \text{if } t = 2 \text{ years} \\ 0.40 & \text{if } t = 3 \text{ years} \\ 0 & \text{if } t \geq 4 \text{ years} \end{cases} \tag{1}$$

This example indicates that the pavement survival rate is 100 percent immediately after the maintenance work is accomplished; 90 percent, 70 percent, and 40 percent respectively at the end of the first, second, and third years; and 0 percent at the end of the fourth year. Suppose the current rating of a specific type of distress k is 5 and the gain of rating of type of distress k is 15 if maintenance strategy j is applied; the rating after the maintenance is done is 20. The rating drops to 18.5, 15.5, and 11 respectively, at the end of the first, second, and third years. The rating will return to 5 after the end of the fourth year.

The maintenance effectiveness when strategy j is applied per unit surface area of highway segment i when type of distress k is present is defined as

$$\sum_{t=1}^{N_T} d_{ijk} P_{ijkt} \tag{2}$$

where

d_{ijk} = potential gains of pavement rating of highway segment i for maintenance strategy j and type of distress k;
P_{ijkt} = pavement survival probability of highway segment i for maintenance strategy j and type of distress k at time t; and
N_T = number of years in analysis period.

Estimating the potential gains of rating of each highway segment can be a painstaking process for highway engineers. For instance, if 100 highway segments are considered in the analysis framework, the data for 100 gain-of-rating matrices as shown in Figure 2 must be assembled. This problem may be simplified by categorizing the existing pavements into several major types, such as (a) surface treatment pavement, (b) hot-mixed asphalt concrete (HMAC) pavement without overlay, and (c) HMAC overlaid pavement. The gain of rating of the three types of pavement at typical traffic and environmental conditions can thus be used to compose three basic matrices. The gain of rating of each individual highway segment can now be derived by multiplying a traffic adjustment index and an environmental adjustment index to the basic matrix. The maintenance effectiveness can be rewritten as

$$\sum_{t=1}^{N_T} D_{njk} \times \max[1 - a_i b_i (1 - P_{ijkt}), 0] \tag{3}$$

where

D_{njk} = potential gains of pavement rating of maintenance strategy j and type of distress k if highway segment i is type of pavement n,
a_i = traffic adjustment index of highway segment i, and
b_i = environmental adjustment index of highway segment i.

The master matrix of probability of pavement survival P_{ijkt} represents characteristic survival curves that may be modified by different traffic volumes and environmental effects. The characteristic curves should be the highest expected probabilities within a given district so

Figure 1. Current pavement condition rating matrix.

DISTRESS TYPE	HIGHWAY SEGMENT NUMBER 1	2	3	... ...	N_H
1					
2					
3					
...					
...					
N_D					

N_H - the total number of highway segments in analysis.

N_D - the total number of distress types.

Figure 2. Gain-of-rating matrix of each highway segment.

MAINTENANCE STRATEGY	DISTRESS TYPE 1	2	3	... ...	N_D
1	12	10	8		10
2	10	10	7		8
3	8	10	6		6
...					
...					
N_S	0	5	-4		-6

N_S - the total number of maintenance strategies

N_D - the total number of distress types.

Figure 3. Pavement survival matrix of each highway segment and type of distress.

MAINTENANCE STRATEGIES	TIME AFTER MAINTENANCE 0	1	2	3	4	... ...	N_T
1	1.00	.90	.70	.40	0.0		
2	1.00	.80	.50	.20	0.0		
3	1.00	.60	.20	0.0			
...							
...							
N_S	1.00	0.50	0.0				

N_S - the total number of maintenance strategies

N_T - the analysis period in years.

that the adjustment factors, a_t and b_t, will always be one or greater. Thus an increase in a_t or b_t will represent increasingly heavier traffic loading or more severe environmental conditions and will reduce the probability of survival. This is shown in Figure 4.

Minimum Rating Requirement

Figure 5 shows a rating requirement matrix of each highway segment. Some instructions for preparing the matrix need to be given.

1. For highway segment i and type of distress k, the rating requirement $R_{ikt} = 0$ if $t \geq T_{ik}$. T_{ik} is the expected service life of the maintenance activity of the next year for that highway segment and type of distress; that is, another maintenance activity will be scheduled for highway segment i at or before time T_{ik}.
2. For highway segment i, the total rating requirement of all types of distress $W_{it} = 0$ if $t \geq T_i$. T_i is the expected service life of the maintenance activity of the next year for highway segment i. Another maintenance activity will be scheduled for this segment at or before time T_i.
3. The total rating requirement is not necessarily the sum of the rating requirements of all types of distress. Usually, for highway segment i and at time t,

$$W_{it} > \sum_{R=1}^{N_D} R_{ikt} \tag{4}$$

where N_D = total number of types of distress in the analysis. The constraint of total rating requirement is unnecessary if

$$W_{it} < \sum_{k=1}^{N_D} R_{ikt} \tag{5}$$

Resource Information

The resource allocation scheme to be described is especially devised for annual budgeting and management. However, a substantial degree of flexibility for decision making has been retained. For instance, seasonal (or even monthly) reviews of the selected maintenance strategies are strongly encouraged so that inflated costs and the scarcity of resources as well as the need for changing pavement rating score requirements can be included in the management analysis framework to alter or justify previous maintenance decisions.

Resources for pavement maintenance and rehabilitation can be categorized in the following groups: (a) material and supply, (b) equipment, (c) personnel, (d) district overhead cost, and (e) betterment budget for contract work. First of all, the number of types of material, equipment, and work forces must be identified. In light of the availability of the resources and the design engineer's preference, the types of materials, equipment, and work forces adopted and used for maintenance and rehabilitation in one district are not necessarily the same as those adopted and used in another district.

PHASE 2: MATHEMATICAL MODEL

Phase 2 of strategic planning for the pavement maintenance and management system is to formulate the mathematical model. The objective of the resource allocation model for highway maintenance is to maximize the

Figure 4. Effects of traffic and environmental adjustment indexes.

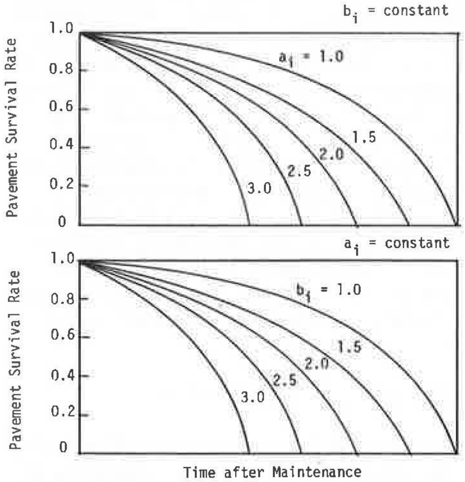
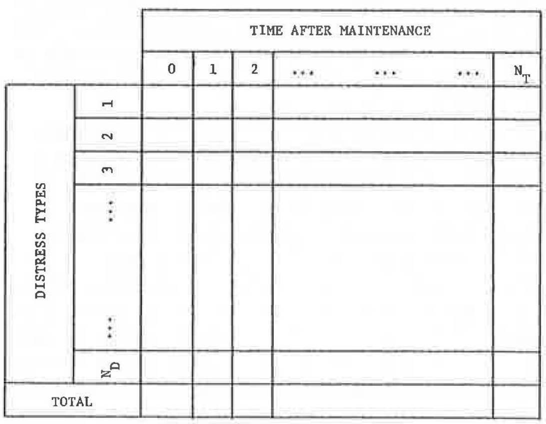


Figure 5. Minimum rating-requirement matrix of each highway segment.



N_D - the total number of distress types.

N_T - the analysis period in years.

overall effectiveness of maintenance activities subject to constraints such as limited resources and minimum requirements of pavement quality and service life. Mathematically, the most basic and simplified form of this problem is written as follows.

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S}\sum_{k=1}^{N_D}\sum_{t=1}^{N_T} L_{1i}L_{2i}d_{ijk}P_{ijkt}x_{ij} \qquad (6)$$

subject to seven constraints. The decision variable constraint is

$$\sum_{j=1}^{N_S} x_{ij} \leq 1 \qquad i = 1, 2, \ldots, N_H \qquad (7)$$

The available supplies constraint is

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S} s_{ijg}L_{1i}L_{2i}x_{ij} \leq S_g \qquad g = 1, 2, \ldots, N_G \qquad (8)$$

The available equipment constraint is

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S} e_{ijf}L_{1i}L_{2i}x_{ij} \leq E_f \qquad f = 1, 2, \ldots, N_F \qquad (9)$$

The available work force constraint is

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S} h_{ijq}L_{1i}L_{2i}x_{ij} \leq H_q \qquad q = 1, 2, \ldots, N_Q \qquad (10)$$

The available overhead is

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S} C_{ij}L_{1i}L_{2i}x_{ij} \leq C \qquad (11)$$

The minimum rating for each type of distress is

$$r_{ik} + \sum_{j=1}^{N_S} d_{ijk}P_{ijkt}x_{ij} \geq R_{ikt} \qquad i = 1, 2, \ldots, N_H;\ k = 1, 2, \ldots, N_D;\ t = 0, 1, \ldots, N_T \qquad (12)$$

The minimum overall pavement rating score is

$$\sum_{k=1}^{N_D}\left\{r_{ik} + \sum_{j=1}^{N_S} d_{ijk}P_{ijkt}x_{ij}\right\} \geq W_{it} \qquad i = 1, 2, \ldots, N_H;\ t = 0, 1, \ldots, N_T \qquad (13)$$

where

N_H = number of highway segments in analysis,
N_S = number of maintenance strategies,
L_{1i} = pavement length of highway segment i in kilometers,
L_{2i} = pavement width of highway segment i in meters,
x_{ij} = a decision variable that will be 1 if maintenance strategy j is selected for highway segment i and 0 otherwise,
s_{ijg} = amount of type g material (or supply) per unit surface area [1.6 km (1 mile) long and 0.3 m (1 ft) wide] required in highway segment i if maintenance strategy j is selected,
S_g = total amount of type g material (or supply) available,
N_G = number of different types of material (or supply),
e_{ijf} = amount of type f equipment required in highway segment i if maintenance strategy j is selected, in equipment days per unit 1.6 km long (1 mile long) and 0.3-m-wide (1-ft-wide) surface area,
E_f = total amount of type f equipment available in equipment days,
N_F = number of different types of equipment,
h_{ijq} = amount of type q work force required in highway segment i if maintenance strategy j is selected, in person-days per unit 1.6 km long (1 mile long) and 0.3-m-wide (1-ft-wide) surface area,
H_q = total amount of type q work force available in person-days,
N_Q = number of different types of work forces,
C_{ij} = overhead cost in dollars per unit surface area 1.6 km (1 mile) long and 0.3 m (1 ft) wide required in highway segment i if maintenance strategy j is selected,
C = total overhead budget available in dollars,
r_{ik} = current pavement rating of highway segment i and type of distress k,
R_{ikt} = minimum required pavement rating of highway segment i and type of distress k at time t,
W_{it} = minimum required pavement rating of highway segment i of all types of distress at time t.

Equation 6 is the objective function that maximizes the overall effectiveness of all maintenance activities. Equation 7 is the feasibility constraint that ensures that at most one maintenance strategy will be selected for the highway segment i because each x_{ij} can have a value of only 0 or 1. Equations 8, 9, 10, and 11 represent respectively the resource availability constraints of material, equipment, work force, and overhead budget. The pavement rating requirement constraints of each individual type of distress and all types of distress are represented by equations 12 and 13.

As has been mentioned, this is a basic and simplified model. To elaborate the model for more versatile and realistic applications, many other variables and constraints can be incorporated. For instance, for political or social reasons, keeping the labor forces busy may be necessary. This constraint can be written as follows:

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S} h_{ijq} L_{1i} L_{2i} x_{ij} \geq H'_q \qquad q = 1, 2, \ldots, N_Q \tag{14}$$

in which H'_q is the minimum required amount of q type of work force in person-days to be used during the year. Also the minimum amount of each specific material type g, which is S'_g, that must be used during the year can be constrained as follows:

$$\sum_{i=1}^{N_H}\sum_{j=1}^{N_S} S_{ijg} L_{1i} L_{2i} x_{ij} \geq S'_g \qquad g = 1, 2, \ldots, N_G \tag{15}$$

The need to keep the pavement rating score of certain highway segments higher than other segments, for political or geometric reasons, can be recognized in the constraints by raising the R_{ikt} and W_{it} values.

PHASE 3: OPTIMIZATION

Many important problems find their mathematical models in linear programming forms with binary variables taking only the values of zero or one. Use of these binary values for the decision variable allows one to make the important decision to do nothing to a given pavement segment. The third phase of the strategic planning for the pavement maintenance and management system is to apply the zero-one integer linear programming algorithm to the mathematical model. The algorithm of zero-one integer linear programming can be found in many operations research textbooks (5, pp. 337-342). The computer code documented in reference (6, pp. 91-104) is readily applicable to the management of pavement maintenance. A maintenance-engineer-oriented input-output system can be added to the computer program to facilitate the implementation of this resource allocation system. A computer program to solve the set of equations in this paper has been written and has been used to establish maintenance priorities on small highway networks.

PHASE 4: ANALYSIS OF SOLUTION

Phase 4 of the strategic planning is the analysis of the solution. The mathematical model assembled in phase 2 is solved in phase 3. However, constraints on resource availability and pavement rating requirements may be too binding to obtain a feasible solution. When this is the case, a management decision is required to increase the availability of specific types of resources such as material, equipment-days, person-days, district budget, and betterment budget or to decrease the rating requirement of specific highway segments or both. After the reformulation of the resource availability or pavement rating requirement constraints, the zero-one integer linear programming algorithm is applied to the revised mathematical model. The problem feasibility is checked again. When infeasible, the procedure mentioned is iterated until a feasible solution is obtained. Then, the solution must be examined carefully by the maintenance engineer and top management. If unacceptable, it is necessary to go back to phase 1 of the strategic planning to reevaluate and readjust the problem analysis and data collection. For instance, certain highway segments may have to be deleted from the analysis because of the scarce resources and high rating requirements of other segments. No maintenance activities will be done to those deleted segments in the next year. In other words, maintenance activities are postponed.

CONCLUSIONS AND RECOMMENDATIONS

A consistent method of establishing priorities for maintenance and rehabilitation work on various segments of a highway network and remaining within management, material, and budgetary constraints is best accomplished by linear programming that uses a zero-one binary decision variable. This paper has presented a formulation of the maintenance resource allocation problem that can be adapted by any highway agency to its own needs. The formulation required the introduction of new concepts to describe the objective of better management of maintenance and rehabilitation resources.

The primary concept is that of "maintenance effectiveness," which is defined mathematically in equation 6. It is assumed that the maintenance management wishes to use all of the available monetary resources in maintaining a minimum acceptable pavement rating score on all segments of a highway network and in making the work that is done last as long as possible. Thus, the money and work force will be best used if there is a low probability of having to redo a given segment in the near future. In accordance with this idea, maintenance effectiveness is defined as the sum of probable increases of pavement rating score due to maintenance work on all highway segments in the network. This sum is a measure of the durability of the maintenance work. Of course, the more durable the maintenance is, the more effective is the maintenance strategy adopted.

To evaluate this overall effectiveness, one must first establish the effectiveness of different types of maintenance work in preventing the reappearance of different kinds of distress. This requires the composition of several maintenance strategy survival matrices that give the probability that a given maintenance strategy will retain its increased pavement rating score when it is applied to a pavement with a given type of distress. In particular, it answers questions such as, How likely is it that this overlay on this transversely cracked pavement will be cracked as severely 3 years from now? Obviously, answers to questions such as these are the subject of much ongoing research. But, at the present, values for such probabilities can be drawn from the opinions of experienced engineers and revised later by combining them with theoretical predictions by using Bayesian inference methods (7).

An essential part of this maintenance effectiveness scheme is the pavement rating method used. The consistency and practical effectiveness of this resource allocation scheme will depend heavily on the consistency of the rating method and of those who rate the pavements. Limited experience with pavement rating schools in Texas has indicated that careful training can substantially improve the rating consistency between teams as well as reduce personal bias.

The formulation of this resource allocation method for strategic planning of maintenance and rehabilitation provides a basic framework within which management decisions can be made and altered while fully recognizing the effect those decisions will have on the quality of pavements within a highway network.

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Pavement Maintenance and Repair Management System

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The main objective of the pavement maintenance and repair management system is to provide guidelines for the efficient and economic use of available maintenance funds. The system can be implemented manually or through the use of computers and consists of the following activities: (a) dividing the pavement network into manageable and consistent sections, (b) inspecting pavement sections, (c) recording significant pavement information, (d) determining maintenance needs and priorities, and (e) developing work plans. A flow chart of the system is shown in Figure 1. Detailed procedures and guidelines for performing the activities are available elsewhere (1, 2). The system is currently being implemented by the Alaska district of the U.S. Army Corps of Engineers and several U.S. Army installations.

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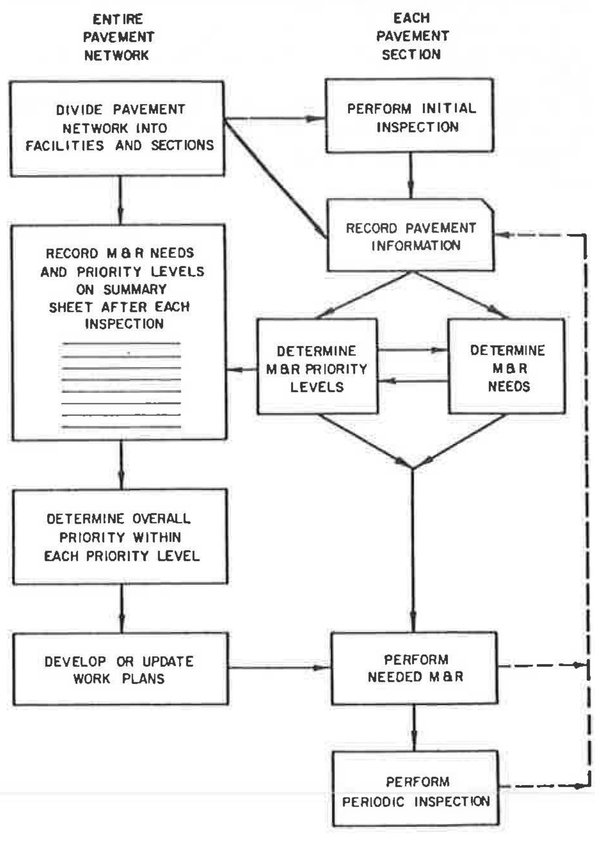
Figure 1. Pavement maintenance and repair management system.



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Maintenance Resealing of Rigid Pavement Joints

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Transverse joints in an existing portland cement concrete pavement were resealed to evaluate a number of sealers and sealing methods. Six liquids were installed, including asphalt cement, field-mixed rubber asphalt, two premixed rubber asphalts, polyvinyl chloride coal tar, and tar-modified polyurethane, as well as three preformed neoprenes. Liquid sealers were placed 50.8 to 63.5 mm (2 to 2½ in) deep in some joints and restricted to 12.7 mm (½ in) deep in others. Some joints were sandblasted to determine the effect of clean joint faces on sealer performance. Three types of joints were resealed: (a) 25.4-mm (1-in) expansion joints spaced at 30.48 m (100 ft); (b) 6.3-mm (¼-in) sawed contraction joints spaced at 18.54 m (60 ft, 10 in); and (c) 9.5-mm (⅜ in) formed contraction joints spaced at 18.54 m (60 ft, 10 in). Sealer performance has been evaluated over 3 winters, and pavement cores and sealer samples have been taken to observe sealer condition. All the neoprene sealers effectively seal the joints except along spalls. Some compression set has occurred in one brand but has not yet affected performance. The polyvinyl chloride sealer has generally performed best of the liquids, and the asphalt and field-mixed rubber-asphalt sealers generally performed poorly. Loss of adhesion between sealer and joint face is the most common failure mode for liquid sealers; entrapment of incompressible debris is a serious problem for asphalt and field-mixed rubber-asphalt sealers. Thorough joint cleaning was essential for some tested materials, but had no apparent effect on others. Generally, sealers placed 12.7 mm (½ in) deep did not perform as well as those placed 50.8 to 63.5 mm (2 to 2½ in) deep.

Transverse and longitudinal joints in rigid pavements are constructed to control expansion and contraction forces resulting from temperature and moisture changes. They are sealed to prevent intrusion of foreign materials that prevent them from functioning. Transverse joints are the more difficult to seal because they are subject to greater movements and are also very susceptible to damage if not properly sealed.

Keeping transverse joints in rigid pavements effectively sealed is a long-standing problem, and in New York it is aggravated by two factors. Joint spacings are very long [18.28 m (60 ft) or more], and the annual pavement temperature differential approaches 83.3°C (150°F). In combination, these factors cause large width changes in transverse joints between summer and winter, as great as 12.7 mm (½ in) in some cases.

New York currently specifies 31.7-mm (1¼-in) neoprene sealers in 15.9-mm (⅝-in) contraction joints, and they are giving good service. Before 1968, narrower neoprene sealers were used in contraction joints; asphalt sealers were used as well in both contraction and expansion joints. Because of the short life provided by these earlier sealers and the realization that the current sealer system will not last indefinitely, an effective resealing program is needed to ensure that transverse joints are sealed throughout the life of a pavement.

The standard maintenance sealer in New York has been 50-60 penetration grade asphalt, but its performance has been poor for a number of reasons. For the past several years, ground reclaimed rubber has been added to the asphalt during melting on a trial basis in an attempt to obtain better performance. The effectiveness of this additive had not been clearly demonstrated.

In 1972, the study reported here was initiated to identify transverse joint sealers and sealing procedures that might provide better performance than the standard asphalt sealer at reasonable cost. Six liquid and three preformed sealers were installed in nearly 300 transverse joints, and their performance has been evaluated over 3 winters.

INVESTIGATION

Selection of Test Sealers

Fifty-one sealer manufacturers were contacted to select sealers for the study. The sealers had to meet three general requirements: (a) they had to be easy to handle, prepare, and install without expensive equipment; (b) they had to be stable and nontoxic; and (c) they had to have a short cure time. Sixteen companies submitted data on 21 sealers, which were screened according to the following criteria:

1. Cost of the sealer (and primer, if required);
2. Ease of handling, mixing, and placing;
3. Stability and nontoxic qualities;

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4. Shelf life, pot life, and cure time;
5. Equipment required for mixing and installation;
6. Solvents and equipment required for cleanup;
7. Compatibility with asphalt (because most joints to be resealed had previously been sealed with asphalt, which is extremely difficult to remove completely from joint faces); and
8. Previous experience with the sealer in earlier studies.

In all, six liquid and three preformed sealers were selected (including two liquids then used by the department); sealer details are given in Table 1. (Names of the manufacturers of the products tested may be obtained from the authors.)

Test Joints and Their Preparation

Sealer test sections were installed in three areas of I-87 north of Albany, a six-lane divided portland cement concrete pavement. Three types of joints were sealed: (a) 25.4-mm (1-in) expansion joints spaced at 30.48-m (100-ft) centers; (b) 6.3-mm (¼-in) sawed contraction joints spaced at 18.54 m (60 ft, 10 in); and (c) 9.5-mm (⅜-in) formed contraction joints spaced at 18.54 m (60 ft, 10 in). All three types of joints are shown in Figure 1, and the highway sections are described in Table 2.

Sealers were placed in only one lane to simplify traffic control during installation and evaluation (the northbound median lane for the expansion joint section and the northbound driving lane for the two contraction types of joint). The joints for resealing were carefully screened to eliminate variable sealer performance caused by spalled and damaged joints or by midslab cracks that would influence joint movement. Brass pins were installed at each joint to monitor joint width changes during the test.

The old sealers were removed by several methods, including pickaxes, powered wire brushes, screwdrivers, and concrete saws. However, none was very efficient (an effective means of removing old sealers is needed). The expansion joints were cleaned to a minimum depth of 38.1 mm (1½ in), and the contraction joints were cleaned for the entire reservoir depth, which was approximately 63.5 mm (2½ in). All joints were blown out with compressed air after removal of the old sealers, and half were sandblasted to determine whether this extra cleaning would improve sealer performance. Half the contraction joints were filled full depth [63.5 mm (2½ in)] with the liquid sealers. In the other half, a filler material was used to restrict sealer depth to 12.7 mm (½ in).

Nine sealers, three types of joints, two joint preparation methods, and two sealer depths were included in the test. Not all combinations of variables were included, however. In addition to the reasons already mentioned, poor weather, equipment failure, and scheduling problems prevented completion of some planned installations. Preformed compression sealers were not placed in the 25.4-mm (1-in) expansion joints because the joints had spalls in which compression seals would have been ineffective and they would have required a sealer about 50.8 mm (2 in) wide, which would have been too expensive. The number of joints to be sealed for each combination of variables was set at five. This limited the total to be sealed and evaluated to a workable number, slightly less than 300. At the same time, five would provide a representative picture of sealer performance. Some variation in performance could be tolerated, and sealer capabilities could still be assessed validly.

Sealer Installation

In general, no serious problems were encountered while the hot-poured liquids were installed, but some had to be heated as long as 3 h before they reached pouring consistency. Minor difficulties were experienced with automatic joint-sealing equipment used with the premixed rubber asphalts and the polyvinyl chloride (PVC) coal tar sealers, but they were isolated instances and would not be expected to occur if these sealers were installed on a production basis. Rapid set times, all less than 30 min, were experienced for all the hot-poured sealers, making it possible to reopen the pavement to traffic soon after resealing was completed. Some problems were experienced with the cold-poured polyurethane sealer because of its long set time; it tended to run out of the low end of the joint groove and through the crack beneath it, making it difficult to fill the joint to the desired level. Because of its slow curing rate, tracking from vehicle tires was also experienced when the pavement was reopened. The preformed neoprene sealers were installed with hand rollers, but, in some very narrow joints, particular care was necessary to hold longitudinal stretch below the specified 5 percent limit, which in some cases was slightly exceeded.

Field Surveys

The joints were surveyed monthly for 2 years and quarterly for a third year. All changes in the sealers were noted, including oxidation or discoloration of the surface, surface cracking or crazing, air bubbles, sealer extrusion and tracking by traffic, depth of sealers in the joints, compression set and web sticking in the preformed sealers, and embedment or entrapment of foreign materials in the sealer or the joint. Total length of adhesion failure between the joint face and sealer and cohesion failure within the sealer were measured at each joint. Many types of failure are apparent from the surface, and their extent can be measured or estimated. Adhesion and cohesion failures, however, are difficult to assess unless in an advanced stage. These were counted when a thin-bladed putty knife could be inserted by applying only slight pressure between the sealer and joint face or in the separation in the sealer. More pressure could not be exerted because of the danger of creating failures where none had existed. Because both types of failure are related to joint width, the rates of failure were greatest in cold weather when joints were widest and varied greatly from survey to survey. Sealer failures from loss of adhesion and cohesion were defined as occurring when the total full-depth failure present exceeded 10 percent of the joint length, or 0.73 m (28.8 in) for two 3.66-m (12-ft) joint faces. Failures not extending the full depth of the sealer were noted but generally had little adverse effect on sealer performance and did not enter into the failure criterion. Because some variability in performance was observed, all five joints in a test group were rated together. In most cases, the 10 percent failure criterion was first exceeded in cold weather, and most sealers appeared to recover with warm weather. However, because intrusion had occurred, the sealer was still considered to have failed.

For liquid sealers, entrapment of incompressible materials was another common failure mode. This was estimated from surface observations supplemented by sampling of the sealers as will be explained later. Consequences of the other failure modes were small compared with the full-depth bond and cohesion failures or entrapment and had no practical influence on determining when failure occurred.

Full-depth core samples were obtained at the end of the second winter and again during the third winter from each joint group to check intrusion of foreign material. They also made it possible to assess the entrapment of foreign material in the liquid sealers and compression set and web sticking in the preformed sealers. Some cores were taken in areas of typical performance for a joint group and others were taken in areas that appeared to be in particularly good condition to determine whether sealers were actually performing as well as they appeared to be from the surface.

Some variation in joint movement was expected and must be accommodated by an effective sealer. Test joint widths were measured periodically to detect abnormal changes that might have pronounced effects on a particular joint or group of joints. As expected, expansion joints moved more because of their greater spacing, and substantial variation was apparent between joints. This variation, however, does not appear to be abnormal and no differences in sealer performance can be related to it.

SEALER PERFORMANCE

Asphalt Control Sealer

Asphalt cement, the department's standard sealer, performed poorly in all three test areas. Of 10 test sections, 3 failed during the first winter and 4 more failed during the second. Three remaining sections approached 10 percent full-depth failure during the third winter. Table 3 gives a summary of the performance of this and the other sealers. It is very temperature-susceptible; it becomes hard and brittle in winter and soft and sticky in warm weather. Massive adhesion failures in cold weather have been one failure mode—one expansion section failed during the first winter and the other failed during the second. The restricted-depth sealers in 6.3-mm (¼-in) contraction joints failed from massive adhesion failure during the first winter. Several more sections failed the same way during the second winter; by the third winter, three sections were still providing marginal service although each had exhibited considerable adhesion failure during cold weather as well as severe embedment and infiltration.

Chunks of this material also were lost during cold weather, and the surface became very irregular. Figure 2a shows the appearance of this sealer during the first winter with infiltration of foreign material into surface voids. With warmer weather and contraction joint closure, it appeared to recover (Figure 2b), but, when a section was removed (Figure 2c), extensive embedment and intrusion of foreign material were apparent. This embedment, accompanied by the joint closure, resulted in sealer extrusion (Figure 2d). Such material is worn away by traffic, and the following winter its capacity to seal is reduced even further.

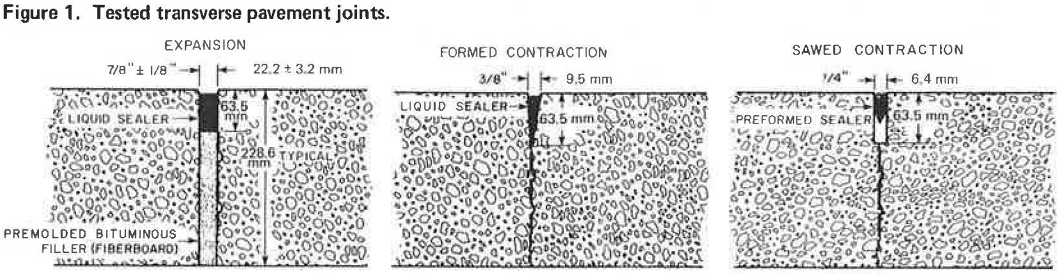
For the three test sections that failed during the first winter through adhesion or cohesion failure, embedment was not a factor. In all other sections, it was found during the first summer; at the end of the second winter, all displayed serious embedment and infiltration of foreign material. These sealers thus cannot be considered to be performing satisfactorily even though they have not

Table 1. Tested sealers.

Sealer	Description	Specification
Liquid	50-60 penetration grade asphalt cement	NYS item 702-50
	Field-mixed rubber asphalt	—
	Premixed rubber asphalt	ASTM D 1190-64 Federal specification SS-S-164(4)
	Premixed rubber asphalt	Federal specification SS-S-1401A
	PVC coal tar	ASTM D 3406-75T
	Tar-modified polyurethane	Federal specification SS-S-00200C
Preformed	Neoprene extrusion	ASTM D 2628-67T NYS item 705-12

Note: Neoprene extrusions made by three different manufacturers were tested.

Figure 1. Tested transverse pavement joints.



Note: 1 mm = 0.0394 in.

Table 2. Test areas.

Test Area	Location	Slab Length (m)	Type of Joint	Joint Dimensions (mm) Width	Depth	Previous Seal
1	Mohawk River to Clifton Park	30.45	Hand-formed expansion	25.4	Full	Asphalt
2	Malta to Saratoga	18.54	Sawed contraction	6.3	63.5	14.4-mm neoprene seal
3	Glens Falls to Lake George	18.54	Insert-formed contraction	9.5	63.5	Asphalt

Note: 1 m = 3.28 ft. 1 mm = 0.0394 in.

reached 10 percent full-depth failure.

Other Liquid Sealers

In terms of full-depth failure, the field-mixed rubber asphalt material performed slightly better than the control asphalt sealer. Only one section, sealed 12.7 mm deep (½ in) in 6.3-mm (¼-in) contraction joints, failed during the first winter. Four more failed during the second winter, and all experienced considerable full-depth failures although five did not reach 10 percent. By the end of the third winter, three more sections had experienced complete failure and only two had less than 10 percent failure. The material exhibited slightly less embrittlement in cold weather than did the asphalt control sealer, but it became sticky in warm weather with extensive embedment and infiltration. When sampled after the second winter, all test sections displayed foreign material in and below the sealer. The same extrusion problems mentioned for the control sealer were experienced here. In winter, it was less susceptible to tearing out of large chunks than the more brittle control,

Table 3. Summary of sealer performance.

Sealer	Test Area	Joint Numbers	Sealer Depth (mm)	Sealer Width (mm)	Joint Preparation	Maximum Full-Depth Failure[a] Length (mm)	Maximum Full-Depth Failure[a] Date	Months in Service at Failure
Asphalt cement (control)	1	1 to 5	Full	–	Cleaned	3 937.0	11/73	13
	1	6 to 10	Full	–	Sandblasted	5 486.4[b]	4/73	6
	2	91 to 95	Full	–	Cleaned	2 463.8	2/74	30+[c]
	2	96 to 100	Full	–	Sandblasted	5 054.6	2/74	14
	2	101 to 105	12.7	–	Cleaned	10 668.0	2/73	3
	2	106 to 110	12.7	–	Sandblasted	7 315.2	2/73	3
	3	86 to 90	Full	–	Cleaned	5 232.4	2/74	16
	3	91 to 95	Full	–	Sandblasted	3 378.2	2/74	30+[c]
	3	96 to 100	12.7	–	Cleaned	4 622.8	2/74	16
	3	101 to 105	12.7	–	Sandblasted	1 701.8	2/74	30+[c]
Field-mixed rubber asphalt	1	11 to 15	Full	–	Cleaned	3 149.6	4/73	30+[c]
	1	16 to 20	Full	–	Sandblasted	4 064.0	2/74	16
	2	111 to 115	Full	–	Cleaned	8 712.2	2/74	16
	2	116 to 120	Full	–	Sandblasted	5 613.4	2/74	16
	2	126 to 130	12.7	–	Cleaned	4 749.8	2/74	4
	2	131 to 135	12.7	–	Sandblasted	3 378.2	2/74	30[d]
	3	111 to 115	Full	–	Cleaned	5 410.2	2/74	16
	3	116 to 120	Full	–	Sandblasted	2 159.0	2/74	30+[c]
	3	121 to 125	12.7	–	Cleaned	1 346.2	2/75	30[d]
	3	126 to 130	12.7	–	Sandblasted	2 032.0	2/75	30[d]
Premixed rubber asphalt 1	1	21 to 25	Full	–	Cleaned	914.4	2/74	30+
	1	26 to 30	Full	–	Sandblasted	2 463.8	2/74	30+
	2	51 to 55	Full	–	Cleaned	5 232.4	2/74	14
	2	56 to 60	Full	–	Sandblasted	4 368.8	2/74	15
	2	61 to 65	12.7	–	Cleaned	3 500.6	2/74	30+
	2	66 to 70	12.7	–	Sandblasted	3 683.0	2/74	14
Premixed rubber asphalt 2	1	31 to 34	Full	–	Cleaned	1 219.2	1/75	30+
PVC coal tar	1	41 to 45	Full	–	Cleaned	7 696.2	6/74	19
	1	46 to 50	Full	–	Sandblasted	1 219.2	1/75	30+
	2	141 to 145	Full	–	Sandblasted	3 479.8	2/75	30+
	2	146 to 150	12.7	–	Cleaned	5 689.6	2/74	15
	2	151 to 155	12.7	–	Sandblasted	2 006.6	2/74	30+
	3	11 to 15	Full	–	Cleaned	1 117.6	2/75	30+
	3	16 to 20	Full	–	Sandblasted	None	–	30+
	3	21 to 25	12.7	–	Cleaned	5 537.2	2/75	26
	3	26 to 30	12.7	–	Sandblasted	304.8	2/75	30+
Tar-modified polyurethane	1	51 to 55	Full	–	Cleaned	3 073.4	1/74	30+
	1	56 to 60	Full	–	Sandblasted	6 045.2	1/75	14
	2	1 to 5	Full	–	Cleaned	3 886.2	2/75	27
	2	6 to 10	Full	–	Sandblasted	5 969.0	2/75	27
	2	11 to 15	12.7	–	Cleaned	7 239.0	1/74	14
	2	16 to 20	12.7	–	Sandblasted	4 419.6	2/75	27
	3	131 to 135	Full	–	Cleaned	10 134.6	2/75	27
	3	136 to 140	Full	–	Sandblasted	609.6	2/75	30+
	3	141 to 145	12.7	–	Cleaned	6 375.4	2/75	14
	3	146 to 150	12.7	–	Sandblasted	1 549.4	2/75	30+
Neoprene 1	2	41 to 45[e]	–	13.7	Cleaned	965.2	2/74	30+
	2	46 to 50[e]	–	13.7	Sandblasted	762.0	2/74	30+
	3	51 to 55	–	25.4	Cleaned	457.2	2/74	30+
	3	56 to 60	–	25.4	Sandblasted	660.4	3/74	30+
Neoprene 2	2	31 to 35	–	10.6	Cleaned	1 041.4	2/74	30+
	2	36 to 40	–	10.6	Sandblasted	1 778.0	2/74	30+
	3	161 to 165	–	19.8	Cleaned	863.6	2/74	30+
	3	166 to 170	–		Sandblasted	1 117.6	2/74	30+
Neoprene 3	2	156 to 160	–	10.6	Cleaned	457.2	2/74	30+
	2	161 to 165	–	10.6	Sandblasted	457.2	2/74	30+
	3	1 to 5	–	19.8	Cleaned	406.4	6/73	30+
	3	6 to 10	–	19.8	Sandblasted	762.0	3/74	30+

Note: 1 mm = 0.0394 in.

[a]Total for five joints; criterion for 10 percent failure = 3657.6 mm (5 x 3657.6 x 2 x 0.1) [144 in (5 x 144 x 2 x 0.1)].
[b]3657.6-mm (144-in) bond failure in one joint; other four failed after 13 months.
[c]Failed through embedment.
[d]Failed July 1975.
[e]Joints 45 and 46 had 10.6-mm (7/16-in) sealer.

although some chunking did occur (Figure 3a).

Of the two premixed rubber asphalts, the first survived 3 winters in the 25.4-mm (1-in) expansion joints. Although less temperature sensitive than the control and field-mixed rubber asphalt, it too became brittle in cold weather and lost some sections in chunks. In the summer, although it became soft and extruded from the joints, it did not become sticky and subject to embedment and entrapment of foreign material. Although the 25.4-mm (1-in) expansion joints had not exceeded the 10 percent full-depth failure limit by the end of the third winter, they did not completely prevent infiltration of foreign material between the sealer and joint face (Figure 3b). In the 6.3-mm (¼-in) contraction joints, three of four test sections failed during the second winter from massive adhesion failure along the joint face, and the fourth nearly reached the 10 percent failure criterion. In spite of these adhesion failures, infiltration past the sealer (Figure 3c) was much less than for the control asphalt and the field-mixed rubber asphalt. Embedment in the sealer itself was also noticeably abent. Because of poor weather during installation, this sealer was not installed in the 9.5-mm (⅜-in) contraction joints.

Because of equipment problems, the second premixed rubber-asphalt installation was not completed before winter, and only four 25.4-mm (1-in) expansion joints were sealed. In terms of full-depth failure, it performed very much as the other premixed rubber asphalt and was still below 10 percent failure after 3 winters. Although it remained more pliable in cold weather, it too allowed some infiltration along the joint face. Although extrusion occurred in hot weather when the joints closed, it did not flow out of the joint where it could be worn away by traffic but rather remained intact (Figure 3d). Like the first rubber asphalt, the second succeeded in preventing embedment of foreign material within the sealer. Based on this small installation, the second premixed rubber asphalt appears to possess better material properties than the asphalt control or field-mixed rubber asphalt and is less susceptible to embedment. However, it does not completely seal a joint for a much

Figure 2. New York State standard liquid asphalt control sealer.

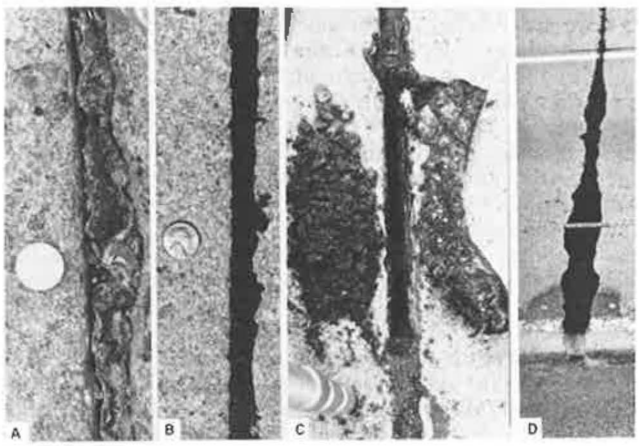


Figure 3. Experimental liquid sealers.

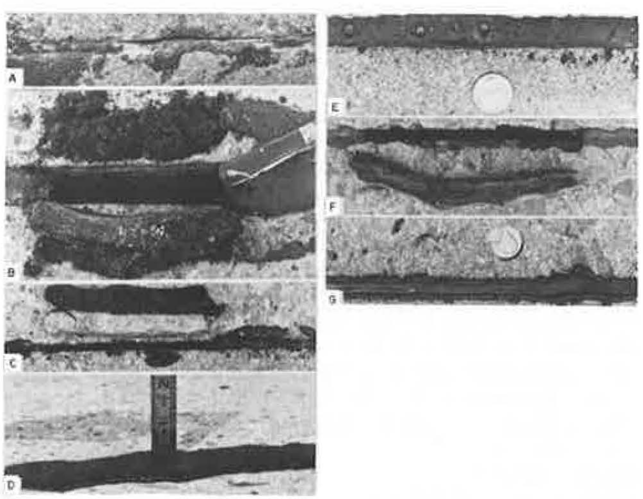


Figure 4. Preformed neoprene sealers.

Table 4. Sealer life expectancy and performance.

Sealer	Description	Expected Years of Service: Contraction	Expected Years of Service: Expansion	Temperature Susceptibility	Embedment Resistance	Failure Mode
Liquid	Asphalt cement	1 or less	1	Poor	Poor	Bond, embedment, extrusion
	Field-mixed rubber asphalt	1	1	Poor	Poor	Bond, embedment, extrusion
	Premixed rubber asphalt	1	3+	Good	Good	Bond
	PVC coal tar	3+	3+	Good	Excellent	Bond
	Tar-modified polyurethane	2 to 3	1 to 3	Excellent	Excellent	Bond, cohesion
Preformed	Neoprene 1	3	—	Excellent	Excellent	Spalls, compression set
	Neoprene 2	3+	—	Excellent	Excellent	Spalls
	Neoprene 3	3+	—	Excellent	Excellent	Spalls

greater period.

Two PVC sealers—both manufactured to the same specification—were intermixed during installation and have been evaluated as one. All test sections survived at least into the second winter, and the sandblasted sections were still in service after 3 winters. All failures took the form of full-depth adhesion failure between the sealer and the joint face. Other than small bubbles in its surface (Figure 3e) and very slight embedment of a few small stones in the surface, it had not deteriorated over 3 winters. The material is very resistant to changes in consistency caused by temperature. It remains pliable to very low temperatures and does not become soft and sticky in hot weather. Extrusion did occur when joints closed in hot weather in areas overfilled during sealing, but the sealer retained its integrity and did not track under traffic. Sandblasting has a pronounced effect on the performance of this sealer. In 25.4-mm (1-in) expansion joints that were not sandblasted, some adhesion failure appeared during the first winter and complete failure occurred during the second, but sandblasted joints were still performing satisfactorily into the third winter. The same is true in 6.3-mm (¼-in) joints; sandblasted joints are still in service after 3 winters, and the others failed during the second. In 9.5-mm (⅜-in) contraction joints, the 12.7-mm-deep (½-in-deep) sealers without sandblasting failed during the second winter, and full-depth joints without sandblasting have experienced some full-depth adhesion failure; considerable adhesion failure occurs along the top of the sealer to a depth of 6.3 to 12.7 mm (¼ to ½ in), but these joints have not reached the 10 percent limit. The sandblasted joints, at both full and restricted depth, are still in excellent condition, and have exhibited little or no failure. Except in areas of full-depth adhesion failure, these joints and sealers remain clean and free of intrusion and embedment (Figure 3f). Based on the excellent condition of some test sections after 3 winters, we anticipate that this sealer will provide several years of service before resealing is required.

Three sections of cold-poured polyurethane failed during the second winter and four more failed during the third. Of three still below the 10 percent criterion, one has nearly reached that limit and the other two have smaller amounts of adhesion failure. In addition to full-depth adhesion failures, others due to the placement problems were encountered. In some cases, multiple layers of material required to fill the joint separated, and chunks were lost. Foam dams used to retard sealer flow during installation were also starting places for failures. Otherwise, the sealer has retained its integrity over 3 winters, and its consistency has not been sensitive to temperature changes; in fact, in hot weather it remains the firmest of all the liquid sealers. When extrusion has occurred in hot weather, it has been similar to that experienced by the PVCs, the entire sealer being extruded above the pavement but remaining intact. In the contraction joints, sandblasting generally resulted in fewer full-depth failures. In the 25.4-mm (1-in) expansion joints, the sandblasted section failed during the second winter, but the nonsandblasted section is still below the 10 percent criterion; the difference in failure between the two sections, however, was not great until the third winter. One other defect noticed with this sealer and attributed to installation problems is its tendency to be low in the joint. In some cases, sealers still in good condition are 12.7 to 19.0 mm (½ to ¾ in) below the pavement surface, and dirt and small stones are trapped above even though the joints are still sealed. In other cases, the sealers remained at the surface of the pavement (Figure 3g). Several cores and sealer samples have been removed. As expected, infiltration was found in joints where failures were observed, but, in areas where sealers were functioning properly, no foreign material was found in the joint or embedded in the sealer.

Preformed Sealers

Their performance having been similar, the three preformed sealers will be discussed together. Through the end of the third winter, the only full-depth adhesion failures observed were in areas of joint spalling (Figure 4a). Nearly all of these spalls were present before resealing; some resulted from efforts to remove old sealers. Because very few full-depth adhesion failures have occurred, the evaluation is based heavily on cores and samples removed from these joints.

Samples of the first brand were obtained after the second winter. In the 6.3-mm (¼-in) joints, sides of the joint and sealer were clean without signs of infiltration past the sealer. Some evidence of compression set was noticed. In the 9.5-mm (⅜-in) joints, no compression set was observed, but silt was found in small quantities on the sides of the sealer. At the end of the second summer, this brand was cored in a 9.5-mm (⅜-in) joint; during the third winter, it was again cored in a 6.3-mm (¼-in) joint. The latter showed dirt penetrating the full depth of the sealer on both sides and considerable compression set. The other core, however, which was in a sandblasted joint, showed satisfactory performance and the lubricant adhesive was still working. In the 9.5-mm (⅜-in) joints, one not sandblasted again showed infiltration starting and compression set in the sealer (Figure 4b), but a sandblasted joint was again in good condition. Close examination after the third winter confirmed that compression set was occurring in several sealers, and dirt was infiltrating the sides of both sandblasted and nonsandblasted joints. The top of the sealer was also folded over in several areas, trapping dirt and small stones in the top of the joint (Figure 4c), but in no case was any appreciable amount of dirt found below the sealer.

The second preformed sealer was also sampled in four joints after the second winter [two 6.3-mm (¼-in) joints and two 9.5-mm (⅜-in) joints]. Although traces of silt were found along the upper portion of the sealer, all were in good condition and no compression set was observed. The following fall, one core was removed from a 9.5-mm (⅜-in) unsandblasted joint and the sealer was again found in good condition with only very slight infiltration on the joint sides. During the third winter, one core was removed from each test section; again, all were in good condition and there were only slight traces of side infiltration (Figure 4d). When surveyed after the third winter, several sealer sections were pulled out of the joints. Aside from some slight infiltration near spalls, they were in good condition and were preventing infiltration of all but fine silt, but debris was also being trapped above the sealer.

The third sealer, when sampled after the second winter, showed slight infiltration on the side of the 9.5-mm (⅜-in) joints, but the 6.3-mm (¼-in) joints were clean. The following fall, a core removed from an unsandblasted 6.3-mm (¼-in) joint revealed slight silt stains starting down one side of the sealer; lubricant adhesive was still adhering on the other side. During the third winter, one core was removed from each test section. In all four cases, only slight silt stains were found on the side of the joints, and most lubricant adhesive was still holding. No compression set was detected at any time. The spring survey following the third winter confirmed that, except for failures in spalled areas and slight silt infiltration along the joint sides, all of these sealers were in excellent condition. In most cases, the preformed sealers were installed lower in the joints than is normal for new construction in an attempt to get below the spalls for an effective seal, but success was limited because of the depth of most spalls (Figure 4e). In some cases, greater sealer depth added to the problem of accumulation of debris above the sealer.

DISCUSSION OF RESULTS

The four principal variables in this experiment were type of sealer, joint geometry, sealer depth, and method of joint preparation. Of these, type of sealer was shown to have the greatest effect on performance.

The standard asphalt sealer performed poorly in most joints and failed both by loss of adhesion and by entrapment of debris within the sealer. Addition of ground rubber to the asphalt sealer reduced its temperature sensitivity somewhat, but performance was little improved. The premixed rubber asphalts performed substantially better than the control asphalt and field-mixed rubber-asphalt sealers, especially in the 25.4-mm (1-in) expansion joints. The first were less temperature sensitive and quite resistant to embedment. The expansion joints resisted full-depth adhesion failures through three winters, and the contraction joints resisted them at least into the second winter. The PVC performed best of any of the liquid sealers; the sealer itself is in excellent condition after 3 winters. Although most joints that were not sandblasted have failed from full-depth adhesion failure, those that were sandblasted have exhibited very little of such failure. Because this sealer is not susceptible to temperature changes and resists embedment, most sandblasted joints are still in excellent condition. The cold-poured polyurethane performed better than the asphalt and rubber-asphalt sealers, particularly in the sandblased joints, but it did not perform as well as the PVC. This sealer also has little susceptibility to temperature and has remained intact over 3 winters. It, too, is resistant to embedment, but installation problems led to some isolated failures.

Except for failures in spalled areas, all three preformed sealers performed well over 3 winters. One brand is now beginning to display compression set. Although they have not prevented intrusion of all silt and water into the joint, they have been successful in preventing entrance of larger material. They are not suitable for joints with more than slight spalling.

Table 4 gives a summary of the properties of each sealer. Expected life, based on their performance in this test, is given for both expansion and contraction joints. For some sealers, sandblasting or other thorough joint cleaning would be required to achieve that life. The susceptibility of each sealer to temperature change and embedment and the principal failure modes that have been observed are given.

Thorough cleaning of joint faces by sandblasting had a pronounced effect on performance of some sealers, and very little on others. For all asphalt and rubber-asphalt sealers, sandblasted joints performed better in some cases, but not in others. In view of the failures caused by embedment and the extrusion problems mentioned earlier, determining whether sandblasting provided any real advantage is impossible. For the premixed rubber-asphalts, no advantage was obtained by sandblasting. However, for the PVC, in three out of four cases, the sandblasted joints performed better. In the fourth case, both the sandblasted and nonsandblasted test sections are still below the 10 percent limit. Three of the four nonsandblasted sections have failed, and the fourth has displayed some full-depth failure. Here, sandblasting appears to be essential for the sealer to perform as intended. For the polyurethane, sandblasting presented no advantage in the 25.4-mm (1-in) joints, but one extra year of service was achieved for the 12.7-mm (½-in) sealers in the sandblasted 6.3-mm (¼-in) joints. For sandblasted 9.5-mm (⅜-in) joints, this treatment appears to offer a significant advantage because they are still in service and the unsandblasted have failed. For preformed sealers, little difference is apparent to date except that lubricant adhesive seems to adhere better in sandblasted joints. Because few failures have occurred, no real difference can be expected at this time.

Both full-depth and 12.7-mm-deep (½-in-deep) liquid sealers were installed in 6.3 and 9.5-mm (¼ and ⅜-in) joints. In those not sandblasted, full-depth sealers generally performed better. In five of eight comparisons, such sealers performed better. In only two cases did they perform worse. The same is true for the sandblasted joints; in six of nine cases, full-depth materials have done better. In only two cases did restricted depths do better. In both of the latter cases, the joints failed early; therefore, no real advantage was gained. In several of the nine comparisons, both full- and restricted-depth sections are still in service, but, when the total amount of full-depth failure is examined, the advantage of full-depth sealing is apparent.

Joint width also has had a pronounced effect on performance. For liquid sealers, 6.3-mm (¼-in) contraction joints have performed considerably worse than the 9.5 or 25.4-mm (⅜ or 1-in) joints. For preformed sealers, both the 6.3 and 9.5-mm (¼ and ⅜-in) joints are generally sealed satisfactorily, but compression set in one sealer was somewhat more noticeable in the narrower joints.

CONCLUSIONS

From observations and measurements in this study, 14 conclusions appear to be warranted:

1. Removal of old sealers was difficult and slow, especially in narrow contraction joints, and, in some cases, resulted in spalled joint faces. Sandblasting cleaned them thoroughly after the old sealers were removed, but this process was very time consuming.
2. The asphalt and field-mixed rubber-asphalt sealers are the simplest of the hot-poured group to prepare and install although both require long heating to achieve pouring consistency.
3. The premixed rubber-asphalt and PVC sealers were more complicated to prepare and place than the asphalt and field-mixed rubber asphalts. Joint-sealing equipment was provided but did not function properly; installation would proceed rapidly with properly adjusted equipment.
4. The cold-poured polyurethane sealer required very little preparation time and no special equipment. Its very slow curing time makes installation difficult. Because it ran down through the bottoms of the joints and out the ends, most had to be refilled. Because of its slow curing, tracking by traffic was also a problem.
5. The preformed compression sealers were generally quite easy to install, but caution was necessary to avoid stretching them in the narrower joints.
6. The asphalt sealer currently specified by the New York State Department of Transportation provided the poorest service of all sealers tested; most test sections failed during the first or second winter from full-depth adhesion failure. It is very temperature susceptible, becoming hard and brittle in cold weather and soft and sticky in the summer. Those joints not failing from full-depth loss of adhesion had embedded foreign ma-

terial by the end of 2 years.

7. The field-mixed rubber-asphalt sealer performed slightly better in terms of full-depth adhesion failure, but only two test sections had less than 10 percent full-depth failure after 3 winters; the others failed from embedment. The most noticeable difference with this sealer is that it appeared to recover quickly in warm weather even though massive embedment had occurred over the winter. The additional cost and effort of adding ground rubber to the asphalt sealer does not appear to be justified by the minor improvements in performance.

8. The premixed rubber-asphalt sealers were less temperature sensitive and were not subject to massive embedment. Performance was similar to the asphalt and field-mixed rubber asphalt in contraction joints, in terms of full-depth failure. However, since little embedment occurred, the only foreign material to enter the joints was between the sealer and the joint face. The 25.4-mm (1-in) expansion joints are effectively sealed after 3 years. These sealers, therefore, appear satisfactory for wide joints and capable of sealing them through at least three winters.

9. The PVC sealer has little temperature sensitivity and successfully resists embedment of foreign materials. In sandblasted joints, it provided 3 winters of service with very low failure rates. Based on this performance, it appears suitable as a maintenance sealer in joints of the geometries tested here provided they are thoroughly cleaned before sealing.

10. The cold-poured polyurethane sealer is the least temperature sensitive of any of the liquids tested, and is not subject to embedment. This sealer provided good service in the sandblasted 9.5-mm ($^3/_8$-in) contraction joints but was similar in terms of full-depth adhesion failure to the asphalt and rubber-asphalt sealers in other joints. Installation problems also led to some isolated failures and, in spite of good performance in some test sections, made this sealer less suitable than others that provided comparable performance.

11. All three preformed compression sealers performed well over 3 winters although one has now begun to exhibit compression set. They have not been effective in spalled areas in spite of their being installed low to try to seal below the spalls. In such areas, dirt and incompressibles have been able to enter the joint. In spall-free joints, they appear capable of providing several years of good performance, but not in spalled joints.

12. For the PVC and cold-poured polyurethane sealers, sandblasting before sealing appears essential for good performance. For others it does not appear warranted.

13. Joints sealed to full depth generally performed better than those restricted to 12.7 mm ($^1/_2$ in), but some optimum depth between these extremes may provide satisfactory performance for some sealers.

14. An effective method of removing old sealers and cleaning joint faces is needed for an effective joint-resealing program.

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The contents of this paper reflect our opinions, findings, and conclusions and not necessarily those of the New York State Department of Transportation or the Federal Highway Administration.

Interested readers may note that an expanded version of this paper is in preparation; further information may be obtained by writing to James E. Bryden in care of the Engineering Research and Development Bureau, New York State Department of Transportation, State Campus, Albany, New York 12232, and inquiring for the first interim report on Research Project 105-1.

Abridgment

Evaluation of a Field Testing Device for Flashing Barricade Warning Lights

Robert C. Klotz, Pennsylvania Department of Transportation

Past studies have shown that up to 30 percent of the type A flashing lights in field use fail to meet Pennsylvania Department of Transportation specifications. A device, shown in Figure 1, was constructed to measure the effective intensity (candelas), flash time (milliseconds), and period (seconds) of flashing lights in the field. It weighs 14.6 kg (32.3 lb), its dimensions are 0.72 by 0.23 by 0.28 m (28.5 by 9.25 by 11.0 in), and it contains all optical, electronic, and power components. A front-mounted gimbal allows measurement angles of ±9 deg horizontal and ±5 deg vertical.

To obtain the three primary specification parameters, the electronic circuitry performs the following calculation in successive integrations, displaying a maximum effective intensity Ie on a meter.

$$Ie = \frac{\int_{t_1}^{t_2} I dt}{0.2 + (t_2 - t_1)} = \frac{\int_{t_1}^{t_2} I dt}{0.2 + \int_{t_1}^{t_2} dt} \quad (1)$$

where

I = instantaneous intensity,
t_1 and t_2 = flash duration limits, and
Ie = a maximum value.

A second meter shows the flash time of the light, and a third meter shows the period. Thus,

$$\text{Percentage of on time} = (0.1 \times \text{flash time})/\text{period} \quad (2)$$

$$\text{Flash rate (f/m)} = 60/\text{period} \quad (3)$$

Without the device, only the flash rate can be determined in the field.

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Evaluation of the device consisted of a direct correlation between values obtained for the aforementioned parameters by means of the device and the department's optical tunnel facilities. Variables under investigation were different light manufacturers, variable light power levels, and different viewing angles. Individual lights were tested initially, and results showed a one standard deviation (σ) for Ie as high as ±1.77 cd at 12-V power but only ±0.4 cd at 8 V. It was apparent that the device was more accurate when reading a light with a low intensity. The device was calibrated to read a light emitting 4.0 cd exactly (specification passing value). Thus it appeared that a light with an effective intensity of about 4.0 cd could be read within ±10 percent with a good degree of confidence.

Further work with additional lights showed definitely that optical centering of the light was important for good correlation. Following this procedure, correlation was within ±0.4 cd at 68 percent confidence with lights operated at or near the pass-fail region of 4.0 cd. Statistical evaluation of data also showed the device could determine the percentage of on time of a type A light to within ±5 percent of the value obtained at 68 percent confidence by using standard lab procedures.

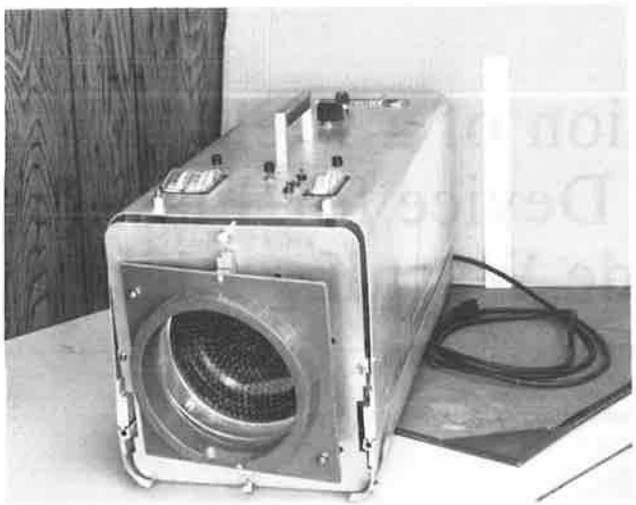
CONCLUSIONS

The device is too heavy and bulky for easy field use. The gimbal used to set viewing angles is not accurate enough or repeatable. The battery power system limits operation to about 1 h after full charge.

After careful optical alignment to a type A light powered with a voltage to cause a failing light (< 4.0 cd at ±9 deg horizontal, ±5 deg vertical), the test device will read within ±10 percent (1σ = ±0.4 cd) of the value currently obtainable with the optical tunnel instruments. Of the ±0.4-cd deviation, about one-third or ±0.12 cd can be statistically attributed to the use of regression analysis techniques; the remaining two-thirds (±0.28 cd) is due to measurement error, reading error, or other experimental errors.

The apparent success of the device in determining quickly the percentage of on time and F-factor (a value currently used in the standard optical tunnel test tech-

Figure 1. Measuring device.



nique) may justify its use as a laboratory testing instrument.

Serious consideration should be given to developing a second-generation device, perhaps of two-piece design and having a lightweight optical head with accurate angular settings obtainable. An electrical cord could connect the head to a digital readout console containing computer and power elements.

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Measurement and Prediction of Preferential Icing Potential of a Bridge Deck

C. Birnie, Jr., Pennsylvania State University, University Park

Preferential icing, the tendency of a bridge deck to freeze before the adjacent roadway, was studied by the Pennsylvania Transportation Institute, Pennsylvania State University, in cooperation with the Pennsylvania Department of Transportation from 1969 through 1973. A bridge deck on I-80 in Pennsylvania was instrumented, and complete data on the meteorological conditions and surface temperatures of both the bridge deck and the adjacent roadway were gathered during the winters of 1971-72 and 1972-73. This paper summarizes these experimental findings and suggests a method for determining an overall rating of the preferential icing potential of a bridge deck. A computer model was developed to determine the thermal response of both a bridge deck and adjacent roadway to ambient conditions. A method is described that, if one uses this model, can predict the possible formation of preferential icing on a bridge deck before it occurs and statistically determine an overall rating of the preferential icing potential of a bridge. A simple instrument package for surveying bridges for preferential icing is described.

Preferential icing, the tendency of a bridge deck to freeze before the adjacent roadway, was the subject of an investigation conducted by the Pennsylvania Traffic Institute, Pennsylvania State University. The project was sponsored by the Pennsylvania Department of Transportation and the Federal Highway Administration. It was initially funded in January of 1969 and was concluded in August 1973.

The project consisted of three major phases. Although each phase had particular goals, the overall objectives were to study the various factors that lead to the preferential icing of a bridge deck, to measure the phenomena at a selected test site, and to determine methods of predicting its occurrence. Summaries of the work done in the first two phases of the project are contained elsewhere (1, 2, 3).

In the final phase of the study, experimental data were gathered at the test site during the winters of 1971-72 and 1972-73 and were subjected to extensive analysis. A good portion of this paper will be devoted to reporting on these experimental data and on the results. A method for determining an overall rating of the preferential icing potential of a bridge deck based on measured data will be suggested.

In addition to the experimental portion of the study, considerable time was devoted during all phases to the development of a transient, heat-transfer simulation model of both the bridge deck and the adjacent roadway. The final version of the model was completed during the last phase and will be described. A suggested method for using the model to predict the possible formation of preferential icing on a bridge deck before it occurs and to statistically determine an overall rating of the preferential icing potential of bridges will be outlined in detail.

TEST SITE DESCRIPTION

The bridge on the eastbound lane of I-80 over Bald Eagle Creek in Centre County, Pennsylvania, was selected as the test site. The site is located some 25.7 km (16 miles) from State College, Pennsylvania, near the US-220 interchange of I-80. It is on the floor of Bald Eagle Valley just east of the foot of the Allegheny Mountains. The creek that flows under the bridge is relatively warm because of the presence of a steam power plant that discharges cooling water into it about 3.2 km (2 miles) upstream. The warm humid air rising from the creek in the winter thus provides additional moisture for condensation on the bridge surface. The bridge selected is relatively new; it was built in 1964. It is an all-concrete structure approximately 61 m (200 ft) long and 6.1 m (20 ft) high. The understructure is box-beam, poured-in-place, stripped-form concrete.

Instrumentation of the site was based on the variables and conditions that were deemed important enough to have the greatest influence on preferential icing, that is, meteorological conditions at the site and the thermal characteristics of the bridge deck and of the approachway. A plan view of the general location of the instrumentation is shown in Figure 1. The measured variables are

1. Temperatures,
2. Humidity,
3. Wind velocity and direction,
4. Precipitation,
5. Ice detection,

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6. Solar radiation, and
7. Event recording.

The thermal gradient within the bridge deck and approachway and some subgrade temperatures were measured by using thermocouple probes. The probes were made from 0.05-cm (20-AWG) thermocouple grade copper-Constantan (type T) extension wire with polyvinyl chloride (PVC) covering over PVC insulation. The temperatures changed slowly enough that response was not a problem. To measure the thermal gradients, two 10-cm-diameter (4-in-diameter) class AA concrete cores were poured; each had 10 thermocouples located axially through a 19-cm (7.5-in) depth. One core was grouted into the roadway, and the other was grouted into the bridge deck at the midpoint between two beams. The cores were grouted in to form an integral part of the surfaces of the bridge deck and approachway. Air temperatures were monitored at the instrument shack, and at various points over and under the bridge. The output of all thermocouples was recorded on a 24-channel balance potentiometer. The potentiometer was connected to a timer that actuated it every hour for a period of approximately 6 min, which was long enough to record all temperatures once. Experience had shown that the values changed slowly enough that readings taken at 1-h intervals were satisfactory. There was enough chart to operate without servicing for 30 days.

Relative humidity and air temperatures over the bridge were measured on a hygrothermograph that was mounted at deck height outside the bridge parapet at midstream (Figure 1). The instrument was mounted so that it was exposed to the humidity flux off the stream. Values were recorded continuously on a 60-day folding strip chart.

Wind velocity and direction were measured 4.8 m (16 ft) above the bridge deck by a sky vane. Output from the sky vane was recorded continuously on a strip chart to measure gross wind speed and direction over the bridge.

The quantity and rate of all types of precipitation were measured by a heated precipitation gauge in 0.25-mm (0.01-in) increments. The gauge was located on the valley floor adjacent to the stream (Figure 1) and was mounted on a steel platform sufficiently high to protect it from flooding. Output from the instrument was recorded continuously on a 7-day drum chart.

The presence of surface moisture on the bridge deck was detected by using an ice detection system. The basic element of this system consists of two moisture sensors located in the bridge deck. In principle, these sensors are capable of determining whether the bridge deck is dry, wet, or frozen. The output signal from the system was recorded continuously on an event recorder.

During the work in the first winter (1969-70), it became evident that the diurnal variation of solar radiation had a major influence on the bridge and roadway surface temperatures. Therefore, a recording solar meter was added to the instrumentation at the test site beginning with the 1970-71 winter. The solar meter was installed on the roof of the instrumentation shack adjacent to the site (Figure 1). It recorded the instantaneous and total solar radiation. Data were collected once a week from a drum chart.

A 10-channel event recorder was used as an overall monitoring device on most of the other instrumentation. On it, periods of precipitation, output of the ice detection system, and periods during which the thermocouple output recorder was on were recorded. This gave a single time on which this information was recorded.

All the recorders for the various probes (with the exception of the hygrothermograph) were housed in a 1.5 by 2.4-m (5 by 8-ft) metal shack located on the berm at the western end of the approach to the bridge (Figure 1). The shack was insulated with styrofoam and was electrically heated. The recorders located in the shack measured temperature, wind, rain, and events.

TEST SITE DATA

A small amount of data was gathered from January 1970 to June 1970 primarily to test out the instrumentation. During the winter of 1970-71, complete temperature profiles for both the bridge deck and the roadway were taken. The bulk of the data taken and discussed here, however, was recorded during the winters of 1971-72 and 1972-73. During this period, the principal variables measured were bridge surface temperature, adjacent road surface temperature, ambient air temperature, hourly precipitation, and solar radiation. Data were recorded on an hourly basis. Because the amount of data was extensive, they were transcribed into an IBM card file. This final file consists of two sections. Section 1 is of 85 selected days of the 1971-72 season and represents those times when preferential freezing [bridge deck below 0°C (32°F) when adjoining roadway was not] most likely occurred. The second section consists of all the daily data taken in the 1972-73 season, a total of 165 days. Table 1 gives a summary of the days when data were taken. In Table 2, a similar summary of the days on which solar radiation data were taken is given.

Precipitation measurements were taken by using the recording rain gauge, and periods when the bridge surface was wet or icy were recorded by using the Econolite ice detection system. These were also incorporated into the data file on an event card.

The data file was used in a number of types of analyses. All these were carried out by using an IBM 370 computer at Pennsylvania State University. The format used on the data cards is such that they can be run on any large digital computer.

Initially, all of the data collected were processed and presented in the form of a series of daily temperature graphs. The principal graph is a plot of ambient air, bridge surface, and road surface temperatures versus time, starting at 1 a.m. and going to midnight of the date. A second plot on the same graph shows the solar radiation (1 kJ/m^2 = 0.024 langley). The daily graphs for the winters of 1971-72 and 1972-73, 250 in all, are available elsewhere (4).

Because periods of preferential icing require that the roadway surface temperature be above the bridge surface, this temperature difference (road-bridge) was determined at each hourly interval. An analysis of the data showed that, most of the time, road surface temperature was above that of the bridge. Figure 2 shows the three conditions under which preferential icing can occur. In this plot, the road and bridge surface temperatures are shown to follow closely parallel paths. This type of variation was frequently observed at the test site. Zones of preferential icing can occur during periods of decreasing temperature (I) and rising temperature (II) and under conditions when the temperatures are steady but around 0°C (32°F). In the case of rising and falling temperatures, the magnitudes of I and II will increase when the road-bridge temperature difference increases. For III, the greater the difference is, the more likely it is that preferential freezing will occur.

SIMULATION OF THERMAL RESPONSE OF BRIDGE DECK AND ADJACENT ROADWAY

Ideally, one would like to have a mathematical model that would, given certain weather conditions, predict the potential of accidents occurring due to preferential icing on a given bridge. The output of such a model could be a statistical probability that a given number of icing accidents per year would occur. The basic approach to the problem is to divide it into two categories: (a) response of the bridge to local weather, which will determine the probability that preferential freezing occurs, and (b) given that ice is present on the bridge, the probability that a vehicle will skid and cause an accident. In this study, effort was directed toward determining the icing probability of a given bridge based solely on data gathered on the thermal characteristic of the bridge structure and adjacent roadway and the local weather most likely to occur. For this purpose a simulation model, SIMULATION III, was developed (5).

SIMULATION III is actually two separate digital computer programs that solve the heat transfer problem existing, first, between the bridge deck and the ambient weather conditions and, second, between adjacent roadway and ambient weather conditions. The bridge deck has been treated as a plane slab of uniform thermal properties. The adjacent roadway is also treated as a slab but is assumed to consist of three layers of different material. These are the concrete roadway proper, a subbase assumed to consist largely of gravel, and a subgrade of earth. The depths of both the bridge and the layers of the roadway, along with the thermal characteristics of each layer, can be varied. This is supplied as input data. Figure 3 shows the geometry and thermal characteristics required. In each case, the heat transfer is assumed to be one-dimensional and perpendicular to the surface. In the case of the bridge deck, ambient weather boundary conditions are applied to both the upper and lower surfaces, although they need not be the same on both. For the roadway, the temperature at the bottom of the subgrade is held constant at a value considered typical of that expected at that depth.

The output of the simulation can be complete temperature profiles for both bridge and roadway at fixed-time intervals. The program can be run for any overall desired time periods. Runs have been made covering periods from 1 day to 1 month. It is anticipated that, in rating bridges, a period corresponding to the winter season (October to March) would be used.

The accuracy of the surface temperatures generated by the model is dependent on the accuracy of the weather boundary conditions applied. The boundary conditions used are time dependent and take into account the three major factors that influence the surface heat transfer. These are ambient air temperature, wind speed, and solar radiation. The values of air temperature, solar radiation, and wind speed supplied must represent those expected to be encountered over a typical winter at the bridge site.

If the thermal response of a bridge over a typical winter season is to be determined by simulation, good statistical models for the ambient air temperature, wind velocity, solar radiation, and precipitation are necessary as boundary conditions. Weather bureau records for the primary weather stations in Pennsylvania were used for this. This represents the most comprehensive collection of data available and was considered to be an adequate basis for the formulation of the required models. Data were obtained from the National Climatic Center on magnetic tape. The data covered 1961 through 1972 for the following stations: Allentown, Philadelphia, Harrisburg, Wilkes-Barre, Williamsport, Erie, and Pittsburgh. Originally, these data were on a number of different tapes that used several formats. Therefore, the data had to be processed. The processing procedure allowed the information to be compacted and put in a uniform format.

As a result, two magnetic tapes containing all the weather information were produced. The first tape, entitled WEATH 1, is a single file containing 325 480 logical records; each record is 104 characters long and represents one set of observations. On this tape, the observations are grouped by stations in ascending, chronological order. The station order is the same as the listing previously given. On the second tape, entitled WEATH F, the data are arranged into seven separate files, one for each city. This is more convenient to use when only the observations of a single station are required. This work is contained in a separate publication (6).

The influence of solar radiation is taken into account by assuming that it varies sinusoidally from sunrise to sunset. Since the period from sunrise to sunset varies in terms of the latitude of the bridge and time of year, this was taken into account. Obviously, the actual solar radiation depends on the amount of existing cloud cover. Although the final version has not been done, it is anticipated that a cloud cover model similar to the temperature model can be developed by regression analysis.

Accurately establishing the effect of wind on the surface heat transfer involves two problems. First, a model giving at least wind magnitude similar to the temperature model is needed. Second, if wind magnitude is given, the actual value of the surface film coefficient of heat transfer has to be determined. Efforts have been put into the latter and reported (7). So far, no attempt has been made to develop a statistical wind magnitude model because the wind effect was not considered to be as important as temperature and solar radiation. Additional work is required in this area.

Finally, it should be noted that the ambient temperature, solar radiation, wind, and precipitation have been treated as essentially independent quantities. This certainly is an area that requires further investigation. As proposed, the combination of models may lead occasionally to the creation of improbable weather conditions such as days of high solar radiation with precipitation or low ambient air temperatures.

METHODS OF MEASURING PREFERENTIAL ICING POTENTIAL OF A BRIDGE

Even when the response of a bridge deck and adjacent roadway to given weather conditions as well as what weather conditions most commonly occur at the bridge can be accurately determined, the question of how to best present the data remains. There are always two factors to be considered: the frequency at which the event occurs without regard to its duration and the actual duration of the event. Often the periods of icing are quite short (15 min or less), and these may be periods when no traffic occurs. The problem is which of these—frequency or duration—is more significant. Because an overall rating is desired, the accumulative hours per year of preferential icing are proposed to be used as a method of rating the response of a bridge to weather. This is not a probability, but it does seem to be a logical parameter for comparing different bridges, perhaps within a state or a region of a state, to determine the worst performers.

To determine the accumulative hours per year of preferential icing requires not only knowledge of when the bridge is below and when the roadway is above 0°C (32°F) but also information on the periods and type of precipitation that falls. Obviously, these data can be obtained on a short-term basis if sufficient instrumentation is installed on a given bridge. However, this is not always practical. A less sophisticated measure of the potential of a bridge to ice would be to consider only surface temperatures and to determine periods of preferential freezing. Preferential freezing will be defined as those periods when the bridge is below 0°C (32°F) but the roadway is not, without regard for precipitation. Again this is a useful figure for comparisons.

Data on preferential icing and freezing can be arrived at in a number of ways: (a) direct measurement of temperature and weather at the site, (b) generation of data by simulation techniques, and (c) a combination of both simulation and direct measurement. All of these techniques have been explored in various degrees, and items

Figure 1. Location of instrumentation at bridge test site on I-80.

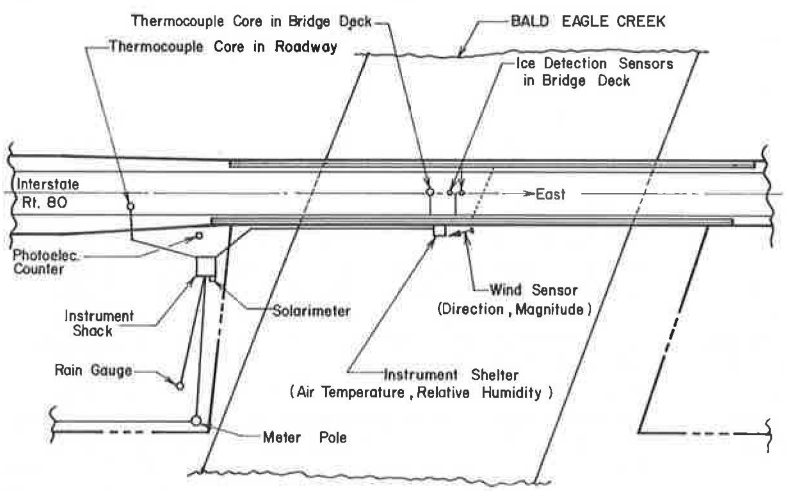


Figure 2. Zones of preferential icing.

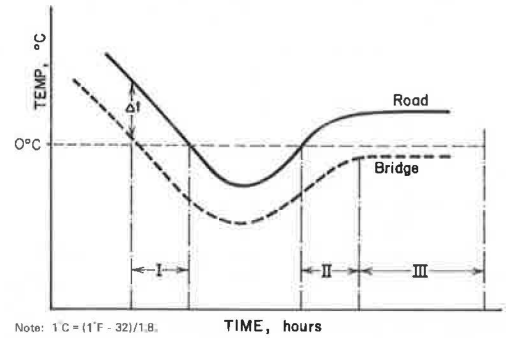


Table 1. When data from I-80 test site were taken.

Season	Month and Year	Date	Number of Days
1971-72	November 1971	9, 12, 24, 25	4
	December 1971	11 to 15, 17 to 23, 29 to 31	15
	January 1972	1 to 24	24
	February 1972	1, 7, 8, 14 to 23, 28, 29	15
	March 1972	1 to 27	27
	Total		85
1972-73	October 1972	6 to 31	26
	November 1972	1 to 30	30
	December 1972	1 to 31	31
	January 1973	1 to 31	31
	February 1973	1 to 28	28
	March 1973	1, 6 to 20, 27 to 29	19
	Total		165

Table 2. When solar radiation data from I-80 test site were taken.

Season	Month and Year	Date	Number of Days
1971-72	November 1971	9, 12, 24, 25	4
	December 1971	11 to 15, 17 to 23	12
	January 1972	5 to 8	4
	February 1972	4, 7, 8, 15 to 23, 29	13
	March 1972	1 to 27	27
	Total		60
1972-73	October 1972	25 to 31	7
	November 1972	1, 3 to 8, 10 to 14, 16 to 26, 29, 30	25
	December 1972	1 to 3	3
	January 1973	12 to 16, 18 to 28	16
	February 1973	6 to 25, 27, 28	22
	March 1973	1, 7 to 11	6
	Total		79

Table 3. Monthly frequency and accumulative hours of preferential freezing for winter of 1972-73.

Month and Year	Days	Frequency	Duration (h)	Mean (h/event)
October 1972	26	5	17	3.4
November 1972	30	22	70	3.18
December 1972	31	21	56	2.67
January 1973	31	28	66	2.36
February 1973	28	25	53	2.12
March 1973	19	5	39	7.80
Total	165		301	

Figure 3. Geometric and thermal characteristic input data to SIMULATION III.

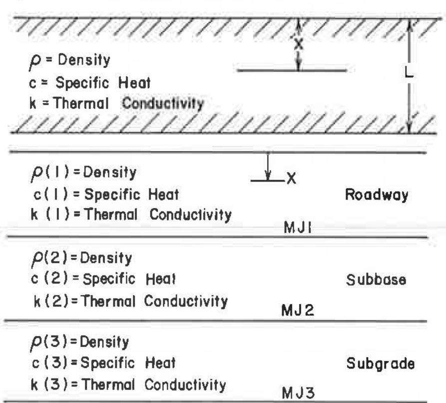


Table 4. Frequency and accumulative hours of preferential freezing at different times of day for winter of 1972-73.

Time Period	Frequency	Duration (h)	Mean (h/event)
Midnight to 4 a.m.	12	34	2.83
4 a.m. to 8 a.m.	15	45	3.0
8 a.m. to noon	33	74	2.42
Noon to 4 p.m.	16	41	2.56
4 p.m. to 8 p.m.	17	46	2.70
8 p.m. to midnight	13	61	4.69

a and b will be reported on. Each has its advantage and disadvantage.

Direct Measurement

A computer program was developed to analyze the data from the test bridge for periods of preferential freezing and icing. In Table 3, a monthly summary is given of the frequency and accumulative hours of preferential freezing for 1972-73 as an example of the program output. In the table, the mean represents the total accumulative hours divided by the frequency of preferential freezing.

Information about the time of day that preferential freezing most often occurs can also be obtained. In Table 4 such an analysis is given.

Tables 5 and 6 give preferential icing data, which were determined by comparing the periods of preferential freezing with the precipitation data. Preferential icing is a more reliable rating of a bridge, but it requires continuous measurement of precipitation.

Direct measurement of periods of preferential icing and freezing requires, at a minimum, the measurement on a common time base of three variables: bridge surface temperature, adjacent road surface temperature, and presence of precipitation. From this information, periods of preferential freezing can be measured. Periods of preferential icing can be reasonably deduced as occurring on the bridge when precipitation falls during periods of preferential freezing. The reliability of the method for determining preferential icing is limited by the instrumentation used to detect small amounts of precipitation. Also the presence of frost is very difficult to measure. In some cases, this could be a serious problem. (Preferential frost formation was never encountered at the test site.) The method also has the disadvantage of requiring the direct installation of instrumentation at the bridge and accompanying costs. Furthermore, even if data are gathered for an entire winter season at a bridge, it would be difficult to say that this was typical of its performance over an extended period. However, the simultaneous gathering and comparison of such data at a group of adjacent bridges in a given locale could possibly serve to initially identify the worst in the group.

Simulation

Data such as those taken at the test site and analyzed to obtain Tables 3 through 6 could also be obtained by determining the thermal response of a bridge deck by simulation. By knowing the thermal characteristics of a given bridge and subjecting the model to boundary conditions that represent a statistically typical winter at the bridge site, one can generate a data file similar to that obtained at the test site. Such an overall modeling procedure is shown in Figure 4.

Simulation of the periods of preferential freezing and icing has the advantage of requiring no installation of instrumentation at the bridge site. Data on the geometrical and thermal characteristics of the bridge can be obtained from the bridge specifications and perhaps one or two on-site inspections for purely local characteristics. Presumably, with the required input specifications, a large number of bridges in a given geographical region could be simulated and rated in terms of accumulative hours of preferential freezing and icing.

METHODS OF PREDICTING PREFERENTIAL ICING

The simulation model provides a possible method of forecasting preferential icing before it occurs. If, instead of a statistical weather model as input to the simulation, the actual weather forecasted for the bridge site were employed, the model should predict future bridge and road surface temperatures. In brief, throughout a winter season, the simulation model would be run essentially on a continuous basis, always receiving as input forecasted weather data and evaluating the response of the bridge to these data to predict road and bridge surface temperatures. By continually analyzing the output of the model, one could predict future periods of preferential freezing. If precipitation was forecasted for the preferential freezing periods, suitable countermeasures could be taken.

Although the method requires the use of a computer, it does have a number of advantages. First, it predicts bridge and road surface temperatures without any actual instrumentation at the bridge itself. Second, the time required for the computer to solve the response problem is very small in terms of the time scale associated with weather changes. The simulation solution could therefore be continually updated on the basis of newly received weather forecasts. Third, although the simulation as now proposed solves the heat transfer equations by numerical integration, a much simpler analog computer solution is possible. By using an analog computer, one could develop a very small unit for solving the simulation problem at reasonable cost. Last, the potential for using a single large computer to handle a number of bridges in a given locale is possible. In Figure 5, a flow diagram outlining this method is shown.

INSTRUMENT PACKAGE FOR MEASURING PREFERENTIAL FREEZING (ICING)

Observations at the test site plus the work on the simulation model indicated a clear need for some method for directly measuring periods of preferential freezing and icing on a bridge deck. This requires a continuous time record of a minimum of three variables: bridge surface temperature, adjacent road temperature, and presence of precipitation. Accumulative periods of preferential freezing and, with reasonable reliability, periods of icing can then be determined. Such an instrument to be practical should have the following characteristics: moderate cost, portability, ability to run unattended for at least a week, and ease of installation. Because no such instrument existed, a prototype was developed and subjected to limited tests as part of the project.

Figure 6a shows a schematic of the system developed. The principal element in the system is a portable strip recorder with a regulated motor drive. The recorder carries enough paper for 30 days of operation and requires a 12-Vdc source. Three channels are on the recorder; two are for temperature and one is used for event recording (in this case, for precipitation). The temperature probes are thermistors located at the end of 30.5-m (100-ft) cables. To detect precipitation, a heated moisture sensor is employed. This consists of a unit that can be pole mounted, either on the bridge or nearby, and that measures precipitation electrically. It consists of two wire grids spaced horizontally and exposed to the sky. The upper grid is heated and controlled by a thermostat that comes on at about 4.5°C (40°F). When snow or sleet falls, it melts on the upper, heated grid. When it comes in contact with the lower grid, an electrical signal is sent to the event channel of the recorder. Power for the heater grid is 12 Vdc.

The output of the unit as finally developed consists of a continuous strip chart of 30-day duration. A section of the chart produced is depicted in Figure 6b. Periods of preferential freezing (PF) and preferential icing

Table 5. Monthly frequency and accumulative hours of preferential icing for winter of 1972-73.

Month and Year	Days	Frequency	Duration (h)	Mean (h/event)
October 1972	26	5	15	3.00
November 1972	30	5	7	1.40
December 1972	31	6	13	2.17
January 1973	31	3	3	1.00
February 1973	28	2	4	2.00
March 1973	19	0	0	—
Total	165		42	

Table 6. Frequency and accumulative hours of preferential icing at different times of day for winter of 1972-73.

Time Period	Frequency	Duration (h)	Mean (h/event)
Midnight to 4 a.m.	2	2	1.0
4 a.m. to 8 a.m.	3	4	1.33
8 a.m. to noon	5	8	1.6
Noon to 4 p.m.	4	6	1.5
4 p.m. to 8 p.m.	5	11	2.2
8 p.m. to midnight	2	11	5.5

Figure 4. Simulation program for generating periods of preferential freezing and icing.

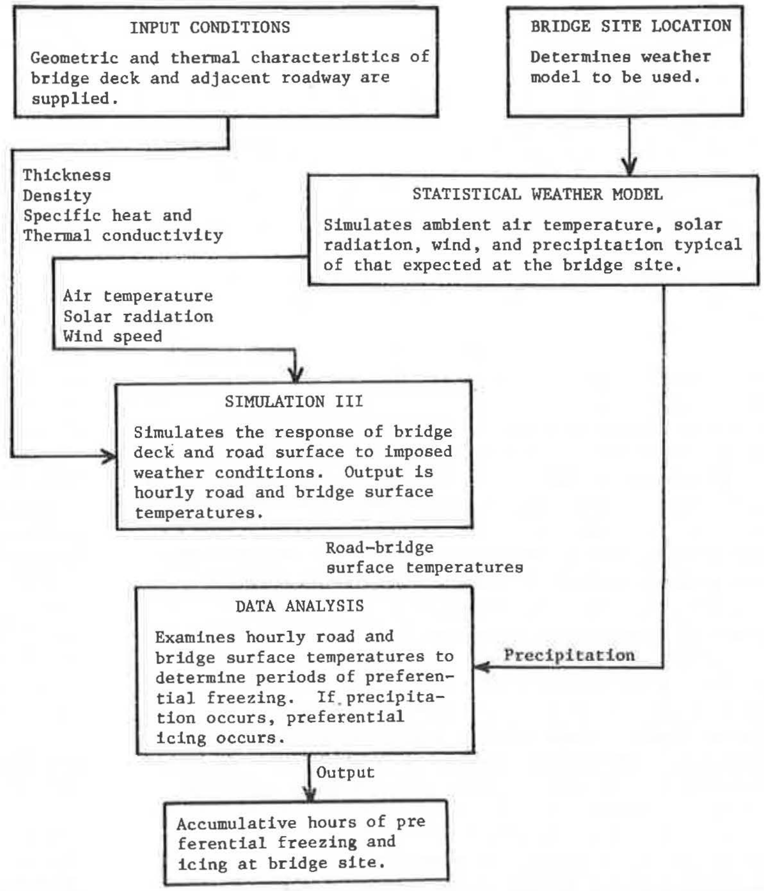


Figure 5. Use of simulation to predict periods of preferential icing.

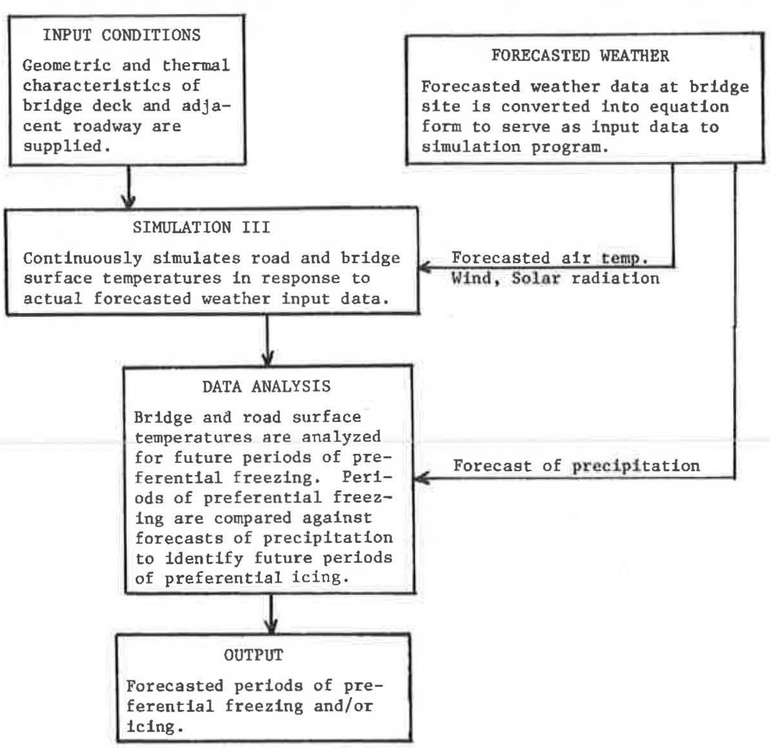
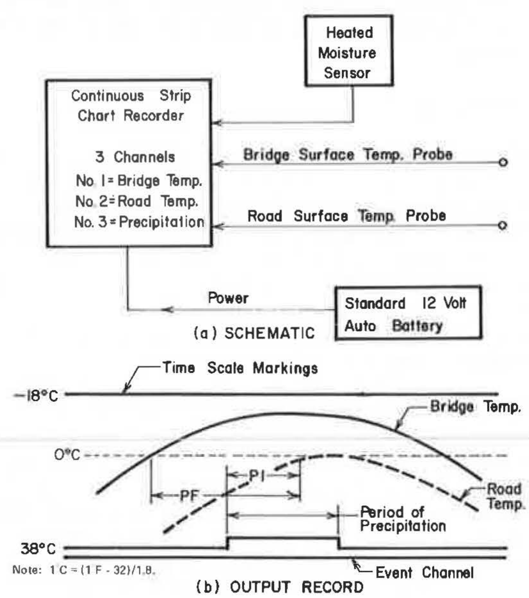


Figure 6. Instrument for measuring preferential freezing (icing).



(PI) are indicated. The completed instrument package consisted of three units: the recorder in a metal, waterproof case; the pole-mounted moisture sensor; and the 12-V battery pack. The units are of moderate weight; the battery (standard automotive) is by far the heaviest item. The cost of the unit and temperature sensors was \$553. Additional costs were incurred in the purchase of the moisture sensor, battery, and necessary modifications. It is estimated that additional units could be built for approximately \$1200.

SUMMARY OF RESULTS

Data File and Data Analysis

Data were collected at the test site for the winters of 1971-72 and 1972-73. These data were tabulated on IBM cards to form a permanent data file. The compilation of a data file makes possible convenient computer analysis of this information in a variety of ways. The data were analyzed to determine accumulative periods of preferential freezing and icing principally to illustrate the possible use of these parameters as a bridge rating factor.

Simulation Model

SIMULATION III, a digital computer program for determining the bridge and road surface temperatures in response to various input weather boundary conditions, was completed. A method for rating bridge decks for preferential freezing (icing) was proposed that employed the simulation model. This method consisted of applying as input boundary conditions to the simulation model (a) the geometrical and thermal characteristics of the bridge to be rated, and (b) a statistical weather pattern that represents a typical winter at the bridge site. The model will then determine the accumulative hours of preferential freezing (icing), which have been suggested as a bridge rating factor. A statistical winter temperature model for Pennsylvania was developed for use with the simulation program. Methods of developing models for wind and solar radiation were also discussed.

The potential use of the simulation model that uses actual forecast weather as a boundary condition for predicting periods of preferential freezing (icing) was discussed.

Instrument Package

A simple, inexpensive instrument package that can be installed on a bridge to measure periods of preferential freezing (icing) was developed. A prototype was tried in a field test and was found to substantially meet all the original objectives set for such an instrument.

ACKNOWLEDGMENT

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The opinions, findings, and conclusions expressed in this publication are mine and are not necessarily those of the Pennsylvania Department of Transportation or the Federal Highway Administration.

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Laboratory Comparison of Calcium Chloride and Rock Salt as Ice Removal Agents

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For ecological as well as economic optimization of highway deicing operations, one must know the relative efficiencies of the two agents most commonly used—calcium chloride and rock salt. Because data on this subject are in conflict, new measurements have been made of the extent to which anhydrous calcium chloride pellets and typical Michigan rock salt undercut a sheet of 0.3175-cm-thick (1/8-in-thick) ice bonded to a concrete block. The extent to which undercutting (the breaking of the bond between ice and concrete) occurred as a function of time and temperature was followed by adding a dye to the chemical, which caused the brine formed by melting ice to fluoresce under ultraviolet light. Commercial anhydrous calcium chloride pellets have a relatively narrow particle size distribution and are very uniform in action; reproducible data could be obtained. However, the action of rock salt was found to be strongly affected by wide variability in size and purity of individual particles. After the action of two particle sizes of pure fused sodium chloride was measured, a reasonable estimate could be made of the average action of a typical commercial Michigan rock salt. The results are presented as a table of the quantity of chemical per unit area required to completely undercut the ice sheet as a function of time and temperature. Rock salt is about equal to calcium chloride above -3.89°C (25°F) for 1 h. As temperatures are lowered and times are shortened, increasingly larger relative quantities of rock salt are required. This must be considered an ecological concern.

A recent comprehensive review of the state of the art of deicing highways by chemicals reported that "calcium chloride, sodium chloride, and their mixtures offer, within certain limitations, the best combinations of useful physical properties and reasonable cost" (1). Sodium chloride (as rock salt) finds the largest use because of its low cost; calcium chloride ($CaCl_2$) is employed for low temperature conditions where its greater effectiveness justifies its higher price. A recent concern is the relative ecological impact of use of the two chlorides (2). Quantitative comparison of the ice-melting properties of the two chlorides is essential for the overall optimization of deicing operations.

Although calcium chloride is generally rocognized to be more effective than rock salt as a deicer at low temperatures, the exact extent of this advantage is not clear from previous laboratory studies. For example, at -6.67°C (20°F) 30 min after application, Toddie (3) found that anhydrous calcium chloride pellets melted about four times as much ice as rock salt. Under the same conditions, Brohm and Edwards (4) found that calcium chloride melted only 1.5 times as much ice as did rock salt. Dickinson (5) reported that calcium chloride melted only 20 percent more ice than rock salt did. The difference therefore ranges from severalfold to insignificant. These previous studies were carried out by spreading the chemical on a tray full of ice, allowing an interval of time to elapse, and pouring off and weighing the brine produced or determining the loss in weight of the ice. This procedure yields only a single data point per experiment and appears to be difficult to carry out accurately.

ICE REMOVAL STUDIES

New Technique

In this study, a laboratory method for continuous observation of ice-melting action in a situation simulating an ice-covered concrete highway was used. A particle of chemical when dropped on the ice will melt down through the ice, and the brine will then spread out underneath the ice as shown in Figure 1. This bond-breaking action is the most significant effect of deicing chemicals because calculations show that it would be prohibitively expensive to melt all the ice. In these experiments, a 0.3175-cm (1/8-in) layer of clear ice was built up on a concrete block and the block was placed in a commercial deep freeze unit held at the desired temperature. A small fan in the freezer circulated the cold air to eliminate temperature gradients. Weighed particles of chemical were lightly dusted with a dye, the sodium salt of fluorescein, and loaded into a dispenser in which the particles were isolated in individual compartments. The dispenser was placed over the ice-covered concrete block, and the system was allowed to come to a uniform temperature as measured by thermocouples or thermometers. The dispenser was then actuated by pulling cords passing through the freezer lid. The particles were dropped on the ice, and the dispenser was drawn

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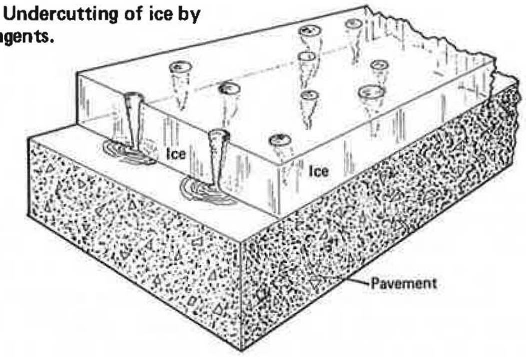
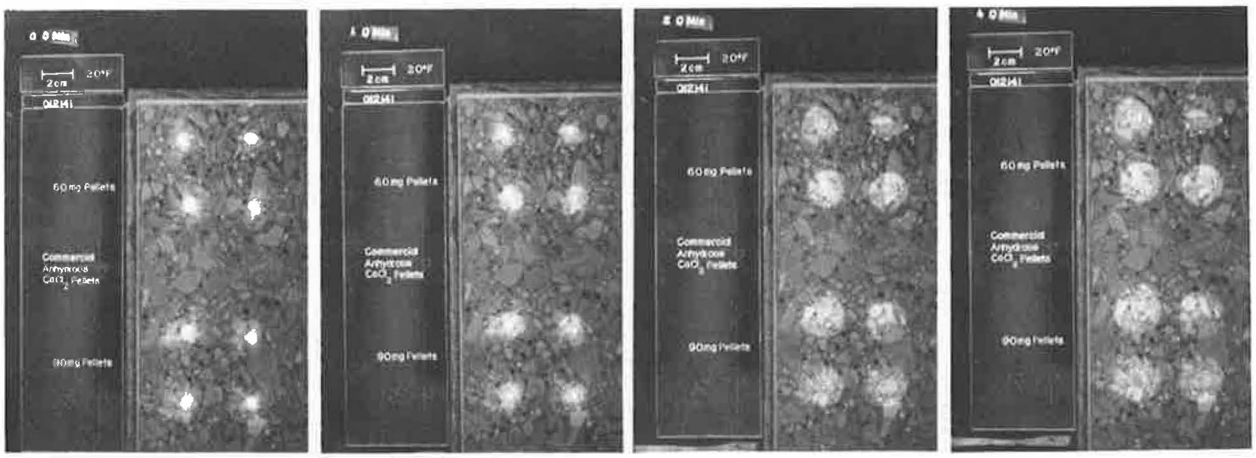
Figure 1. Undercutting of ice by chemical agents.



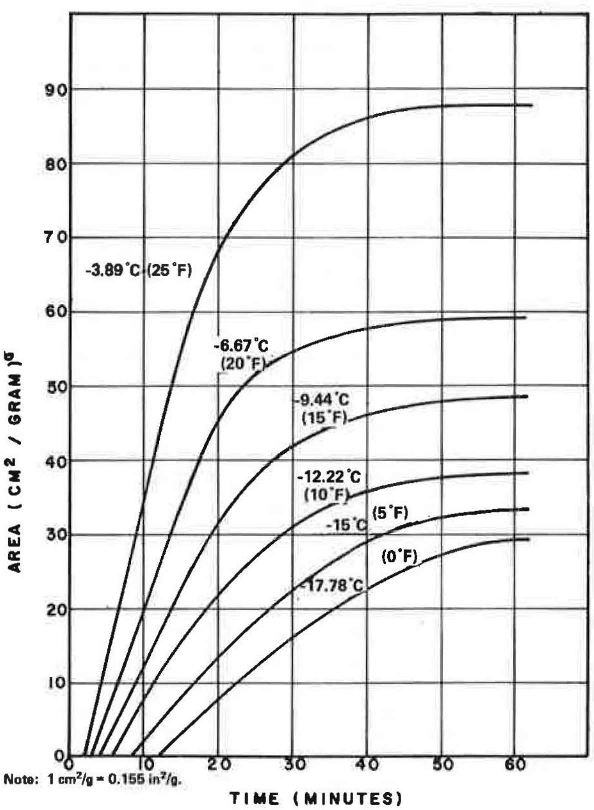
out of the way. As melting action began, the dye dissolved and fluoresced because of an interior ultraviolet light source. The fluorescent brine clearly outlined the extent to which the bond between the ice and the concrete was dissolved. Color photographs of the melting action were taken through a plastic window in the freezer lid at appropriate intervals. Selected photographs from an experiment with anhydrous calcium chloride pellets at -6.67°C (20°F) are shown in Figure 2. Unfortunately, the distinctiveness in melting and undercutting action evidenced is not clearly shown in black and white photographs. The initial hole melted through the ice becomes surrounded by a lighter colored area, which is the thin layer of brine between the ice and concrete. The area

Figure 2. Fluorescent dye technique for measuring undercutting of ice by chemical agents.



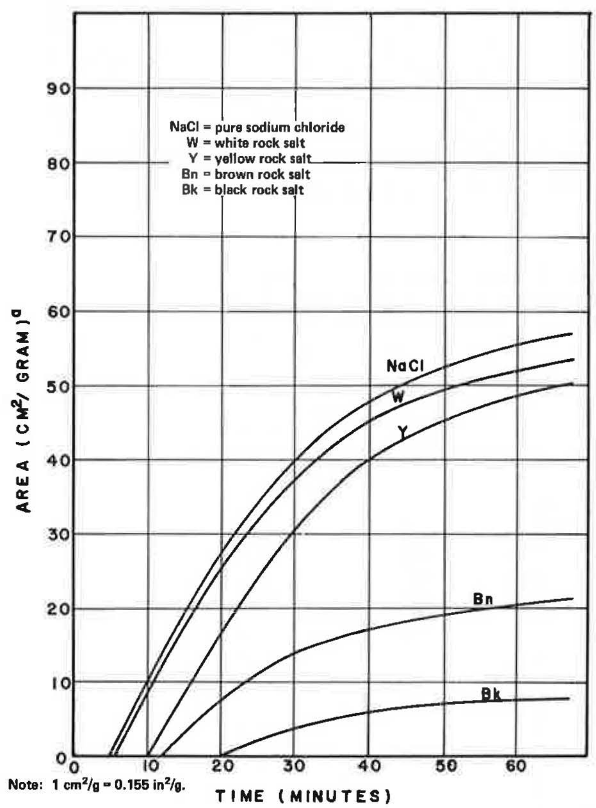
Note: 1 cm = 0.394 in. 1°C = (1°F - 32)/1.8.

Figure 3. Area of ice undercut by anhydrous $CaCl_2$ pellets as a function of time and temperature.



Note: 1 cm²/g = 0.155 in²/g.

Figure 4. Area of ice undercut by 90-mg particles of sodium chloride.



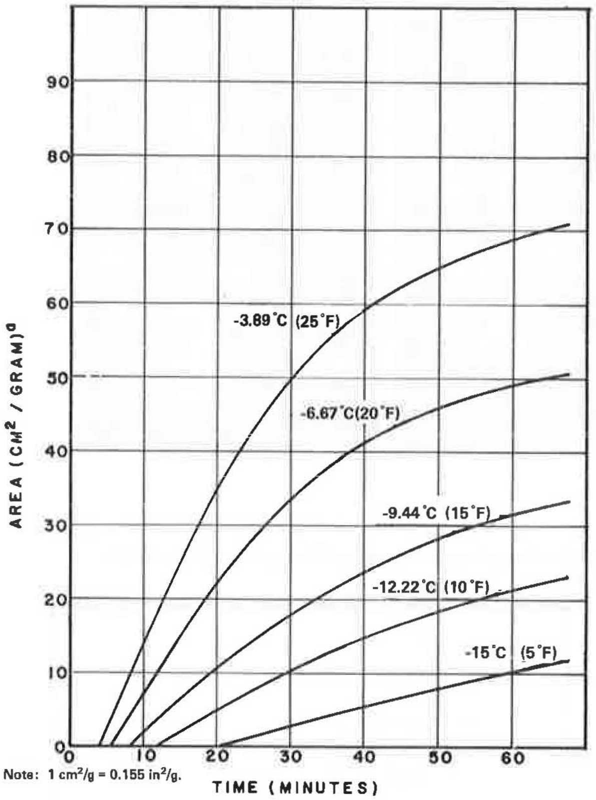
Note: 1 cm²/g = 0.155 in²/g.

undercut by each chemical particle is determined by comparison with the 2-cm (0.79-in) standard length in each picture. The rate at which the ice melts as well as the ultimate area of undercutting can be determined in one experiment at a given temperature.

Calcium Chloride

Experiments on calcium chloride were limited to commercially available anhydrous pellets. The pellets met specifications of ASTM D98 for chemical analysis and grading. Calcium chloride content of various samples was between 94 and 95 percent, and alkali chloride content was between 4 and 5 percent. Particle size was nearly 100 percent passing the 4.75-mm (No. 4) sieve and retained on the 850-μm (No. 20) sieve. About 70 percent was passing the 4.75-mm (No. 4) sieve and being retained on the 2.36-mm (No. 8) sieve. Particles weighing 60 mg were selected as typical and were used to determine the extent of ice melting as a function of time and temperature. The results are shown in Figure 3. After a short period required to melt through the ice, the area undercut increases rapidly and then levels off at a maximum characteristic for each temperature. The maximums relate to each other approximately as expected from solubility data for calcium chloride. The time required to achieve 90 percent of the ultimate action varies from about 30 min at -3.89°C (25°F) to about 1 h at -17.78°C (0°F).

Figure 5. Area of ice undercut by Michigan rock salt as a function of time and temperature.



Note: 1 cm²/g = 0.155 in²/g.

Table 1. Quantity of chemical required for complete undercutting of 0.3175-cm-thick ice.

Time (min)	Temperature (°C)	Quantity (kg/m^2) CaCl₂ Pellets	Rock Salt
15	-3.89	0.18	0.40
15	-6.67	0.29	0.66
15	-9.44	0.44	1.54
15	-12.22	0.65	4.76
15	15	1.25	—[a]
15	-17.78	3.12	—[a]
30	-3.89	0.12	0.20
30	-6.67	0.18	0.30
30	-9.44	0.24	0.55
30	-12.22	0.32	0.97
30	-15	0.44	3.63
30	-17.78	0.61	—[a]
60	-3.89	0.11	0.15
60	-6.67	0.17	0.21
60	-9.44	0.21	0.31
60	-12.22	0.27	0.47
60	-15	0.30	0.98
60	-17.78	0.34	—[a]

Note: 1°C = (1°F - 32)/1.8. 1 kg/m^2 = 0.205 lb/ft^2.
[a]No undercutting by rock salt under these conditions.

Rock Salt

Difficulty was encountered as soon as an attempt was made to develop similar data for rock salt. Particles of Michigan rock salt contain calcium sulfate (anhydrite) in a concentration that, according to Kaufmann (6, p. 322), correlates roughly with particle color. Calcium sulfate dissolves very slowly and greatly inhibits the melting action of the colored particles as shown in Figure 4. Colored particles constitute from 5 to 15 percent of typical samples of Michigan rock salt. Many white particles contain enough calcium sulfate to noticeably slow up their melting action relative to a fused reagent grade sodium chloride that was used as a standard. Because this technique involves only a few particles per experiment, a truly representative sample could not be selected. For the study, measurements were made on a uniformly pure fused reagent grade sodium chloride, and the action of typical Michigan rock salt was estimated to be 90 percent of this standard.

A second difficulty was the selection of a typical particle size for rock salt. Wide variations are allowed by the ASTM specifications for highway deicing salt, and attrition in handling may cause an appreciable change in size distribution. A sample of Michigan rock salt was obtained from a 23-kg (50-lb) bag selected at random from a local supply used for deicing purposes. A screen analysis showed 60 percent passing the 4.75-mm (No. 4) sieve and retained on the 600-μm (No. 30) sieve and 40 percent passing the 9.5-mm (⅜-in) sieve and retained on the 4.75-mm (No. 4) sieve. To correspond with the calcium chloride pellet range, 60-mg particles were selected as typical of the 60 percent fraction and 240-mg particles were selected as typical of the 40 percent fraction. The average action of this sample of Michigan rock salt was taken as 90 percent of the weighted average of 60-mg and 240-mg particles of pure fused sodium chloride. The results for various times and temperatures are shown in Figure 5. At low temperatures, rock salt does not reach its maximum effect in an hour, and at -17.78°C (0°F) no appreciable undercutting occurs in less than an hour.

DISCUSSION OF RESULTS

From this research, one can conclude that at least part of the variation in the data reported in previous studies is due to the inhomogeneity of rock salt. Small samples of rock salt would be expected to vary considerably in purity and particle size. Meaningful comparisons of deicing salts will require careful definition of the actual materials used.

A second factor greatly affecting previous data is the rate of application of chemical. At -6.67°C (20°F), these data indicate that complete undercutting of the ice sheet will be achieved by optimum distribution of about 0.16 kg/m^2 (0.3 lb/yd^2) of calcium chloride pellets. However, Toddie (3) used a loading of about 0.54 kg/m^2 (1 lb/yd^2), and the Brohm and Edwards (4) loadings ranged from 0.39 to 3.9 kg/m^2 (0.72 to 7.20 lb/yd^2). These high loadings imply that many particles were too close together for maximum efficiency and that, in some cases, the amount of ice rather than the amount of chemical was the limiting factor. At temperatures above about -9.44°C (15°F), differences between calcium chloride and rock salt were overwhelmed by the high load factor.

For example, to undercut 70 cm^2 (10.85 in^2) of ice at -3.89°C (25°F) with 1 g of chemical, calcium chloride pellets would require about 20 min and rock salt would require about 60 min. If 3 g of chemical were used, calcium chloride pellets would require about 8 min and rock salt would require about 14 min. Because the first determination made by Toddie (3) and Brohm and Edwards (4) was at 15 min, the 3-g loading would show little difference between the two chemicals; both would have completely loosened the ice in 15 min.

The current concern over the ecological damage done by highway deicing operations demands that minimum quantities of chlorides be used. The current data permit one to calculate the amount of chloride needed to completely undercut 0.3175 cm (1/8 in) of ice as a function of time and temperature. The data given in Table 1 show that in 1 h at -3.89°C (25°F), only slightly more rock salt is needed than calcium chloride pellets. However, the difference increases rapidly if a shorter time is required or if lower temperatures are involved. Thus, if it is desired to clear the ice in 15 min at -3.89°C (25°F), about twice as much rock salt will be needed as calcium chloride pellets. If plowing or scraping is done at 15 min, much of the excess salt will be thrown to the roadside. Similarly, if an hour is allowed but the temperature is -12.22°C (10°F), about twice as much salt is needed. Although the price differential makes rock salt the cheaper agent even at the two to one ratio, the excess chloride charged to the environment should not be dismissed lightly. The data of Table 1 and Figures 3 and 5 should be very helpful in optimizing deicing operations.

CONCLUSIONS

On the basis of the data obtained in this research, two conclusions can be drawn.

1. Wide variations in previous comparisons of calcium chloride and rock salt are probably due to variations in the purity and particle size of the rock salt. Comparisons are meaningless without careful specification of the rock salt sample.
2. Commercial anhydrous calcium chloride pellets and a commercial Michigan rock salt were found to be about equal in undercutting ice at temperatures above -3.89°C (25°F) and a time of 1 h. For more rapid action or for lower temperatures, the quantity of rock salt required is greater than the quantity required for calcium chloride, ranging to severalfold at -9.44°C (15°F) and 15 min. The data are of value in assessing the ecological impact of the excess rock salt required.

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