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Traffic Management in Highway Work Zones and Setting Optimal Maintenance Levels and Rehabilitation Frequencies

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Decisions in Selection of Maintenance Levels of Service

RAM B. KULKARNI and CECIL J. VAN TIL

ABSTRACT

The selection of levels of service for maintenance of the various elements of a highway (e.g., traveled way, drainage, or roadside) is influenced by such multiple, often conflicting, considerations as safety, riding comfort, and aesthetics. Decisions regarding maintenance levels of service are now usually made by maintenance personnel in a generally informal, intuitive manner, on the basis of their experience. In research for NCHRP Project 14-5, a more formal methodology was developed to assist in decisions regarding optimal maintenance levels of service for those highway elements that are subject to the constraints of available money, manpower, and equipment. In Phase II of this study, a user manual was developed to provide detailed instructions for highway agency personnel in the application of the methodology to their specific highway system. The main steps of the user manual are described in this paper. The manual is designed to be self-explanatory: no outside assistance from persons experienced in the formal methodology will be necessary. It is also intended that the manual be comprehensive: the instructions cover all of the steps necessary to implement the methodology within a highway agency. These desirable features of the manual were tested in Arizona, New Jersey, and Virginia. Results of testing indicate that it is practical to develop the necessary inputs for the implementation of the methodology and that a highway agency should benefit by using the methodology, especially when attempting to document the impact of budget cuts on levels of service and to provide an objective and defensible basis for the selection of levels of service.

A primary objective of this paper is to describe a methodology for the selection of levels of service for highway maintenance. A level of service for maintenance of a given highway element defines the threshold deficiency level of the element that should trigger an appropriate maintenance activity (e.g., grass should be mowed when it is 12 in. high or a drainage ditch should be cleaned when 50 percent of its area is blocked). The levels of service affect decisions about where, when, and how much maintenance is required. Thus management responsibilities for work scheduling, work priorities, budget estimation, and resource allocation are significantly influenced by the selection of levels of service.

Decisions about levels of service are now usually made by maintenance personnel in a generally informal, intuitive manner, on the basis of their experience. In research for NCHRP Project 14-5, a more formal methodology was developed to determine the levels of service that maximize highway user benefits subject to the constraints of available resources (money, manpower, equipment, and materials).

In Phase II of this project, a user manual was developed to provide detailed instructions for highway agency personnel in the application of the methodology to their specific highway system. The manual was designed to be self-explanatory: no outside assistance from persons experienced in the formal methodology would be necessary. It was also intended that the manual be comprehensive: the instructions cover all of the steps necessary to implement the methodology within a highway agency. The manual was tested in Arizona, New Jersey, and Virginia. The purpose of this testing was to check whether the instructions in the manual were sufficiently clear and complete to enable a highway agency to implement the methodology within its currently available resources (staff and computer facilities).

The manual is organized in 12 well-defined and distinct steps. The following sections of this paper describe how each step is to be completed and what is intended to be accomplished as a result of completion of the step. Results of testing the manual in the three state agencies are also discussed.

STEP 1: PREPARE A LIST OF MAINTENANCE ELEMENTS

In this first step, the entire highway system is divided into a limited number of physical categories referred to as maintenance elements. For example, these eight might be selected to represent an entire typical highway system:

1. Traveled way, flexible,
2. Traveled way, rigid,
3. Shoulders and approaches,
4. Roadside,
5. Drainage,
6. Structures,
7. Traffic control and service facilities, and
8. Snow and ice control.

Results of initial testing indicate that these eight elements can be used without modifications in all but a few exceptional cases. Exceptional cases in which an element might be deleted from this list might be, for example, delete 2 if an agency had no portland cement concrete pavement in its system, or delete 8 if climate were such that snow and ice control were unnecessary. An example of an exceptional case in which an element would be added is if a ferry system were operated and maintained by the highway agency.

The result of completion of Step 1 is a list of elements selected to represent the entire highway system under study, such as the example.

STEP 2: PREPARE A LIST OF CONSIDERATIONS AND ASSIGN CONSIDERATIONS TO ELEMENTS

In this step, a list of considerations that can be used to evaluate the performance of the maintenance elements previously listed is first prepared. Appropriate considerations from this list are then assigned to each element.

Considerations are the factors that are used to evaluate how well each maintenance element serves its intended function. For example, "safety" is an important consideration by means of which the performance of most of the listed elements may be evaluated, including "traveled way" (both flexible and rigid), "shoulders and approaches," "traffic control and service facilities," and "snow and ice control." However, "safety" would not likely be chosen as an important consideration for the element "roadside."

Following are examples of considerations that might be applicable:

1. Safety,
2. Riding comfort,
3. Preservation of investment,
4. Aesthetics,
5. User cost, and
6. User convenience.

These six considerations should be adequate for use by most highway agencies, and adding to or deleting from this list should be done only in exceptional cases. It should be noted that although "maintenance cost" is an important consideration in the usual sense, it is not included in this list. In this system, maintenance costs are viewed as constraints on the system not as user-related considerations and are accounted for in a subsequent optimization part of the methodology.

To complete Step 2, one or more considerations are assigned to each maintenance element to be used in evaluating it. For example, if the considerations listed were to be used in the evaluation of the maintenance elements listed in Step 1, they might be assigned as presented in Columns 1 and 2 in Table 1. Note that only those few considerations that play a major part in its evaluation are assigned to an element. For example, although "aethestics" might have some part in evaluating other elements, it is assigned only to the element "roadside," where it plays a dominant role.

The assignment of considerations shown in Table 1 should be reasonable for most highway agencies. However, revisions in this table may be made if considered to be essential by the agency.

The result of completion of Step 2 is the assignment of considerations to elements in the form of a table such as Table 1.

STEP 3: SELECT AN ATTRIBUTE FOR EACH CONSIDERATION

In this step, an attribute is selected to express the level of each consideration on a numerical scale. For example, for the consideration "safety," which has been assigned to the maintenance element "traveled way, flexible," the attribute selected might be "percentage change in frequency of accidents."

An attribute provides a numerical scale for measuring the effects of alternative levels of service on a given consideration. There are two general types of attributes to consider--natural and constructed. A natural attribute is one the levels of which are physically measurable. For example, for the consideration "safety," a natural attribute may be "percentage change in frequency of accidents" relative to the elements "traveled way, flexible" and "traveled way, rigid," or it may be "percentage of drivers who cannot recover after driving over edge of traveled way" relative to the element "shoulders and approaches."

A constructed attribute is one for which a physical measurement is not possible. In such cases, a subjective scale or index must be constructed to define the levels of this attribute. For example, the consideration "aesthetics" cannot be measured objectively, so a constructed attribute "degree of pleasing appearance" with a subjective scale of 1 to 4 might be used to define it. Each number on the subjective scale should be described in sufficient detail so that the associated level of impact of each is communicated clearly and unambiguously. Pictures may be used to provide additional communication of a visual nature.

Examples of attributes that might be selected for various considerations are shown in Column 3 of Table 2. Each attribute should be numbered sequentially, as shown. One and only one attribute is assigned to each consideration. Unlike the examples of elements and considerations presented in Columns 1 and 2, which should require little change, the attributes shown in Column 3 are presented as preliminary suggestions only and may be revised or replaced by the user agency.

The result of completion of Step 3 is the selection of an attribute for each of the considerations previously assigned to the elements.

STEP 4: SELECT CONDITIONS FOR EACH ATTRIBUTE

In this step, at least one, but no more than three, maintenance conditions applicable to each of the attributes previously listed is selected. The conditions should be such that, at some level of deficiency of the condition, repair or correction will be required and that a change in the level of the condition would be expected to have an influence on the associated attribute. For example, for the attribute "percentage change in the frequency of accidents" previously selected as an example applicable to the consideration "safety" for the maintenance element "traveled way, flexible," the three conditions "rutting," "slippery surface," and "roughness" might be selected. This example, as well as examples of maintenance conditions that might be selected as applicable to all other examples of attributes presented in Step 3, are presented in Column 4 of Table 3. Note that the same condition may be appropriately used for more than one attribute for a given maintenance element.

Each selected maintenance condition should be such that alternative levels of service could be considered for it. If only one level of service is applicable for a particular condition, it should not be included in this methodology. Thus, for example, nonfunctioning major signals may not be included as a maintenance condition if the policy is to repair these as they are reported.

The examples of conditions in Column 4, like the examples of attributes in Column 3, are presented as preliminary suggestions only. Because all of them have not as yet been tested in trial applications with highway agencies, this list should be used by a highway agency as a guide for preparing its own preliminary list only. Meetings should be held with appropriate specialists to generate lists of conditions that are appropriate for the specific highway agency. To keep the analysis tractable, it is desirable to include in the set of maintenance conditions only those that are of major concern. Usually, it should be possible to define a total of 20 to 25 maintenance conditions for which 70 to 80 percent of the annual maintenance budget is expended.

TABLE 1 Suggested Format for Recording Maintenance System Data, Columns 1 and 2: Assignment of Considerations to Elements

Column 1 Maintenance ELEMENTS	Column 2 Maintenance Element CONSIDERATIONS	Column 3 ATTRIBUTES of the Considerations	Column 4 Maintenance CONDITION Affecting Attributes	Column 5 PARAMETERS for Defining Maintenance Conditions	Column 6 Alternate Maintenance Levels of Service, in Terms of Parameters
Traveled Way, Flexible	*Safety*				
	Riding Comfort				
	User Cost				
	Preservation of Investment				
Traveled Way, Rigid	*Safety*				
	Riding Comfort				
	User Cost				
	Preservation of Investment				
Shoulders and Approaches	*Safety*				
	Preservation of Investment				
Roadside	*Aesthetics*				
	User Convenience				
Drainage	*Safety*				
	Preservation of Investment				
Structures	*Preservation of Investment*				
Traffic Control and Service Facilities	*Safety*				
	User Convenience				
Snow and Ice Control	*Safety*				
	User Convenience				

TABLE 2 Suggested Format for Recording Maintenance System Data, Column 3: Selection of an Attribute for Each Consideration

Column 1 Maintenance ELEMENTS	Column 2 Maintenance Element CONSIDERATIONS	Column 3 ATTRIBUTES of the Considerations	Column 4 Maintenance CONDITION Affecting Attributes	Column 5 PARAMETERS for Defining Maintenance Conditions	Column 6 Alternate Maintenance Levels of Service, in Terms of Parameters
Traveled Way, Flexible	*Safety*	*1. Percent change in frequency of accidents*			
	Riding Comfort	*2. Present Serviceability Index (PSI)*			
	User Cost	*3. Percent increase in excess user costs*			
	Preservation of Investment	*4. Frequency of rehabilitation of pavement*			
Traveled Way, Rigid	*Safety*	*5. Percent change in frequency of accidents*			
	Riding Comfort	*6. Present Serviceability Index (PSI)*			
	User Cost	*7. Percent increase in excess user costs*			
	Preservation of Investment	*8. Frequency of rehabilitation of pavement*			
Shoulders and Approaches	*Safety*	*9. Percent of drivers who cannot recover after driving over edge of traveled way*			
	Preservation of Investment	*10. Percent increase in pavement rehabilitation cost*			
Roadside	*Aesthetics*	*11. Degree of Pleasing Appearance*			
	User Convenience	*12. Degree of cleanliness at rest areas*			
Drainage	*Safety*	*13. Percent of time water accumulates on pavement*			
	Preservation of Investment	*14. Percent of time water accumulates on pavement*			
Structures	*Preservation of Investment*	*15. Percent change in useful life of structures*			
Traffic Control and Service Facilities	*Safety*	*16. Maximum percent of traffic signals which would be ineffective at a given time*			
	User Convenience	*17. Maximum percent of signs, markings, and lights which would be ineffective at a given time*			
Snow and Ice Control	*Safety*	*18. Number of hours road is open under adverse driving conditions*			
	User Convenience	*19. Percent of road mileage closed following storm*			

TABLE 3 Suggested Format for Recording Maintenance System Data, Column 4: Selection of Conditions for Each Attribute

Column 1 Maintenance ELEMENTS	Column 2 Maintenance Element CONSIDERATIONS	Column 3 ATTRIBUTES of the Considerations	Column 4 Maintenance CONDITION Affecting Attributes	Column 5 PARAMETERS for Defining Maintenance Conditions	Column 6 Alternate Maintenance Levels of Service, in Terms of Parameters
Traveled Way, Flexible	*Safety*	*1. Percent change in frequency of accidents*	*1. Rutting*		
			2. Slippery Surface		
			3. Roughness		
	Riding Comfort	*2. Present Serviceability Index (PSI)*	*1. Rutting*		
			3. Roughness		
	User Cost	*3. Percent increase in excess user costs*	*1. Rutting*		
			3. Roughness		
	Preservation of Investment	*4. Frequency of rehabilitation of pavement*	*4. Ravelling*		
			5. Cracking		
			3. Roughness		
Traveled Way, Rigid	*Safety*	*5. Percent change in frequency of accidents*	*6. Slippery Surface*		
			7. Settlement, heave, or distortion		
	Riding Comfort	*6. Present Serviceability Index (PSI)*	*8. Faulting*		
			7. Settlement, heave, or distortion		
	User Cost	*7. Percent increase in excess user costs*	*9. Cracking*		
			8. Faulting		
			7. Settlement, heave, or distortion		
	Preservation of Investment	*8. Frequency of rehabilitation of pavement*	*9. Cracking*		
			10. Spalling		
			8. Faulting		
Shoulders and Approaches	*Safety*	*9. Percent of drivers who cannot recover after driving over edge of traveled way*	*11. Edge of traveled way drop-off*		
			12. Surface deterioration of shoulders		
	Preservation of Investment	*10. Percent increase in pavement rehabilitation cost*	*11. Edge of traveled way drop-off*		
Roadside	*Aesthetics*	*11. Degree of Pleasing Appearance*	*13. Grass Growth*		
			14. Noxious weeds and brush		
			15. Litter and debris		
	User Convenience	*12. Degree of cleanliness at rest areas*	*16. Rest Areas*		
Drainage	*Safety*	*13. Percent of time water accumulates on pavement*	*17. Blocked or damaged drainage structures*		
	Preservation of Investment	*14. Percent of time water accumulates on pavement*	*17. Blocked or damaged drainage structures*		
Structures	*Preservation of Investment*	*15. Percent change in useful life of structures*	*18. Structural deficiencies*		
			19. Structure cleaning and painting		
Traffic Control and Service Facilities	*Safety*	*16. Maximum percent of traffic signals which would be ineffective at a given time*	*20. Traffic signals*		
	User Convenience	*17. Maximum percent of signs, markings, and lights which would be ineffective at a given time*	*21. Signs and markings*		
			22. Lighting		
Snow and Ice Control	*Safety*	*18. Number of hours road is open under adverse driving conditions*	*23. Snow and ice buildup*		
	User Convenience	*19. Percent of road mileage closed following storm*	*23. Snow and ice buildup*		

On completion of the selection, the conditions should be tabulated opposite the attributes to which they are assigned in a form similar to Column 4 in Table 3. Conditions for a given maintenance element must be numbered sequentially, as shown, with a condition assigned the same number, regardless of the number of attributes to which it is assigned.

The results of completion of Step 4 are the selection of one to three maintenance conditions applicable to each of the attributes previously selected, tabulation of these conditions in the appropriate position in the fourth column of a table similar to Table 3, and numbering the conditions as shown for the examples in this table.

STEP 5: ESTABLISH A PARAMETER FOR EACH CONDITION

A parameter to define alternate levels of service for each maintenance condition is established in this step. For example, for the maintenance condition "rutting," an example of a parameter that might be selected to define it is "depth of rut and percentage of lane area affected." This example is the first item in Column 5 of Table 4 and is presented opposite the maintenance condition "rutting" in Column 4. Column 5 also presents examples of parameters that might be used for defining each of the other examples of maintenance conditions listed in Column 4.

Parameters should be capable of being expressed numerically or by simple, easily understood descriptions. The numerical or descriptive definitions should be able to differentiate clearly between different levels of the condition to which the parameter applies. There should be one, and only one, parameter assigned to each condition. A parameter may consist of a single definitive item (such as "skid resistance in terms of skid number at 40 mph" for the condition "slippery surface") or may have two items paired to make a combined definition (such as "depth of rut and percentage of lane area affected" for the condition "rutting" or "width of cracks and percentage of lane area affected" for the condition "cracking").

Where development of a numerical parameter does not appear to be feasible, a descriptive parameter may have to be used. For example, if the parameter selected for the condition "structural deficiencies" relative to the maintenance element "structures" is "appearance when repair should be done," the description of appearance should be as unequivocal as possible. Photographs may be used to supplement the descriptions if they would contribute to a better understanding of the description.

The results of the completion of Step 5 are the establishment of a parameter for defining alternate levels of service for each of the maintenance conditions previously selected and the tabulation of these parameters in the appropriate position in the fifth column of a table similar to the example in Table 4.

STEP 6: SPECIFY ALTERNATE LEVELS OF SERVICE FOR EACH CONDITION

In this step, numerical values of the parameters used to define alternate levels of service for the maintenance conditions are established. A maintenance level of service specifies a threshold value of a parameter that triggers the scheduling of an appropriate maintenance activity. For example, if one alternate maintenance level of service for the parameter "height of grass and width of mowing" is "mow at 8 in. height, full width," maintenance activity in mowing would be scheduled to be done when this condition was reached. Some general guidelines for generating appropriate alternate levels of service are

- The description of each level of service should be definitive and nonambiguous (i.e., it should communicate clearly to maintenance personnel when they are expected to work on a maintenance condition).
- The description of a level of service should not involve complicated measurements on the part of the field maintenance personnel--they would be difficult to make in the field and likely to be ignored. Ideally, only visual inspections and simple measurements, quickly made, should be involved.
- Each of the alternate levels of service should be feasible. For example, if the analysis results in selection of the lowest level of service for a maintenance condition, the agency should be willing to adopt that level of service.
- The resource requirements (dollars, manpower) of the levels of service should be significantly different from each other so that truly different options are represented by each. If two levels of service differ only slightly with respect to their maintenance costs, they might better be combined to represent a single level of service.

At the conclusion of this step, a range of alternate levels of service from the highest (ideal) to the lowest (barely tolerable) will have been generated. A general procedure for developing alternate levels of service follows.

First, department personnel with special knowledge of a given maintenance condition are asked to assume that there are no constraints on resources (dollars, manpower) for alleviating this condition. They are then asked the question: How would you improve the current level of service for this condition? Discussion of this question would normally lead to suggesting a level of service somewhat higher than the current practice within the agency--"ideal" but physically attainable. Next, they are told to assume that a severe cut in budget for this condition has been made and that a reduced level of service will have to be adopted. They are then asked the second question: How would you reduce the current level of service for this condition for this reduced budget? This would normally result in suggesting a level of service considerably lower than the current practice, possibly barely tolerable. With these two levels of service as the upper and lower bounds, and the current level of service between them, three alternate levels of service have now been described. Three levels of service are usually adequate for a condition. However, if the range between them is great, the possibility of one or two additional intermediate levels of service should be considered. Five levels of service should be considered a maximum for all but the most unusual of cases because analysis becomes increasingly more complicated as the number of alternate levels of service increases.

Physical measurement and appearance provide direct measures of levels of service to be maintained in the field and these are the preferred modes. Frequency or quantity of work performed assume that certain levels of service are automatically maintained if the amount of effort or material is expended according to established procedures, without direct measurement of results in the field. Although generally less desirable, frequency or quantity of work may provide reasonable specification of levels of service if direct measurement would be impractical and description of the desired appearance would be too cumbersome.

TABLE 4 Suggested Format for Recording Maintenance System Data, Column 5: Establishment of a Parameter for Each Condition

Column 1 Maintenance ELEMENTS	Column 2 Maintenance Element CONSIDERATIONS	Column 3 ATTRIBUTES of the Considerations	Column 4 Maintenance CONDITION Affecting Attributes	Column 5 PARAMETERS for Defining Maintenance Conditions	Column 6 Alternate Maintenance Levels of Service, in Terms of Parameters
Traveled Way, Flexible	*Safety*	*1. Percent change in frequency of accidents*	*1. Rutting*	*Depth of rut and percent of lane area affected*	
			2. Slippery Surface	*Skid resistance* (SN_{40})	
			3. Roughness	*Mays Ride Meter Index*	
	Riding Comfort	*2. Present Serviceability Index (PSI)*	*1. Rutting*	*Depth of rut and percent of lane area affected*	
			3. Roughness	*Mays Ride Meter Index*	
	User Cost	*3. Percent increase in excess user costs*	*1. Rutting*	*Depth of rut and percent of lane area affected*	
			3. Roughness	*Mays Ride Meter Index*	
	Preservation of Investment	*4. Frequency of rehabilitation of pavement*	*4. Ravelling*	*Severity and percent of lane area affected*	
			5. Cracking	*Width of cracks, and percent of lane area affected*	
			3. Roughness	*Mays Ride Meter Index*	
Traveled Way, Rigid	*Safety*	*5. Percent change in frequency of accidents*	*6. Slippery Surface*	*Skid resistance* (SN_{40})	
			7. Settlement, heave, or distortion	*Height, and percent of lane area affected*	
	Riding Comfort	*6. Present Serviceability Index (PSI)*	*8. Faulting*	*Height, and percent of joints affected*	
			7. Settlement, heave, or distortion	*Height, and percent of lane area affected*	
	User Cost	*7. Percent increase in excess user costs*	*9. Cracking*	*Width of cracks, and percent of lane area affected*	
			8. Faulting	*Height, and percent of joints affected*	
			7. Settlement, heave, or distortion	*Height, and percent of lane area affected*	
	Preservation of Investment	*8. Frequency of rehabilitation of pavement*	*9. Cracking*	*Width of cracks, and percent of lane area affected*	
			10. Spalling	*Width of spalls, and percent of joints affected*	
			8. Faulting	*Height, and percent of joints affected*	
Shoulders and Approaches	*Safety*	*9. Percent of drivers who cannot recover after driving over edge of traveled way*	*11. Edge of traveled way drop-off*	*Average height of drop-off*	
			12. Surface deterioration of shoulders	*Severity of localized depressions*	
	Preservation of Investment	*10. Percent increase in pavement rehabilitation cost*	*11. Edge of traveled way drop-off*	*Average height of drop-off*	
Roadside	*Aesthetics*	*11. Degree of Pleasing Appearance*	*13. Grass Growth*	*Height of grass and width of mowing*	
			14. Noxious weeds and brush	*Number of applications of herbicide per year*	
			15. Litter and debris	*Frequency of clean up of litter and debris*	
	User Convenience	*12. Degree of cleanliness at rest areas*	*16. Rest Areas*	*Frequency of clean up of rest areas*	
Drainage	*Safety*	*13. Percent of time water accumulates on pavement*	*17. Blocked or damaged drainage structures*	*Appearance when repair or clean out should be done*	
	Preservation of Investment	*14. Percent of time water accumulates on pavement*	*17. Blocked or damaged drainage structures*	*Appearance when repair or clean out should be done*	
Structures	*Preservation of Investment*	*15. Percent change in useful life of structures*	*18. Structural deficiencies*	*Appearance when repair should be done*	
			19. Structure cleaning and painting	*Frequency of cleaning and painting*	
Traffic Control and Service Facilities	*Safety*	*16. Maximum percent of traffic signals which would be ineffective at a given time*	*20. Traffic signals*	*Frequency of inspection and priority of corrective measures*	
	User Convenience	*17. Maximum percent of signs, markings, and lights which would be ineffective at a given time*	*21. Signs and markings*	*Frequency of inspection and priority of corrective measures*	
			22. Lighting	*Frequency of inspection and priority of corrective measures*	
Snow and Ice Control	*Safety*	*18. Number of hours road is open under adverse driving conditions*	*23. Snow and ice buildup*	*Frequency of inspection and priority of corrective measures*	
	User Convenience	*19. Percent of road mileage closed following storm*	*23. Snow and ice buildup*	*Frequency of inspection and priority of corrective measures*	

The results of completion of Step 6 are the establishment of three to five alternative levels of service (in terms of the established parameters) for each of the maintenance conditions previously selected and tabulation of these alternate levels of service in Column 6 of Table 4.

For example, the element selected as an example for completion in this and the following steps is "roadside." As shown in Table 4, two considerations were selected for this element--"aesthetics" and "user convenience" (Column 2). The attribute selected for "aesthetics" was "degree of pleasing appearance," and for "user convenience" it was "degree of cleanliness of rest areas" (Column 3). Three conditions were selected as affecting the attribute "degree of pleasing appearance"--"grass growth," "noxious weeds and brush," and "litter and debris." One condition "rest areas" was selected as affecting the attribute "degree of cleanliness of rest areas" (Column 4). The parameters selected to define these four conditions were "height of grass and width of mowing," "number of applications of herbicides per year," "frequency of cleanup of litter and debris," and "frequency of cleanup of rest area," respectively (Column 5).

In Step 6, four alternate levels of service were selected for the condition "grass growth." These were expressed in terms of its parameter "height of grass and width of mowing." Column 6 of Table 5 shows these four alternate levels of service, as well as three alternate levels of service for each of the three other conditions selected for this example. Note that Table 5 is a portion of the table developed in previous steps for this example, showing only those considerations, attributes, conditions, parameters, and levels of service applicable to the one example element "roadside."

STEP 7: DETERMINE EFFECTS OF ALTERNATE LEVELS OF SERVICE ON CONSIDERATIONS

For each of the numerical values of alternate levels of service established for a condition, its effect on the consideration to which it is applicable is determined in this step. The effect on a consideration (e.g., "safety") is estimated in terms of the attribute of that consideration (e.g., "percentage of drivers who cannot recover"). Ideally, the procedure for estimating the effects should be based on objective data (i.e., on field measurements). However, the results of the study in which this system was developed indicated that available data would not be adequate for directly estimating the effects of alternative levels of service. The procedure developed for estimating these effects involves structured interviews with specialists to supplement such data as may be available. This proposed procedure involves the following tasks:

1. Prepare summaries of pertinent information and data available from agency records or the literature.
2. Select two to five specialists to participate in structured interviews. Local experience as well as general background and knowledge of the specialty area and interest in participating in the program are major criteria for selection of these specialists. Distribute the summaries of available information to the specialists in advance of the interviews, with instructions to read and become familiar with the information.
3. Organize a meeting with the specialists. Establish a scale for each attribute and tabulate each scale in a form similar to that shown in Figure 1. Explain the scale of each attribute being evaluated and the consideration and element to which each applies. Also describe the alternate levels of service in terms of the parameter used to define the maintenance condition that affects the attribute. A completed Table 5 for each element involved is used to assist in these descriptions. Review and discuss the summaries of information that were distributed before the meeting.
4. Select and complete the appropriate form, Figure 1, 2, or 3. Figure 1 is used if only one parameter and one condition are involved. Figure 2

TABLE 5 Suggested Format for Recording Maintenance System Data, Column 6: Specification of Example Alternate Levels of Service Related to the Element "Roadside"

ELEMENTS	CONSIDERATIONS	ATTRIBUTES	CONDITIONS	PARAMETERS	Alternate Levels of Service
Roadside	*Aesthetics*	*11. Degree of Pleasing Appearance*	*13. Grass Growth*	*Height of grass and width of mowing*	*1. Mow @ 8" height, full width*
					2. Mow @ 12" height, 30' maximum width
					3. Mow @ 18" height, one machine pass width
					4. Mow for safety reasons only
			14. Noxious Weeds and Brush	*Number of applications of herbicide per year*	*1. Three time per year*
					2. Once a year
					3. Do not apply herbicide
			15. Litter and Debris	*Frequency of clean up of litter and debris*	*1. Once a month*
					2. Once every three months
					3. Once a year
	User Convenience	*12. Degree of Cleanliness of Rest Areas*	*16. Rest Areas*	*Frequency of clean up of rest areas*	*1. Twice a day*
					2. Four time a week
					3. Twice a week

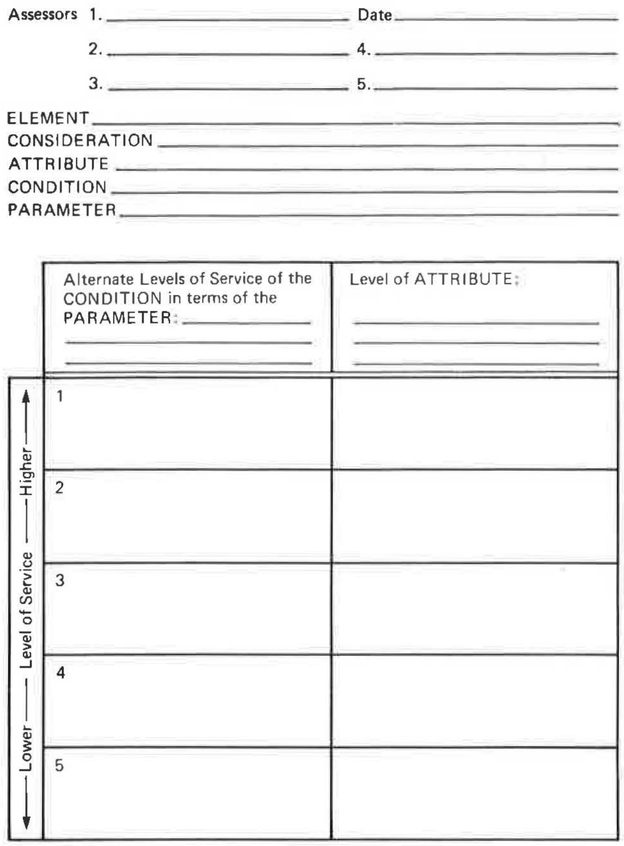

FIGURE 1 Form for recording estimates of the effects of a single maintenance condition on a consideration in terms of its attribute.

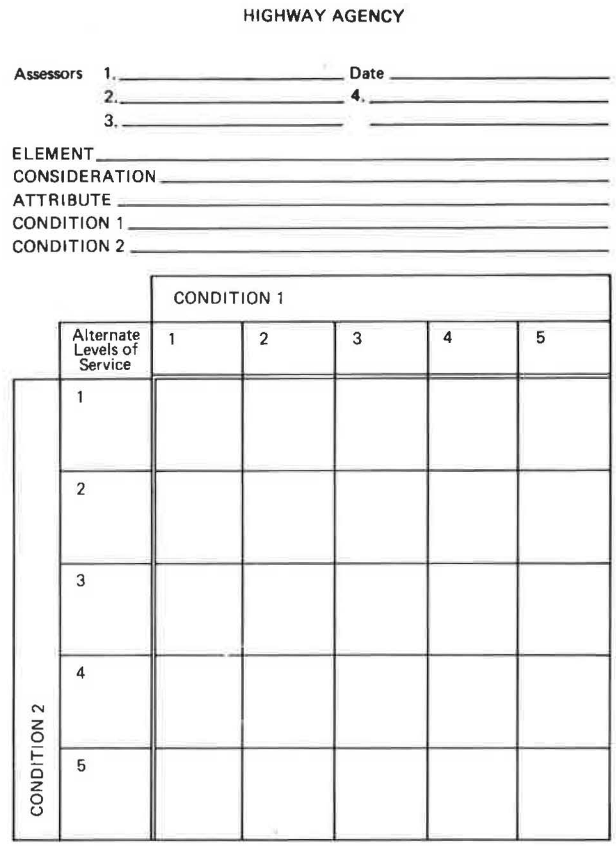

FIGURE 2 Form for recording estimates of the effects of two maintenance conditions on a consideration in terms of its attribute.

is used for two parameters and two conditions and Figure 3 for three parameters and three conditions. The objective of the interview meeting is to obtain a consensus of the specialists regarding the estimates to be entered on the form. Because sufficient objective data are seldom available, the specialists will have to use their judgment, based on experience and logic, to extrapolate from the available data to arrive at the estimates. If significant differences of opinion occur, they should, if possible, be resolved through discussion during the meeting. If these differences cannot be resolved, they should be noted and further investigated during the sensitivity analysis described in a later step.

The result of the completion of Step 7 is a completed form (Figure 1, 2, or 3) for each consideration under study.

A computer program has been designed so that the information from the completed form (Figure 1, 2, or 3) can be directly coded as input data without external calculations.

STEP 8: ESTIMATE RESOURCE NEEDS FOR EACH LEVEL OF SERVICE

In this step the resources required to maintain each maintenance condition at each of its alternate levels of service are determined. The results of these estimates can be conveniently tabulated in the format shown in Figure 4. If a maintenance management system is being used by the highway agency, a significant amount of information needed for this tabulation may be readily available because some of the alternative levels of service may have already been used or considered for use. For alternative levels of service not previously used or considered for use, hard data for estimation of resource requirements will be lacking and judgmental estimates will be required. Best estimates must be made from data available now and from the experience of those making the estimates. With time, more information should become available, and more precise estimates of resource requirements can be made.

The result of the completion of Step 8 is the completion of a form such as the one shown in Figure 4 for each of the conditions and their levels of service developed in previous steps.

STEP 9: ASSESS DESIRABILITY OF EACH LEVEL OF EACH ATTRIBUTE

In this step the relative desirability (value) of the different levels of each attribute selected in Step 7 is assessed. For example, how much better or worse is one level of an attribute (e.g., percentage of drivers who cannot recover = 5) relative to another level of this attribute (e.g., percentage of drivers who cannot recover = 10)? The relative desirability is determined by assessing how much it would be worthwhile to spend to maintain an improved level of the attribute.

This step requires the completion of the following three sequential tasks:

1. Preparation for group value assessments,
2. Conducting group assessment meetings, and
3. Analysis of assessment data.

A description of each task follows.

HIGHWAY AGENCY

Assessors 1. ________ 4. ________ Date ________

2. ________ 5. ________

3. ________ ELEMENT ________ CONSIDERATION ________

ATTRIBUTE No. ________

CONDITION No. 1 ________

No. 2 ________

No. 3 ________

CONDITION No. 3	CONDITION No. 2	CONDITION No. 1: 1	2	3	4	5
1	1					
	2					
	3					
	4					
	5					
2	1					
	2					
	3					
	4					
	5					
3	1					
	2					
	3					
	4					
	5					
4	1					
	2					
	3					
	4					
	5					
5	1					
	2					
	3					
	4					
	5					

FIGURE 3 Form for recording estimates of the effects of three maintenance conditions on a consideration in terms of its attribute.

Maintenance CONDITION ________

Alternate Levels of Service	Resources Required Annually: Type 1 (e.g. Labor–hours or days)	Type 2 (e.g. Materials–dollars)	Type 3 (e.g. Equipment–hours, days, or dollars)
Level 1			
Level 2			
Level 3			
Level 4			
Level 5			

FIGURE 4 Form for tabulating estimates of resources required for alternate levels of service.

Preparation for Group Value Assessments

This task involves the selection of a panel of individuals whose value judgments will be incorporated in the methodology, preparation of assessment forms, and compilation of background information to facilitate assessments.

To obtain value judgments that represents a broad spectrum of viewpoints, it will be desirable (although not necessary) to arrange for the participation of individuals with differing background and experience (e.g., maintenance engineers, legislators, and highways users). Such a panel of individuals should be selected and provided with background information about project objectives, descriptions of selected attributes, and the different levels of each attribute. It will also be useful to compile information about the approximate percentage of the available maintenance budget spent to maintain the current level of each attribute. This can be done by estimating the percentage of the budget spent on different maintenance conditions that affect each attribute.

The final part of this task is to design the assessment forms. One form will be required for each attribute. The basic assessment question is: What maximum proportion of the total available maintenance budget would you be willing to spend in order to maintain a specified level of an attribute? The higher the proportion of the budget people are willing to spend for a particular level of the attribute, the higher the relative value of that level. A typical assessment form is shown in Figure 5.

Conducting Group Assessment Meetings

A group meeting of all the assessors should be held to explain the purpose of the study and the important role of the assessors in the determination of relative weights of different attributes. The selected attributes should be described and, when appropriate, pictures of actual highway conditions displaying different attribute levels should be shown. The format of the assessment forms that each assessor will be asked to complete should be discussed. It will be important to point out that the assessors should use "percentage of the total available maintenance budget" as an indication of the value they placed on maintaining the attribute at each of the levels described, not what might be the actual cost of maintaining it at this level.

Sufficient assessment forms should be completed during the group session to make certain that the assessors understand the concept and to resolve any difficulties that might be faced. The remaining forms may be completed afterwards by each of the assessors and returned to the principal investigator within some specific time period.

HIGHWAY AGENCY

Assessor __________ Date __________

ELEMENT __________

CONSIDERATION __________

ATTRIBUTE __________

Desirability	Level* of Attribute:	Maximum Percent of Total Available Maintenance Budget "Willing to Pay"
Least		
Most		

* Values assessed in Step Seven and recorded on form shown as Figure 1,2 or 3.

FIGURE 5 Form for use in recording each assessor's judgment about the relative desirability of the levels of an attribute.

Analysis of Assessment Data

After receiving the completed assessment forms, the principal investigator proceeds with the analysis of the data. Forms for each attribute are analyzed sequentially. For each given attribute, the following procedure is followed:

1. Responses for each attribute level are arranged in an ascending order.
2. The median of all responses is determined. The median, rather than the mean, is used to represent group consensus because median is not affected much by extreme responses.
3. The relative value of each attribute level is calculated from the following equation:

$$\text{Relative value} = (PB_i - PB_L)/(PB_M - PB_L) \qquad (1)$$

where

PB_i = maximum percentage of budget the group is willing to pay to maintain the attribute at the ith level,

PB_L = maximum percentage of budget the group is willing to pay to maintain the attribute at the least desirable level, and

PB_M = maximum percentage of budget the group is willing to pay to maintain the attribute at the most desirable level.

4. Plot attribute levels on X-axis and the corresponding relative values on Y-axis. Pass a smooth curve through the plotted points. Find the attribute level that corresponds to a relative value of 0.5. This is called the midvalue level of the attribute.

After the analysis for all of the attributes is completed, the relative weight of each attribute is calculated from the following equation:

$$W_i = [(PB_M - PB_L) \text{ for ith attribute}] \div [\Sigma\ (PB_M - PB_L) \text{ for all attributes}] \qquad (2)$$

The result of completion of Step 9 is the calculation of the midvalue level and the relative weight of each attribute.

STEP 10: ORGANIZE AND INPUT DATA FOR COMPUTER PROGRAM

All the data necessary to run the computer program are obtained in Steps 1-9. In Step 10, these data are organized in a format required for the program. Detailed instructions are provided in the User Manual.

STEP 11: RUN COMPUTER PROGRAM AND PRINT OUT RESULTS OF ANALYSIS

In this step the computer program is executed with the input data prepared in the previous step. The program output displays the input data so that their accuracy can be checked and describes the optimum maintenance policy in terms of the preferred level of service for each maintenance condition. Additional parts of the output include the available and used amounts of resources, the contributions of individual attributes to the overall value of the policy, and the overall value itself on a scale of 0 to 1. In addition, results of sensitivity analyses that may have been specified by the user are also printed. The types of sensitivity analyses that could be conducted include change available resources, change relative weights of attributes, include or exclude specified level of service, and find the second-best solution. For each specification of sensitivity analysis, the program finds and displays the optimum maintenance policy.

STEP 12: FORMULATE RECOMMENDATIONS

The program identifies the optimum maintenance policy for given amounts of resources. Before recommendations for implementation of this policy are made, the costs of the policy in terms of resources used should be compared to the resources available. This would help in identifying any imbalance among the different types of resources. For example, the dollar amount of budget may not be fully used, but the number of manhours may be used to the limit. If it were practical to convert some of the dollar amount to additional labor hours (for example, by contracting out some of the work), the program could be rerun with this change to determine whether the selected policy would be affected. If a policy with a higher value is found, this should be taken into account in recommending the selection of a maintenance policy.

It will also be desirable to examine results of sensitivity analysis before making final recommendations. For example, the program might be run to assess the impact of changes in the current maintenance budget on the levels of service. Of particular interest are those situations (such as appreciable reductions in the budget) that could result in significantly lower levels of service. This is useful information to communicate to those responsible for approving maintenance budgets because any adverse effects of budget cuts can be identified explicitly.

RESULTS OF TESTING OF THE USER MANUAL

A draft of the User Manual was initially tested in Arizona and Virginia. Results of this testing indicated that some organizational and editorial changes in the manual would increase the clarity of the instructions. Appropriate revisions to the manual were made to reflect the recommendations of these two agencies. It was encouraging, however, that testing in neither agency required any change in the basic methodology or the computer program.

The revised manual was then tested in New Jersey. No particular difficulty was experienced by New Jersey Department of Transportation personnel in developing the required input data, organizing and entering the data in the computer, executing the computer program, and interpreting program output to establish maintenance levels of service. Although trips were made to Arizona and Virginia to get the testing program started, no such trip was required to New Jersey, nor were any telephone consultations necessary.

Given the different conditions and maintenance practices in the three states involved in the testing program, it appears that the methodology for establishing optimal maintenance levels of service should be applicable to most highway agencies and that the User Manual should enable any agency to implement the methodology without any outside assistance.

CONCLUSIONS

The User Manual developed and tested in this study provides a comprehensive and self-explanatory document that can be used by any transportation agency, without outside assistance, to establish the most appropriate maintenance levels of service for different components of a highway system. The levels of service determined from the methodology described in the manual will maximize user benefits subject to the constraints of available agency resources (dollars, manpower, equipment, and materials). The only major constraint on the use of the methodology is that the agency should have a working maintenance management system in place.

The use of the methodology can be extended beyond highway maintenance levels of service to include levels of service for the maintenance of other modes of transportation and to address the allocation of an overall maintenance budget among all competing modes of transportation.

The computer program documented in the User Manual provides an efficient zero-one integer programming algorithm that can be used, with some modifications, on problems beyond the highway maintenance problem. For example, the question of which highway construction projects should be funded in each year of a multiyear construction program can be analyzed using the algorithm with appropriate modifications.

The potential benefits of implementing the methodology include the following:

- Defensible and well-documented process for establishing maintenance level of service;
- Improved communication among all levels of management and field personnel within the agency regarding maintenance needs and priorities;
- Potential for constructive participation by maintenance engineers, legislators, and citizens in the assessment of the relative importance of evaluation criteria (attributes); and
- Selection of maintenance policies that make optimal use of limited resources.

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Bridge Deck Rehabilitation Decision Making

PHILIP D. CADY

ABSTRACT

Policies for the protection, repair, rehabilitation, and replacement of concrete bridge decks were investigated, with the goal of providing recommendations based on minimum life-cycle costs. Present policies in most states consist of decision matrices or flow diagrams based on a few parameters related to deck condition and, sometimes, to service. Few appear to possess the capacity to reflect the cost-effectiveness of feasible alternative strategies. The development of a mathematical model for evaluating alternative strategies for bridge deck protection, repair, rehabilitation, and replacement, which forms the basis for current policy of the Pennsylvania Department of Transportation, is described. Detailed procedures for data acquisition are presented, and a typical calculation is illustrated.

The existence of a serious national problem, deteriorating bridge decks, was first recognized in the early 1960s when the Portland Cement Association and the Bureau of Public Roads (now the Federal Highway Administration) reported on studies carried out in cooperation with 10 state highway agencies (1,2). In 1973 the Federal Highway Administration (FHWA) estimated the cost for bridge deck repairs in the United States to be $70 million per year (3). Two years later the same agency revised this figure to $200 million per year (4). In 1977 FHWA reported that 65,507 bridges (about 10 percent of the nation's bridges) had badly deteriorated decks (5). In the same year The Road Information Program (TRIP) reported that, in the winter of 1976-1977 alone, 1,626 bridges had been rendered unusable, mainly as a consequence of spalling. Repair or replacement costs for these bridges were estimated to be $1 billion. An Environmental Protection Agency study published in 1977 placed the annual damage to bridge decks at $500 million (6). In 1979 the General Accounting Office (GAO) reported to Congress that the cost for repairing the country's deteriorated bridge decks stood at $6.3 billion (7). A 1981 GAO report (8) estimated the number of deficient bridges to be well in excess of 100,000. The estimated rehabilitation or replacement cost was placed at $33.2 billion. Significant proportions--about one-half of the number of bridges and one-third to one-half of the projected cost--are related to bridge deck deterioration problems. The remainder reflect structural or functional deficiencies.

The major cause of bridge deck deterioration is deicing salts applied to maintain trafficable winter roadway conditions. The problem has become so acute during the past two decades because of the increasing use of deicing salts in pursuit of an all-weather "bare pavement" policy promulgated by highway agencies beginning in the 1950s. In 1947 less than one-half million tons of road salt was used in the United States. Salt usage increased to a peak of 11 to 12 million tons in the mid-1970s. One report indicated that the average road salt application on 25 bridge decks studied in the Denver, Colorado, area was over 1 lb per square foot per year (9).

The deicer salts (sodium chloride and calcium chloride) produce corrosion of the reinforcing steel in concrete. Normally (i.e., at low chloride ion concentrations), concrete provides an environment that inhibits corrosion of reinforcing steel. Corrosion may begin when chloride concentrations reach about 1.2 lb per square yard of concrete at the location of the steel (10). The resulting corrosion products occupy more space than did the original steel, producing stresses that cause the concrete to crack along horizontal fracture planes, which eventually become spalls.

The study reported here was undertaken to develop rational strategies for the protection, repair, rehabilitation, and replacement of bridge decks based on cost-effectiveness. The objective of the study was to provide means for optimizing the allocation of limited funds. The methodology described has since been adopted by the Pennsylvania Department of Transportation as official policy.

POLICIES

Policies for the protection, repair, rehabilitation, and replacement of concrete bridge decks require decision making at two levels. First, criteria are necessary to define the points at which various actions are required, and, second, decisions must be made about what action shall be taken. The action criteria for five current or recent bridge deck policies are given in Table 1 as typical of the range of prevailing attitudes. With the exception of the criterion for replacement in the Ontario policy, the decision criteria are quite arbitrary--a factor that is underscored by the wide variation in the values presented in Table 1.

When action has been triggered by the appropriate criterion, a decision is needed about methodology. Figure 1 shows a compendium of the potential methods for effecting protection, repair, rehabilitation, or replacement of bridge decks. Some of the methods presented are still experimental (e.g., deep polymer impregnation) and some are not currently in favor due to questionable effectiveness (e.g., galvanized reinforcement) or technical problems (e.g., internally sealed concrete).

The review that was carried out for the development of the current Pennsylvania Department of Transportation policy revealed little rationale, nationwide, both for the selection of trigger levels for action criteria and for the selection of method after the need for action is indicated. Furthermore, contemplation of current practices and policies from several points of view inevitably led to the deduc-

TABLE 1 Action Criteria Associated with Some Current or Recent Bridge Deck Policies

Percent of Deck With:	FHWA (11)			AASHTO (12)			Minn. (13)			Neb. (12)			Ontario (14)		
	Spot Repair/ Protect	Reha- bili- tate	Re- place	Spot Repair/ Protect	Reha- bili- tate	Re- place	Spot Repair/ Protect	Reha- bili- tate	Re- place	Spot Repair/ Protect	Reha- bili- tate	Re- place	Spot Repair/ Protect	Reha- bili- tate	Re- place
Visible Spalls (incl. patched areas)	0%, or	0-5%, or	>5%, or	≤ 10% (combined) or	11-59% (combined) or	≥ 60% (combined) or	< 20% (combined)	20-40% (combined)	> 40% (combined)	0% or	≤ 25% (combined) or	≥ 25% (combined) or	No policy criteria	>5% or	If economic evaluation dictates
Delaminations	0-5% (combined)	5-40% (combined)	> 40% (combined)												
Corrosion potentials > 0.35 v (CSE)				≤10%	11-59%	≥60%				<25%	25-50%	>50%		>20%	

tion that the most rational approach is one that results in minimum life-cycle costs for bridges, given present deck conditions. This has the effect of combining the two levels of decision making into only one that has an identifiable and quantifiable basis (life-cycle cost). In broad terms the policy developed for the Pennsylvania Department of Transportation involves the identification of technically feasible alternative strategies for perpetual service for each bridge deck, based on the present condition of the deck, and evaluation of the alternatives using accepted engineering economic analysis procedures.

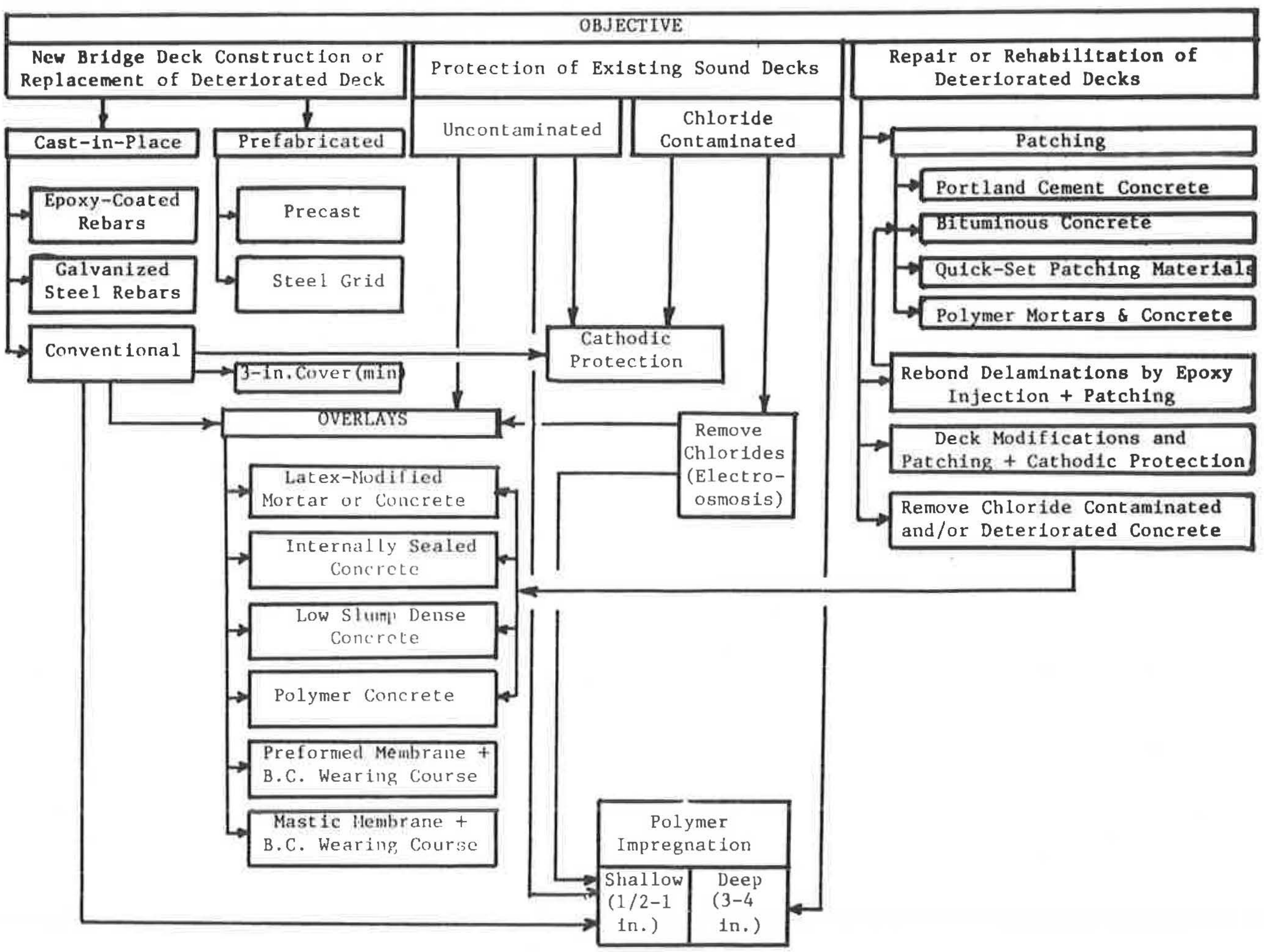

FIGURE 1 Potential methods for effecting protection, repair, rehabilitation, or replacement of bridge decks.

ECONOMIC ANALYSIS MODEL

Scenarios and Possible Actions

At any given time in the life of a given bridge deck, one of two scenarios is applicable: the deck is either sound or deteriorated to some degree. If sound, nothing can be done or the deck can be protected from future deterioration. The protective action taken will depend on whether the deck is critically contaminated with chloride ion (as shown in Figure 1). If deteriorated, the actions available depend on the degree of deterioration (Figure 1) and, in order of increasing severity, fall into the following categories:

- Spot patching (bituminous or rigid),
- Bituminous concrete overlayment (for rideability),
- Rehabilitation (using rigid overlays following removal of deteriorated concrete), and
- Deck replacement.

Depending on the present deck condition and the actions taken in a particular alternative, the likelihood exists that a deck will eventually have to be replaced. Therefore, it is also necessary that the expected life-cycle cash flow situation for deck replacement, based on the state-of-the-art situation for new deck construction, be determined.

Planning Horizon

In general, bridge deck sites are long-lived features. Therefore, it is most convenient and sufficiently accurate in most instances to assume perpetual service as opposed to the selection of a specific planning horizon. However, if it is known that a specific bridge site will be used for less than 50 years, a planning horizon of specific length should be used. The cost difference between 50-year, or greater, and perpetual service is small relative to the uncertainties in predicting future actions and costs. Therefore, the economic model presented here will be based on perpetual service.

Model

The economic model used here involves the determination of the present worth of perpetual service (capitalized cost) for each alternative using the principles of engineering economic analysis. Discrete cash flow and discrete compounding are assumed. In generalized form the mathematical model is

$$\text{Capitalized cost} = A + B(P/A,i,f) + \sum_{n=1}^{a} [C_n(P/G,i,b_n) + D_n(P/A,i,b_n)]\,(P/F,i,c_n) + \sum_{o=1}^{d} E_o(P/F,i,e_o) + (P/F,i,f)(1/i)\Bigg((A/P,i,g)\Bigg\{F + \sum_{p=1}^{h} [H_p(P/G,i,j_p) + J_p(P/A,i,j_p)] \times (P/F,i,k_p) + \sum_{q=1}^{\ell} K_q(P/F,i,m_q)\Bigg\} + G\Bigg)$$

where

A = initial repair costs;
B = uniform annual maintenance and operating costs for present deck from present to time of deck replacement;
C = annual increases in maintenance costs for present deck due to increasing deterioration (e.g., spall patching);
D = cost in first year of annually increasing maintenance costs for present deck;
E = single future expenditures for present deck;
F = first cost of replacement deck;
G = uniform annual maintenance and operating costs for replacement deck over life of replacement deck;
H = annual increases in maintenance costs for replacement deck due to increasing deterioration (e.g., spall patching);
J = cost in first year of annually increasing maintenance costs for replacement deck;
K = single future expenditures for replacement deck;
a = number of periods of increasing maintenance costs for present deck;
b = duration of increasing maintenance costs for present deck due to progressive deterioration;
c = time from present to beginning of increasing maintenance costs for present deck due to progressive deterioration;
d = number of single future expenditures for present deck;
e = time to single future expenditures for present deck;
f = time to expected deck replacement;
g = life of replacement deck;
h = number of periods of increasing maintenance costs for replacement deck;
i = interest rate (decimal);
j = duration of increasing maintenance costs for replacement deck due to progressive deterioration;
k = time to beginning of increasing maintenance costs for replacement deck (from time of deck replacement);
ℓ = number of single future expenditures for replacement deck;
m = time to single future expenditures for replacement deck (from time of replacement);
n,o,p,q = counters;
(A/P) = capital recovery factor (A/P,i%,n) $= i(1+i)^n/[(1+i)^n - 1]$;
(P/A) = uniform series present worth factor (P/A,i%,n) $= [(1+i)^n - 1]/i(1+i)^n$;
(P/F) = single payment present worth factor (P/F,i%,n) $= 1/(1+i)^n$; and
(P/G) = gradient present worth factor (P/G,i%,n) $= (1/i)\{[(1+i)^n - 1/i(1+i)^n] - [n/(1+i)^n]\}$.

A cash flow diagram representing the mathematical model is shown in Figure 2. The model is readily adaptable to microcomputer application.

Costs and Service Lives

The primary data needed for determination of capitalized cost of alternative strategies are the costs and service lives of the components of the strategies. The problems involved in the accurate determination of service lives for systems lacking in field experience are obvious. Cost data are less difficult to determine, but they are also less generally applicable. That is, average cost figures have virtually no meaning for individual cases. This point is emphasized in NCHRP Synthesis of Highway Practice 57 (15,p.44):

> Wide variations in costs can be expected for the same method of repair applied to dif-

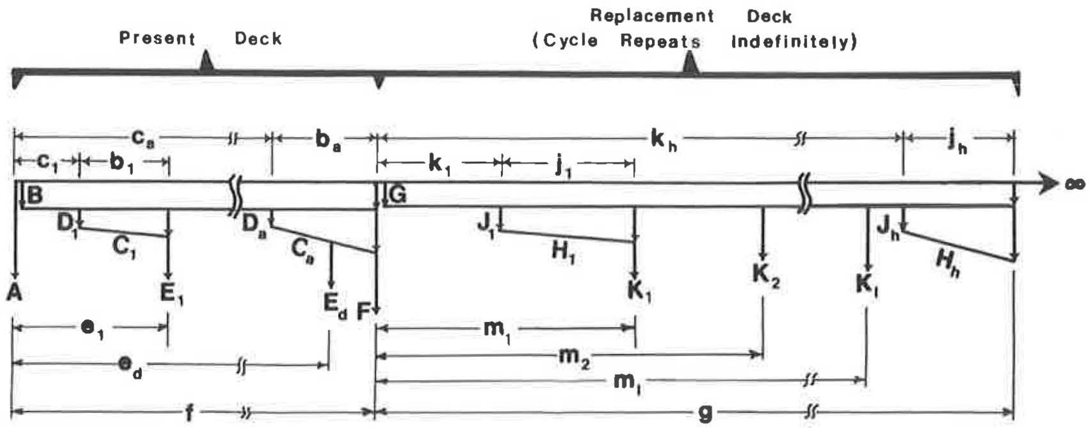

FIGURE 2 Cash flow diagram for capitalized cost evaluation of bridge deck strategies.

ferent structures depending upon the size and location of the structure, traffic volumes, other work included in the same contract, scheduling, and the overall volume of construction work at the time of bidding.

The costs associated with the protection, repair, rehabilitation, and replacement of bridge decks may be classified as follows:

- Installation;
- Annual maintenance;
- Traffic maintenance and protection; and
- Road user costs associated with periods of construction, repair, rehabilitation, or maintenance.

Installation costs are those costs associated with the installation or replacement of a system (e.g., an overlay or a cathodic protection system). They include all direct costs for labor, materials, and deck preparation or modification.

Annual maintenance costs cover the more or less continuous activities necessary to maintain a level of serviceability. They are generally considered to consist of a series of equivalent end-of-year costs that are generally presumed to remain constant or to increase uniformly over a stated period of time. An example is the costs of periodically patching potholes or spalls with bituminous concrete.

Traffic maintenance and protection costs and road user costs to be considered are those associated with periods of construction, repair, rehabilitation, or maintenance.

Within the span of the planning horizon, each alternative might entail several actions, each of which has a service life. For example, a particular strategy for maintaining the serviceability of a certain bridge deck in perpetuum (infinite planning horizon) may involve installing a new deck every 50 years and a new waterproof membrane and wearing course every 10 years. The service lives of the deck and membranes, therefore, are 50 and 10 years, respectively.

The difficulty in attempting to predict service lives is somewhat mitigated by two factors:

- As service life increases, variation in service life has diminishing effect on calculated equivalent cost. As noted previously, there is little difference, economically, between a long service life (50 to 100 years) and infinite service life.
- If the average service lives of relatively short-lived actions are reasonably well known, rather large variations on an individual basis will have relatively little effect over the long run. For example, it can be shown that, if the service life for a particular action falls between 9 and 21 years, 95 percent of the time (typical of rigid overlays), the equivalent cost based on using an average life of 15 years will underestimate the true cost by only 4 percent in the long run (16).

Interest Rate

Interest rate is the expression of the time value of money in engineering economic evaluations. Prevailing interest rates are generally not appropriate because they include an inflation factor. The true cost of long-term borrowing is considered to be on the order of 4 to 6 percent (17). There are conditions in the evaluation of alternatives in the highway sector where inflation should be taken into account. For a detailed discussion of the latter point see Cady (18). In the Pennsylvania Department of Transportation policy the interest rate is taken to be 5 percent and inflation is ignored.

PROCEDURE

The procedure involved in applying the bridge deck protection, repair, rehabilitation, or replacement policy described in this paper is summarized in Figure 3.

Uncovered Decks

The average rebar cover should be determined for all bridge decks not overlaid with bituminous concrete using commercial devices available for this purpose. It has been reported that the standard deviation for rebar cover on bridge decks averages about 0.4 in. (19). Assuming that the reported ±1/8-in. accuracy (15) of the cover measurement equipment represents the 95 percent confidence interval, the number of random readings required to determine the average rebar cover within the accuracy of the equipment is

$$[(1.960)(0.4)]/(0.125)^2 = 40$$

Annual inspections of uncovered decks should include

- Visual examination;
- Sounding (delamination detection); and
- Coring, if necessary.

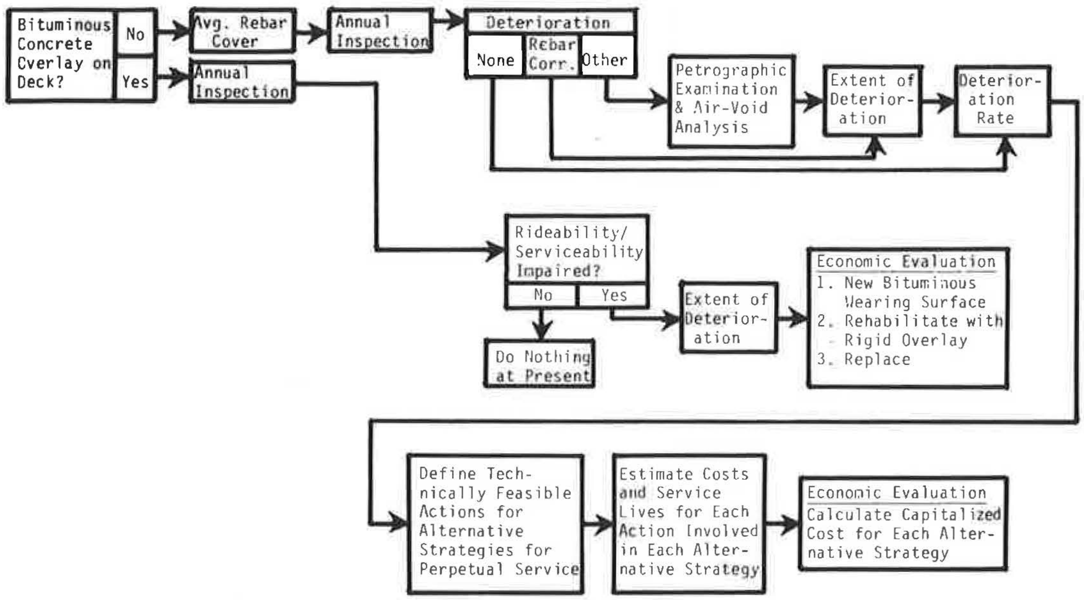

FIGURE 3 Summary of policy for protection, repair, rehabilitation, or replacement of bridge decks.

Visual inspection provides rough, approximate answers to two questions relative to bridge deck condition: what are the sources and what is the extent of the problem? The experienced observer can, from the physical appearance of forms of deterioration present, usually determine the sources of problems. Although corrosion of reinforcement is the most serious cause of bridge deck distress, it is not the only problem area found on bridge decks. The appropriate actions will, of course, vary with the nature of the distress. For example, if a bridge deck is deteriorating under the action of freezing and thawing as a result of insufficient air entrainment, cathodic protection will do nothing to mitigate the problem. The extent of visible deterioration should be mapped. Photographs should be taken for documentation.

Sounding to determine the extent of fracture planes (delaminations) that have not yet produced visible surface manifestations should be carried out using a chain drag or equipment commercially available for this purpose.

Coring would be done only if necessary to determine the cause or causes and extent of deterioration other than reinforcement-corrosion-related for the purposes of strength, petrographic, and air-void analyses. The general rule for core sampling uncovered decks is at least one specimen per 2,000 ft² (20).

Annual inspections provide the data required to define the extent of needed repair or rehabilitation and the rates at which deterioration may be expected to proceed. Deterioration rates are important even for decks found to be in sound condition because they permit evaluation of protection alternatives. Whenever possible, deterioration rates for individual decks should be based on successive annual inspections. This, of course, is not possible for decks that have not yet begun to deteriorate, nor for the first evaluation of any deck. If the cause or potential cause of deterioration is rebar corrosion, a technique is available for estimating the time to development of deterioration and the deterioration rate based on average rebar cover (21).

When the deck condition (including the time to expected deterioration of currently sound decks) and deterioration rate have been determined, alternative strategies for perpetual service can be defined. The costs and service lives associated with the actions contained within each alternative strategy must then be estimated. Finally, the appropriate strategy for the bridge is determined by comparing capitalized costs computed using the economic analysis model.

Covered Decks

Annual visual inspections should also be carried out on bridge decks covered with bituminous concrete wearing surfaces. When rideability or serviceability conditions require remedial action, the structural adequacy of the deck should be evaluated and economic analyses carried out to determine whether the deck should (a) receive a new wearing course; (b) be rehabilitated with a rigid, low-permeability overlay; or (c) be scheduled for replacement. If coring is necessary to ascertain the condition of a deteriorating covered deck, at least one specimen per 500 to 700 ft² should be obtained (20).

APPLICATION

The application of the methodology described in this paper will be demonstrated for one alternative solution involving a common bridge deck scenario.

Bridge Data

- Average rebar cover by Pachometer survey = 1.8 in.,
- Percentage of deck spalled (including areas previously patched with bituminous concrete) = 4 percent, and
- Deck age = 10 years.

There are no prior inspection data.

Evaluate the Alternatives

- Continue patching until 20 percent of the deck surface is deteriorated (spalls, bituminous concrete-patched spalls, and delaminations);

• Apply bituminous concrete overlays (estimated life 8 years) until 40 percent of the original deck surface is deteriorated (spalls and fracture planes);

• Apply latex-modified concrete (LMC) overlay; or

• Install new deck with epoxy-coated rebars 10 years after the LMC overlay.

Procedure

1. Using Figure 5 and Equation 21 in Cady and Weyers (21), the expected deterioration (spalls and delaminations) at the beginning of the maintenance (patching with bituminous concrete) period = (0.5) (10%) = 5%.

2. Using Figure 6 in Cady and Weyer (21), the estimated deck age at rehabilitation (LMC overlay) = 20 years (i.e., 10 years from present).

3. Using Figure 7 in Cady and Weyer (21), the estimated deck age at the beginning of maintenance (bituminous concrete patching) period = 5 years (i.e., 5 years ago).

4. Calculate the estimated deterioration rate: (40% - 5%)/(20 yr - 5 yr) = 2.3%/yr.

5. Calculate the time to bituminous concrete overlaying from the present: Percentage deterioration at present = (4)(4%) = 16% based on the assumption that the area of visible spalls is typically about one-fourth of the total deteriorated area (spalls and delaminations). Time to bituminous concrete overlay = (20% = 16%)/2.3%/yr = 1.7, or 2 yr.

6. It has estimated (see item 2) that the deck will have to be rehabilitated (LMC overlay) in 10 years. Therefore, the period of time over which bituminous concrete overlays will be used is 10 - 2 = 8 years.

7. Estimated costs and services lives:

Action	(Cost ($/ft²)	Expected Life (yr)
Bituminous concrete patching	1.23	0.67
Bituminous concrete overlay	0.44	8
Deck modifications	1.00	
Traffic maintenance and protection	0.25	
Total	2.92	
Latex-modified concrete overlay	4.09	10
Scarification of 60% of deck surface at $0.61/ft²	0.37	
Concrete removal to 1 in. below reinforcing steel on 40 percent of deck surface at $14.75/ft²	5.90	
Traffic maintenance and protection	1.00	
Total	11.36	
New deck (epoxy-coated reinforcing bars)	14.29	40
Old deck removal	11.56	
Traffic maintenance and protection	2.00	
Total	27.85	

8. Estimate the period of patching on the replacement deck having epoxy-coated rebars. Research work carried out by the Federal Highway Administration (22) indicates that epoxy-coated rebars should give about five times as long maintenance-free service as black steel. This translates to average values of 35 years for 2-in. average cover to 70 years for 2-in. minimum cover. Therefore, assume 40 years for the expected maintenance-free life of epoxy-coated rebar reinforced bridge decks. At the end of this period, assume that the deterioration rate is the same as for black steel. (a) Therefore, patching on the replacement deck begins 40 years after installation = 40 + 20 = 60 years from present. (b) Assuming 2.1%/yr deterioration rate (21) for 20% deterioration (the point at which bituminous concrete overlay is required), period of bituminous concrete patching = 20%/(2.1%/yr) = 9.5 or 10 yr (i.e., years 61-70).

9. Assume an 8-year life for the bituminous concrete overlay and a 10-year life for the subsequent LMC overlay.

10. Summary of actions to be taken.

Step	Time (yr)	Action
1.	0 (present) -2	Patch with bituminous concrete
2.	2	Overlay with bituminous concrete
3.	10	Overlay with latex-modified concrete
4.	20	Replace deck (epoxy-coated rebars)
5.	61-70	Patch with bituminous concrete
6.	70	Overlay with bituminous concrete
7.	78	Overlay with latex-modified concrete
8.	88	Repeat Steps 4-7 in perpetuum.

11. Interest rate: Assume average long-term borrowing rate that includes effects of infilation ("true" interest rate) = 5 percent.

12. Cash flow diagram--see Figure 4.

13. Calculations

A = 0

B = 0

C_1 = [($1.23/ft²)/0.67 yr] (0.023)(1/4) = $0.011/ft²yr

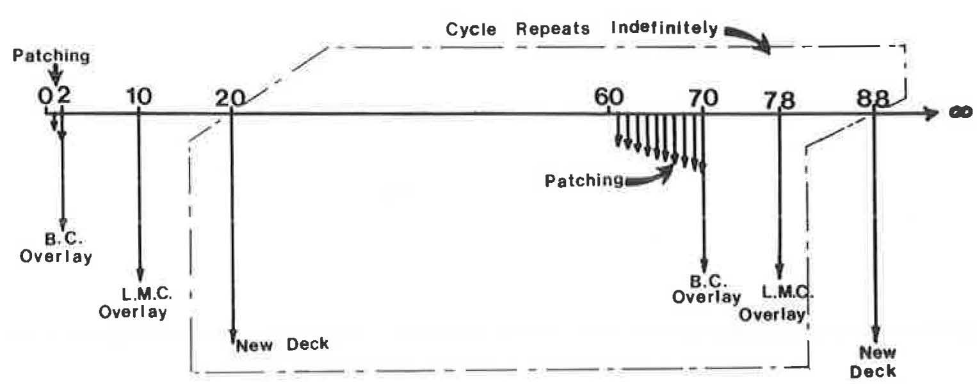

FIGURE 4 Cash flow diagram for example.

D_1 = [($1.23/ft^2)/0.67 yr] (0.04) = $0.073/ft^2/yr (presently 4% of deck is spalled)
E_1 = $1.69/ft^2
E_2 = $11.36/ft^2
F = $27.85/ft^2
G = 0
H_1 = [($1.23/ft^2)/0.67 yr] (0.021)(1/4) = $0.010/ft^2/yr
J_1 = [($1.23/ft^2)/0.67 yr] (0.021)(1/4)(1) = $0.010/ft^2/yr
K_1 = $1.69/ft^2
K_2 = $11.36/ft^2
a = 1
b_1 = 2 yr
c_1 = 0 yr
d = 2
e_1 = 2 yr
e_2 = 10 yr
f = 20 yr
g = 68 yr
h = 1
i = 0.05 (5%)
j_1 = 10 yr
k_1 = 40 yr
ℓ = 2
m_1 = 50 yr
m_2 = 58 yr

$$
\begin{aligned}
\text{Capitalized cost} = {} & 0 + 0 + [(0.011)\overset{0.906}{(P/G,5\%,2)} \\
& + (0.073)\overset{1.8594}{(P/A,5\%,2)}]\overset{1.0000}{(P/F,5\%,0)} \\
& + [(1.69)\overset{0.9070}{(P/F,5\%,2)} + (11.36) \\
& \overset{0.6139}{(P/F,5\%,10)}] + \overset{0.3769}{(P/F,5\%,20)}(1/0.05) \\
& \Big(\overset{0.05188}{A/P,5\%,68)}\{27.85 + [(0.010) \\
& \overset{31.649}{(P/G,5\%,10)} + (0.010) \\
& \overset{7.7216}{(P/A,5\%,10)}]\overset{0.1420}{(P/F,5\%,40)} \\
& + (1.69)\overset{0.0872}{(P/F,5\%,50)} + (11.36) \\
& \overset{0.0590}{(P/F,5\%,58)}\} + 0\Big) = \$19.89/\text{ft}^2.
\end{aligned}
$$

Similar calculations would be carried out to evaluate the other technically feasible strategies for this bridge deck. The course of action is then dictated by the lowest capitalized cost.

A word of caution regarding the foregoing example: The costs and service lives of the various actions involved in the strategy shown are presented for illustrative purposes only. The values of these parameters (particularly costs) may be expected to vary widely with time, geographic location, and other factors. This underscores the need for sound engineering judgment in developing and costing out the strategies for bridge decks on an individual basis.

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Setting Maintenance Levels for Aggregate Surface Roads

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ABSTRACT

Aggregate surface road maintenance activity frequency guides based on minimizing total maintenance and user costs were developed for Bolivia's national highway department. The guides result from predicting road roughness and simulating the operation of vehicles on the road. Presented are equations that predict road roughness as a function of traffic volume and equations that relate road roughness to vehicle operating costs in Bolivia.

Aggregate surface road ride quality is defined by the service level for surface maintenance activities. Many agencies execute maintenance in response to surface condition. Others base maintenance frequencies on resource availability (e.g., the number of motor graders that are operational).

Responding to condition on the basis of judgment depends on the person controlling the maintenance activity. Because supervisors change and there are a number of different persons controlling maintenance in any jurisdictional area, there will be a lack of uniformity when this approach is used.

When maintenance service levels are based on resource availability, deficient levels may result. It is useful, therefore, to have objective guides to use in establishing service levels and resource requirements.

SETTING MAINTENANCE LEVELS FOR AGGREGATE SURFACE ROADS

One basis for establishing objective guides is to compare the costs of alternatives, not only agency costs but user costs. Evaluating the effect of road condition on user costs has received considerable attention in recent years. The World Bank has encouraged and supported a number of studies worldwide to develop relationships between road surface conditions and user costs (1-4). These relationships allow analyses to be made of the user costs to operate on roads in different condition (5-7).

Vehicle operating costs are influenced by the road's traveling surface. This is where the interaction between the vehicle and the road occurs. Therefore, it is primarily defects in this traveling surface that adversely affect road users.

Maximum benefits to the road user occur when a road is kept in its newly constructed condition. This is not economically practical so a lesser level is always sought. The optimum economic level is determined by comparing the costs to maintain or rehabilitate the road with the costs to users at different levels of deterioration. A level is selected on the basis of a strategy that minimizes total overall costs. This optimum strategy depends on the number and composition of users plus the characteristics of the road and the environment in which the road is situated.

From 1981 through 1983, the Bolivian national highway department, Servicio Nacional de Caminos (SNC), conducted studies to improve their highway maintenance practices. The single most costly maintenance activity performed by SNC is aggregate road surface maintenance. Consequently, objective criteria were developed to set maintenance levels for this work. The criteria proposed for establishing maintenance service levels were to minimize total maintenance and user costs. The maintenance levels were defined by specifying the frequency of surface maintenance activities.

ANALYSIS REQUIREMENTS

The analysis suggested as a basis for establishing maintenance frequencies required that the following information be determined for a road section:

1. Quantitative measurement of road surface condition and the ability to predict this condition,
2. Quantitative measurement of the change in the road surface condition that can be achieved through maintenance or rehabilitation,
3. Cost of any maintenance or rehabilitation,
4. Volume and composition of traffic on the road, and
5. Road user costs to operate on the road for each condition.

Surface Condition

For aggregate surface roads, the condition that most influences the motorist is ride quality, defined by road roughness. A number of different measurement units have been developed to define roughness, most of which relate to the measurement process. However, standard roughness units have not yet been defined.

In the Brazil cost study (3), a series of equations was developed to relate road roughness to vehicle operating costs. The unit of roughness used in those equations was termed quartercar index (QI). These equations were modified to reflect high-altitude vehicle use in Bolivia, and the QI units were adopted as a measure of roughness. The units represent the response of a quartercar (Figure 1) to a road profile. In Brazil QIs were assigned to road calibration sections by

1. Running a GM profilometer over a 300-m road section to get the wheel path profile,
2. Electronically simulating the response of a mathematically defined quartercar to the profilometer-generated profile at a simulated speed of 34 mph, and
3. Defining the response as counts per kilometer (the quartercar simulated a BPR roughometer and generated inches of displacement).

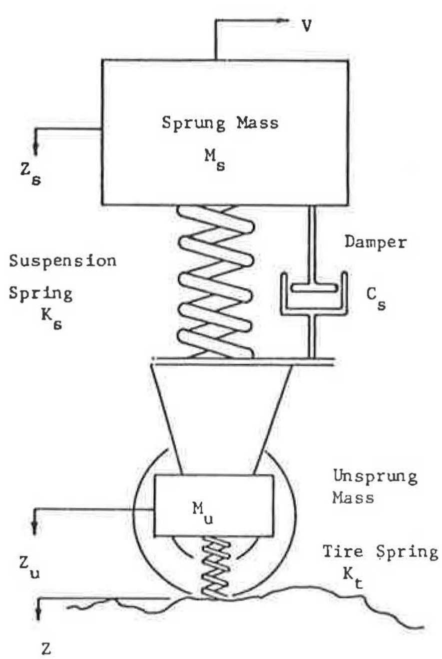

FIGURE 1 Quartercar model.

QI was therefore a dimensionless unit assigned to a road section to define the section's roughness. Twenty such sections made up a calibration course. Maysmeters were run over the calibration course and an equation was established to convert the Maysmeter numbers generated at 300-m sampling intervals to QI units.

Road Roughness Prediction

A study was initiated to evaluate unpaved road performance in Bolivia. A Maysmeter was used to monitor the deterioration of a newly regraveled road with average annual daily traffic (AADT) of 672. The regraveling included watering and rolling. The Maysmeter roughness measurements obtained during March and April 1982 are shown in Figure 2.

Three existing unpaved road roughness prediction equations were examined. The first was the Kenya equation (8) that predicts roughness in millimeters per kilometer and is a function of accumulated traffic:

$$R = 3250 + 84T - 1.62T^2 + 0.016T^3 \qquad (1)$$

where R is roughness (mm/km) and T is cumulative traffic volume in both directions that has used road since grading, measured in thousands of vehicle passes.

The second roughness equation was taken from a preliminary Brazil report (9) and is as follows:

$$\ln QI = \ln RA + D(0.0070 + 0.000013\ AADT - 0.0036S - 0.000035RA - 0.00000006\ AADT \times RA - 0.0136\ W/RAD) \qquad (2)$$

where

ln QI = natural log of roughness in QI units;
ln RA = natural log of roughness following grading in QI units;
D = days since last blading;
AADT = average annual daily traffic in both directions;
S = seasonal variable where dry = 0, wet = 1;
W = road width in meters; and
RAD = curve radius in meters.

The last equation examined was developed at the University of Texas by A. Visser (10). Visser used data from Brazil and developed the following equation:

$$\ln QI = \ln RA + D[0.4314 - 0.1705T2 + 0.001159\ NC + 0.000895\ NT - 0.000227NT * G + S\ (-0.1442 - 0.0198G + 0.00621\ SV - 0.0142PI - 0.000617NC)] \qquad (3)$$

where

ln QI = natural log of roughness, in QI units;
ln RA = natural log of roughness following grading, in QI units;
D = number of days since last blading, in hundreds;
T2 = surface-type variable, 1 = clay, 0 = other;

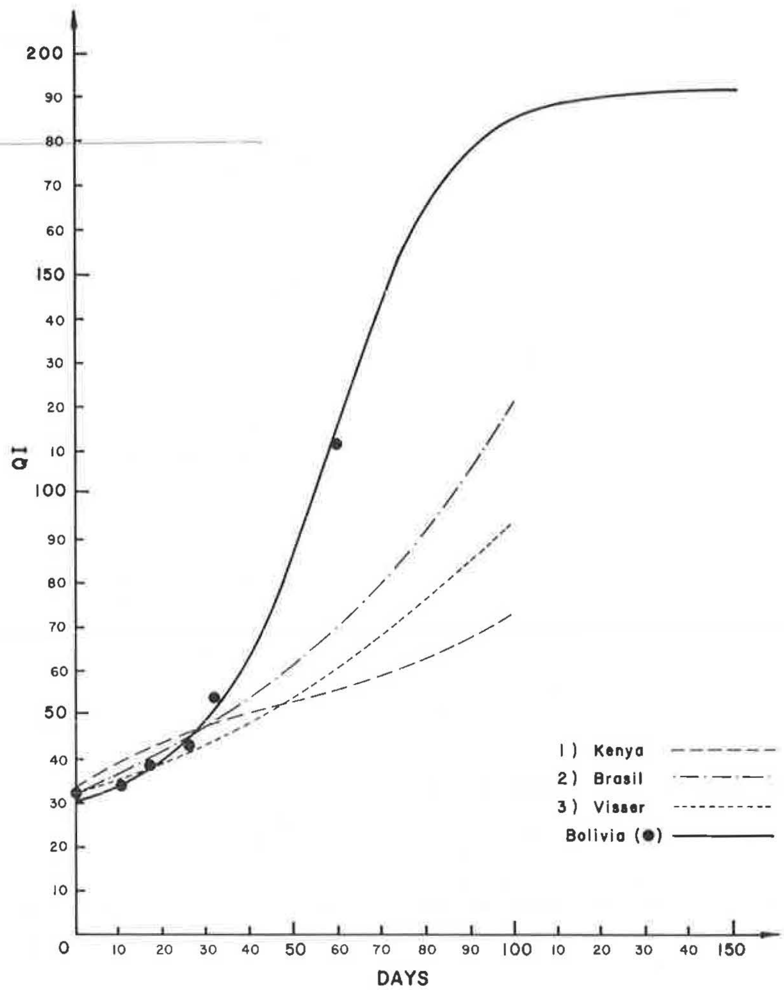

FIGURE 2 Performance curves for Rio Seco-Desaquadero Road.

NC = average daily car and pickup traffic in both directions;
NT = average daily bus and truck traffic in both directions;
G = absolute value of grade, as percentage;
S = seasonal variable where dry = 0, wet = 1;
SV = percentage of surfacing material passing the 0.074-mm sieve; and
PI = plasticity index of surfacing material, as a percentage.

The traffic averaged 672 AADT. This traffic level was used to establish the Kenya road performance prediction shown in Figure 2. The Kenya model predicts roughness in millimeters per kilometer. This was converted to QI units of roughness using an equation established in Brazil for this purpose:

$$QI = 0.0251\ BI^{0.93} \tag{4}$$

where QI is roughness unit and BI is roughness measured with a bump integrator (mm/km).

For the Brazil and Visser model, the distribution of car, bus, and truck traffic was based on the following established values:

Roughness after blading	QI = 32
Road width	8 m
Radius	1000 m
Average daily traffic	672
Light vehicles	141
Heavy vehicles	531
Plastic index	10%
SV (0.074 material)	10%
Grade	0%
Surface T2	0

Substituting these values into the Brazil and Visser equations produced the curves shown in Figure 2.

The measured deterioration on the Bolivian road, reflected by increasing roughness with time, was compared with predictions based on the three equations identified earlier (i.e., Kenya, Brazil, and Visser). The Bolivian roads became rougher more quickly than predicted using the existing equations. Therefore, a series of studies was made in Bolivia to collect road performance information for roads with different traffic levels.

Roughness Studies

Two different maintenance activities were practiced by the Bolivian National Highway Department (SNC). First, at intervals, motor graders were used to grade the road surface and improve road surface and riding conditions (grade). Second, major surface aggregate replacement and rehabilitation work was performed to rebuild the road surface (rehabilitate).

Aggregate roads with different traffic volumes were selected and sections were established where road roughness was monitored using a Maysmeter. The

Maysmeter was calibrated on a series of calibration sections where a rod and leveling procedure was used to establish a QI measure of roughness for each calibration section. An analysis of the roughness data generated by the Maysmeter produced the following equations:

Performance (roughness) of Road Following Grading

$$QI = 7.922 + 177 \left\{ \left[e^{(AxBxC - 0.78)/1.04} \right] \div \left[3 + e^{(AxBxC - 0.78)/1.04} \right] \right\} \quad (5)$$

where

A = 5.8,
B = D (0.0059 + 0.000011 AADT),
C = $(575/\text{AADT})^{0.4175}$,
D = days, and
AADT = average annual daily traffic.

Performance (roughness of road following rehabilitation)

$$QI = 29.189 + 185 \left\{ \left[e^{(AxBxC - 5.47)/2.135} \right] \div \left[5 + 3^{(AxBxC - 5.47)/2.135} \right] \right\} \quad (6)$$

where

A = 5.2,
B = D (0.00608 + 0.00001138 AADT),
C = $(575/\text{AADT})^{0.2682}$,
D = days, and
AADT = average annual daily traffic.

Equations 5 and 6 are plotted in Figures 3 and 4 for a range of traffic volumes.

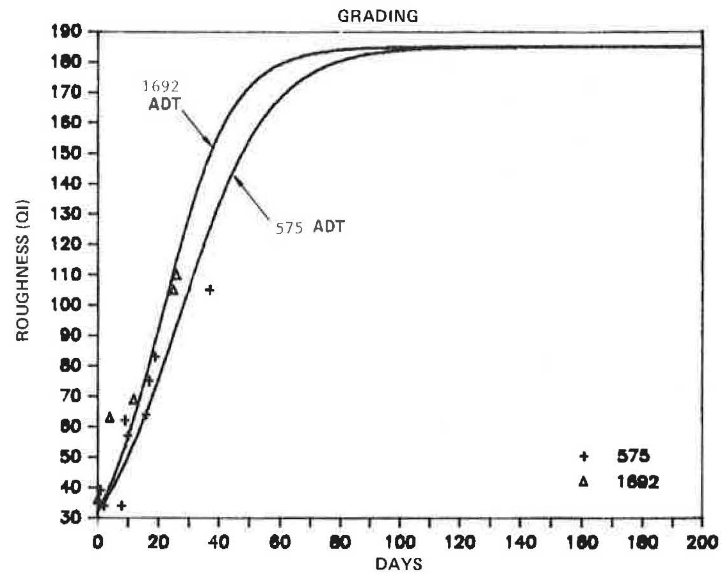

FIGURE 3 Roughness measurements following grading of a gravel road.

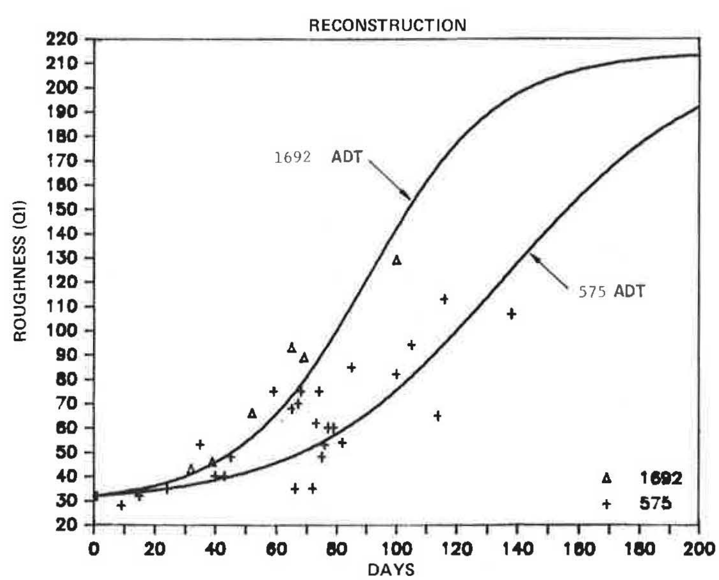

FIGURE 4 Roughness measurements following rehabilitation of a gravel road.

Maintenance Costs

The two aggregate road surface maintenance activities, grading and rehabilitation, were both specified in maintenance performance standards that describe the procedures followed in executing these activities. The standards also provide estimates of the production expected to be achieved and define the labor, equipment, and material requirements per unit of production.

Performance standards were used to calculate the costs of both aggregate surface activities for a base year period (1981). Information available from SNC fiscal records established a maintenance-related administrative charge of 31 percent. Direct maintenance costs were multipled by a factor of 1.31 to accommodate the administrative overhead.

User Costs

Literature studies were made in Bolivia to establish user equations that relate roadway characteristics to vehicle operating costs. Sources ranged from primary studies, such as Winfrey (11), Claffey (12), Bonney and Stevens (2), Hide et al. (1), GEIPOT (3), CRRI (4), De Weille (13), and Zaniewski et al. (14). In addition, users throughout the country were contacted to assess the level of vehicle operating costs they were experiencing on Bolivian roads. The information collected was used to verify existing relationships.

Fuel Consumption

These equations were derived from the Brazil study with adjustments made where necessary to the intercept so that predictions reflected vehicle use in Bolivia. Fuel consumption (FC) is expressed in units of liters/100 km, a unit conventionally used in cost studies.

$$\text{Light vehicle: } \ln(FC) = 5.078 + 0.00141\ QI \tag{7}$$

$$\text{Bus: } \ln(FC) = 5.675 + 0.00061\ QI \tag{8}$$

$$\text{Truck: } \ln(FC) = 5.887 + 0.00108\ QI \tag{9}$$

A range of predictions, for various levels of QI, follows:

QI	50	100	150	200
Light vehicle	172	185	198	213
Bus	300	310	319	329
Truck	380	401	424	447

The percentage increase in fuel consumption on moving from 50 QI to 200 QI is light vehicle, 24 percent; bus, 10 percent; and truck, 18 percent. Worsening surface condition causes a vehicle to alter its speed and these speed changes, together with less efficient engine speeds, cause fuel consumption to rise as the surface deteriorates. Light vehicles travel at higher speeds on good roads so the speed changes are greater. Trucks have heavier loads so the speed changes cause their engines to work harder than do those of buses.

Oil and Grease Consumption

These are small user cost items, often constituting less than 1 percent of total vehicle operating costs. No equations were included for these items.

Parts Consumption

This was based on Brazil equations that were calibrated using Bolivian data. The parts equations developed were:

$$\text{Light vehicle: } PC = [K^{0.302} \exp(6.497 + 0.00426\ QI)]\ 0.5 \tag{10}$$

$$\text{Bus: } PC = [K^{0.485} \exp(5.703 + 0.00323\ QI)]\ 0.5 \tag{11}$$

$$\text{Truck: } PC = (305 + 105.7\ QI)\ 0.5 \tag{12}$$

PC is parts cost per 1000 km in Bolivian pesos at December 1981 prices. K is the vehicle age in 1000-km units. A range of predictions for various levels of QI follows:

Class	Age (K)	QI 50	100	150	200
Light vehicle	150	1,866	2,309	2,853	3,530
Bus	350	3,018	3,547	4,169	4,900
Truck	-	2,795	5,438	8,080	10,723

The percentage increase in parts consumption on moving from 50 to 200 QI is light vehicle, 89 percent; bus, 62 percent; and truck, 384 percent.

Labor Costs

These were also based on equations developed in the Brazil study in which equations predicted labor costs as a function of parts cost and road roughness. The equations are

$$\text{Light vehicle: } LC = \{\exp[3.33 + 0.548 \ln(PC) + 0.00403\ QI]\}\ 0.5 \tag{13}$$

$$\text{Bus: } LC = \{\exp[3.231 + 0.516 \ln(PC) + 0.00514\ QI]\}\ 0.5 \tag{14}$$

$$\text{Truck: } LC = \{\exp[3.396 + 0.519 \ln(PC)]\}\ 0.5 \tag{15}$$

LC is labor cost in Bolivian pesos at December 1981 prices.

A range of predictions for various QI values follows and the PC variable is derived from the parts prediction table.

QI	50	100	150	200
Light vehicle	1,549	2,130	2,925	4,021
Bus	1,461	2,053	2,887	4,056
Truck	1,314	1,856	2,215	2,641

The coefficients on the ln (PC) variable for light vehicles and buses indicate that as parts cost increase, the proportion attributed to labor falls, as long as roughness is constant. When roughness increases, the labor costs for these two vehicle classes start to move upward until they equal or even exceed (at very high roughness) parts cost values.

Tire Consumption

These equations were based on Brazilian data on more than 3,000 tires lives. The dependent variable is tire life in 10 000 km.

Light vehicle:

$$TL = e^{(14.6488 - 0.9432 \ln QI)/10{,}000} \tag{16}$$

Bus: TL = 4.181 - 0.00951 QI (17)

Truck: TL = 3.933 - 0.00951 QI (18)

A range of predictions (in km) for various QI values follows:

QI	50	100	150	200
Light vehicle	57,468	29,888	20,389	15,544
Bus	37,045	32,300	27,545	22,790
Truck	34,575	29,320	25,065	20,310

Depreciation and Interest Costs

The depreciation charge attributable to the characteristics over which the vehicle operates together with the opportunity cost of the capital tied up in vehicle ownership is required in the analysis. Depreciation is based on lifetime kilometer use and the effect of highway characteristics on this life use (15). The effect of roughness and geometry on life utilization was estimated using Brazilian data. A basic vehicle life in kilometers is modified to reflect the reduced lifetime utilization resulting from operation on roads with inferior surfaces or poor geometry. Depreciable lifetime value (initial purchase price less tires and residual value) is divided by the adjusted lifetime kilometers to give a per kilometer depreciation cost.

The interest component requires the estimation of service life in years for each vehicle class. Brazilian data were examined and adjustments made to fit Bolivian conditions. The following values were used:

Depreciation

Lifetime kilometers (based on a QI of 50) are

Light vehicle	380 000
Bus	477 000
Truck	575 000

Lifetime

Lifetime in years and residual values are

Light vehicle	10 years	20 percent
Bus	12 years	12 percent
Truck	12 years	12 percent

Interest

Rate of interest was assumed to be 16 percent in real terms.

Calculations

Depreciation was made a function of lifetime utilization, and lifetime utilization was adjusted for roughness by reducing life mileage 0.1 percent per unit increase in QI. The depreciation and interest equation therefore was

$$D\&I = (.08 \cdot V \cdot 10 + V \cdot RV)/\theta \quad (19)$$

$$\theta = LM/[1+(QI/1{,}000)] \quad (20)$$

where

D&I = depreciation and interest cost per kilometer,
V = vehicle acquisition cost,
QI = roughness,
LM = life mileage for QI roughness level equal to zero, and
RV = fraction of acquisition cost.

An interest rate on the residual acquisition costs of 16 percent was built into the equation.

Unit Prices

Unit prices were calculated in financial terms at 1981 prices. Maintenance data were already in this form as a result of the unit costs tables developed for labor, equipment, and material used to expand the maintenance performance standards. The user inputs had to reflect a similar cost basis. These data had the added advantage of being rapidly obtainable from vehicle operators and dealers. The user data are

Item	Financial Cost ($ Bolivian)
Gasoline	8 per liter
Diesel	8 per liter
Car tire	2,675
Bus tire	7,750
Truck tire	8,500
Car price	375,000
Bus price	1,125,000
Truck price	800,000
Vehicle Age in Kilometers	
Car	80 000
Bus	300 000

ANALYSIS

The logit equation representing the performance of aggregate surface roads together with the equations relating road roughness to vehicle operating costs were incorporated in a small computer program (RSML). The program allows the unit costs of highway maintenance and vehicle operating consumables to be entered at the beginning of a program run. Output from the program RSML is given in Tables 1-3.

The general methodology built into RSML is as follows:

1. A road section is assumed to be just rehabilitated and its roughness following this activity is defined.
2. The roughness of the road for each succeeding day is predicted.
3. User costs associated with traffic using the road each day, for each roughness condition, are accumulated.
4. User costs are accumulated to some defined roughness threshold (i.e., a level of roughness that will activate a maintenance response).
5. The maintenance sequence specified was two gradings followed by rehabilitation. This made up a complete cycle.
6. Ten cycles are simulated for each roughness threshold and the combined user and maintenance costs for that roughness threshold are determined on an annual basis along with the average cycles per year.
7. The same analysis was performed for a range of roughness thresholds.

In addition, three classes of traffic were defined: light vehicles, buses, and trucks. The percentage of each is given in Tables 1-3. Also given is a factor for administration. The analysis can be run for any traffic volume level and given in Tables 1-3 are AADTs of 100, 300, and 500.

TABLE 1 Annual Agency and User Costs of a Maintenance Cycle Consisting of Rehabilitation of a Gravel Road Followed by Two Gradings for AADT = 100, by Roughness Threshold (maintenance level)

REHABILITATION AND GRADING

** ROAD SURFACE MAINTENANCE LEVELS **

MAX ROUGHNESS	MAINTENANCE COSTS	USER COSTS	CHANGE	TOTAL CHANGE	ANNUAL FREQUENCY	TOTAL COSTS	TOTAL ADJUSTED
35	888532.	355962.	0.	0.	35.3	1244494.	697927.
40	459075.	365177.	9215.	9215.	18.3	824252.	277685.
45	348665.	371111.	5934.	15149.	13.9	719776.	173208.
50	296177.	375325.	4214.	19363.	11.8	671502.	124935.
55	259854.	379352.	4027.	23390.	10.3	639205.	92638.
60	233428.	383207.	3856.	27246.	9.3	616635.	70068.
65	218607.	385957.	2750.	29995.	8.7	604565.	57997.
70	202533.	389355.	3397.	33393.	8.1	591888.	45320.
75	192619.	391896.	2541.	35934.	7.7	584515.	37947.
80	181214.	395145.	3249.	39183.	7.2	576358.	29791.
85	173236.	397833.	2688.	41871.	6.9	571069.	24501.
90	163955.	401289.	3457.	45327.	6.5	565245.	18678.
95	158302.	403692.	2403.	47730.	6.3	561994.	15427.
100	153025.	406138.	2446.	50176.	6.1	559163.	12596.
105	146513.	409393.	3254.	53431.	5.8	555906.	9339.
110	141982.	411925.	2532.	55963.	5.6	553907.	7340.
115	136359.	415300.	3375.	59338.	5.4	551659.	5092.
120	132425.	417901.	2601.	61939.	5.3	550327.	3759.
125	128114.	420918.	3017.	64956.	5.1	549032.	2464.
130	124636.	423561.	2643.	67599.	5.0	548196.	1629.
135	120282.	427078.	3517.	71116.	4.8	547360.	792.
140	116222.	430615.	3537.	74653.	4.6	546836.	269.
145	112427.	434158.	3543.	78196.	4.5	546585.	17.
150	108872.	437696.	3538.	81734.	4.3	546567.	0.
155	105132.	441660.	3965.	85698.	4.2	546792.	225.
160	101267.	446006.	4346.	90044.	4.0	547273.	706.
165	97330.	450718.	4712.	94756.	3.9	548048.	1481.
170	93056.	456148.	5430.	100186.	3.7	549204.	2637.

PERCENT BUSES	.48	AVERAGE DAILY TRAFFIC	100.
PERCENT TRUCKS	.31	COST OF GRADING	7351.
PERCENT AUTOS	.21	COST OF REHABILITATION	42904.
ADMINISTRATIVE OVERHEAD	.31		

TABLE 2 Annual Agency and User Costs of a Maintenance Cycle Consisting of Rehabilitation of a Gravel Road Followed by Two Gradings for AADT = 300, by Roughness Threshold (maintenance level)

REHABILITATION AND GRADING

** ROAD SURFACE MAINTENANCE LEVELS **

MAX ROUGHNESS	MAINTENANCE COSTS	USER COSTS	CHANGE	TOTAL CHANGE	ANNUAL FREQUENCY	TOTAL COSTS	TOTAL ADJUSTED
35	834682.	1064464.	0.	0.	33.2	1899146.	546122.
40	437214.	1090738.	26274.	26274.	17.4	1527952.	174929.
45	335909.	1107902.	17165.	43438.	13.4	1443811.	90788.
50	278227.	1122256.	14354.	57792.	11.1	1400483.	47460.
55	245933.	1134026.	11770.	69562.	9.8	1379959.	26935.
60	222133.	1145171.	11146.	80707.	8.8	1367304.	14281.
65	205556.	1154569.	9397.	90105.	8.2	1360125.	7101.
70	191281.	1164333.	9764.	99869.	7.6	1355614.	2591.
75	178860.	1174450.	10116.	109986.	7.1	1353311.	287.
80	171084.	1181940.	7489.	117476.	6.8	1353024.	0.
85	162026.	1191436.	9496.	126972.	6.4	1353463.	439.
90	154744.	1200030.	8594.	135566.	6.2	1354775.	1751.
95	149698.	1206805.	6775.	142341.	6.0	1356504.	3480.
100	142718.	1216972.	10167.	152508.	5.7	1359690.	6666.
105	137037.	1226131.	9159.	161667.	5.4	1363168.	10145.
110	131792.	1235466.	9335.	171002.	5.2	1367258.	14234.
115	128114.	1242680.	7215.	178216.	5.1	1370794.	17771.
120	123518.	1252238.	9557.	187774.	4.9	1375756.	22732.
125	119240.	1261907.	9669.	197443.	4.7	1381147.	28124.
130	115249.	1271656.	9750.	207192.	4.6	1386905.	33882.
135	111516.	1281463.	9806.	216999.	4.4	1392979.	39955.
140	108018.	1291299.	9837.	226835.	4.3	1399317.	46293.
145	104732.	1301139.	9840.	236675.	4.2	1405871.	52847.
150	100896.	1313350.	12211.	248886.	4.0	1414245.	61222.
155	97330.	1325496.	12146.	261032.	3.9	1422826.	69802.
160	93689.	1338722.	13226.	274258.	3.7	1432411.	79387.
165	89722.	1354049.	15327.	289585.	3.6	1443770.	90747.
170	86077.	1368990.	14941.	304526.	3.4	1455066.	102043.

PERCENT BUSES	.48	AVERAGE DAILY TRAFFIC	300.
PERCENT TRUCKS	.31	COST OF GRADING	7351.
PERCENT AUTOS	.21	COST OF REHABILITATION	42904.
ADMINISTRATIVE OVERHEAD	.31		

TABLE 3 Annual Agency and User Costs of a Maintenance Cycle Consisting of Rehabilitation of a Gravel Road Followed by Two Gradings for AADT = 500, by Roughness Threshold (maintenance level)

REHABILITATION AND GRADING
** ROAD SURFACE MAINTENANCE LEVELS **

MAX ROUGHNESS	MAINTENANCE COSTS	USER COSTS	CHANGE	TOTAL CHANGE	ANNUAL FREQUENCY	TOTAL COSTS	TOTAL ADJUSTED
35	888532.	1772346.	0.	0.	35.3	2660879.	521119.
40	466856.	1814425.	42078.	42078.	18.6	2281281.	141521.
45	357721.	1843030.	28606.	70684.	14.2	2200751.	60991.
50	296177.	1866939.	23909.	94592.	11.8	2163116.	23357.
55	259854.	1888025.	21087.	115679.	10.3	2147879.	8120.
60	235423.	1906078.	18053.	133731.	9.4	2141501.	1741.
65	216886.	1922874.	16796.	150528.	8.6	2139760.	0.
70	202533.	1938325.	15451.	165978.	8.1	2140858.	1098.
75	189962.	1954329.	16005.	181983.	7.6	2144291.	4532.
80	181214.	1967595.	13266.	195249.	7.2	2148809.	9049.
85	172153.	1982367.	14772.	210021.	6.8	2154520.	14761.
90	162985.	1999664.	17297.	227318.	6.5	2162649.	22890.
95	157397.	2011676.	12012.	239329.	6.3	2169073.	29313.
100	150516.	2027504.	15829.	255158.	6.0	2178021.	38261.
105	144971.	2041512.	14008.	269166.	5.8	2186484.	46724.
110	139114.	2057960.	16448.	285614.	5.5	2197074.	57315.
115	135022.	2070655.	12695.	298309.	5.4	2205678.	65918.
120	130543.	2085271.	14616.	312925.	5.2	2215814.	76054.
125	125774.	2102264.	16993.	329918.	5.0	2228038.	88278.
130	121341.	2119388.	17124.	347042.	4.8	2240730.	100970.
135	117712.	2134383.	14995.	362036.	4.7	2252094.	112335.
140	113820.	2151628.	17245.	379281.	4.5	2265448.	125688.
145	110178.	2168870.	17242.	396524.	4.4	2279048.	139288.
150	105940.	2190246.	21376.	417900.	4.2	2296186.	156427.
155	102017.	2211468.	21222.	439122.	4.1	2313485.	173725.
160	98373.	2232435.	20967.	460088.	3.9	2330808.	191048.
165	94009.	2259163.	26728.	486817.	3.7	2353171.	213412.
170	90015.	2285161.	25999.	512815.	3.6	2375176.	235417.

PERCENT BUSES	.48	AVERAGE DAILY TRAFFIC	500.
PERCENT TRUCKS	.31	COST OF GRADING	7351.
PERCENT AUTOS	.21	COST OF REHABILITATION	42904.
ADMINISTRATIVE OVERHEAD	.31		

Figure 5 shows the general simulation procedure for a roughness threshold of QI equal to 110.

Table 3 indicates that eight columns of information were generated for each run. Column 1, "Max Roughness," indicates the level of maintenance in terms of road surface roughness in QI units. The QI numbers 35 through 170 cover the range of roughness thresholds examined.

The column headed "Maintenance Costs" shows the annual road maintenance costs needed to keep the road from exceeding the roughness threshold specified. The column entitled " User Costs" is the annual cost to operate the indicated traffic for the threshold roughness level. The traffic is 500 vehicles per day with a composition of 21 percent light vehicles, 48 percent buses, and 31 percent trucks.

Column 4 is entitled "Change" and shows the user costs associated with moving from one maintenance

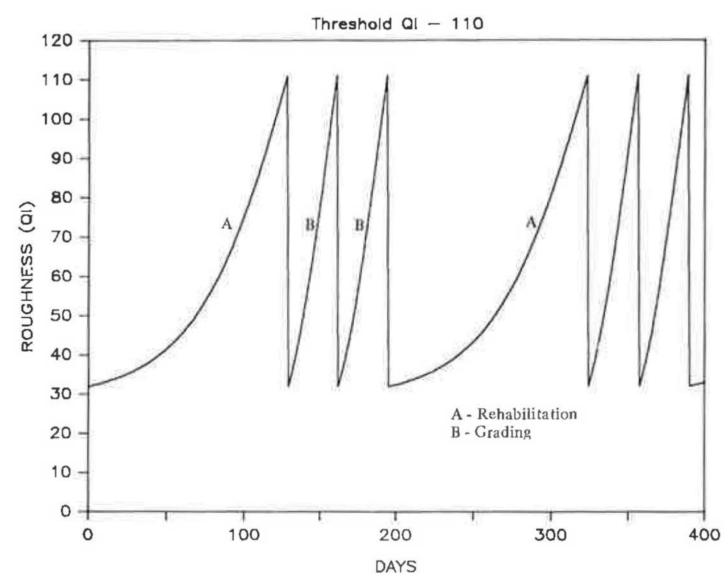

FIGURE 5 Gravel road maintenance activity sequence.

threshold to the next (i.e., allowing the maximum roughness to increase from a QI of 50 to 55 results in a user cost change of 21,087 pesos per year for the composite traffic). The column "Total Change" is the cumulative user costs change from the initial roughness following grading to any roughness threshold. The sixth column is the annual maintenance frequency needed to meet the indicated roughness threshold. The two gradings and one rehabilitation program analyzed reflects the average frequency of the combination (i.e., the 10-cycle frequency divided by 3). The "Total Costs" column is maintenance plus user costs. The last column is titled "Total Adjusted" and was obtained by screening the "Total Costs" column for the minimum total cost and then subtracting this cost from each total costs value. The minimum costs occurred at a roughness threshold of 65 so the value at this point in the "Total Adjusted" column is zero. This is the optimum maintenance level for the traffic composition given with a volume of 500 vehicles per day.

MAINTENANCE SERVICE LEVELS

The objective analysis presented was used to define maintenance service levels in Bolivia. Figure 6 shows the frequency of the grading and rehabilitation cycle studied as a function of traffic volume. The curve was used in a performance budgeting program to generate the annual frequency of grading and rehabilitating aggregate surface roads.

The program RSML was also used to analyze maintenance service levels for grading and rehabilitation separately. The resulting optimum frequencies are shown in Figure 7.

SUMMARY

A procedure has been presented for objectively establishing aggregate road maintenance frequencies in Bolivia. The procedure is based on minimizing total annual maintenance and vehicle operating costs.

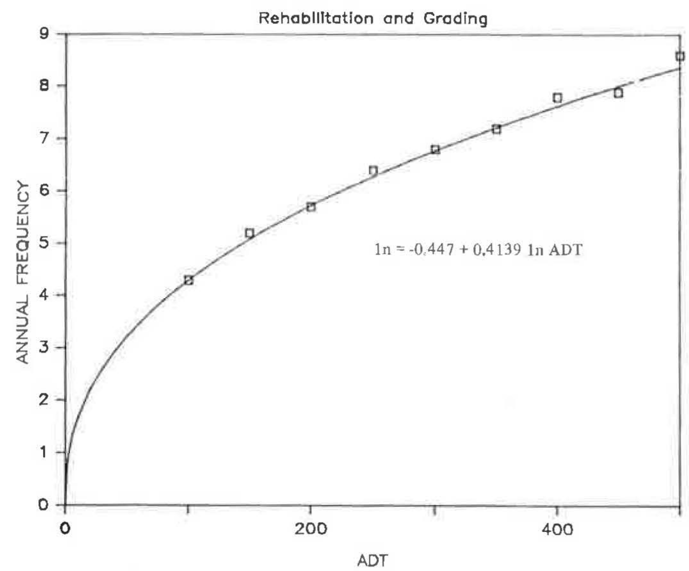

FIGURE 6 Annual frequency of grading and rehabilitation versus ADT.

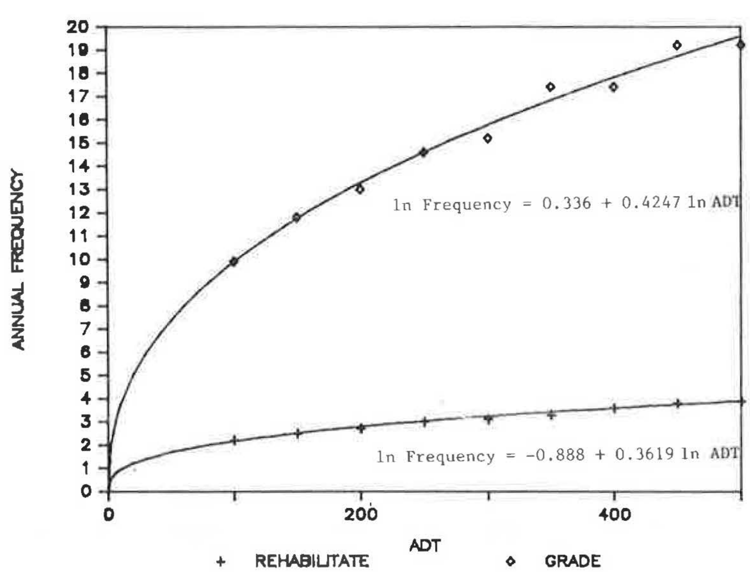

FIGURE 7 Optimum frequencies of grading and rehabilitation.

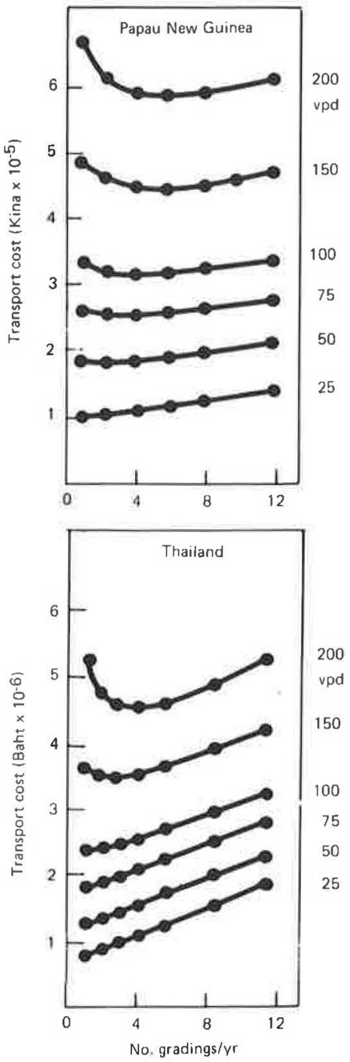

FIGURE 8 Transport cost versus frequency of grading for Papua New Guinea and Thailand.

It requires that road surface conditions (roughness) be predicted over time as a function of traffic. Also needed are equations to relate road roughness to vehicle operating costs. The procedure can be made applicable to any nonpaved road in any country given the appropriate road performance, user costs equations, and grading costs.

ACKNOWLEDGMENT

The República de Bolivia, Ministerio de Transportes, Comunicaciones y Aeronáutica Civil, Servicio Nacional de Caminos (SNC) sponsored the study that provided the basis for this paper. Employees of SNC collected Maysmeter roughness data that were used to develop gravel road performance equations. SNC employees also assisted in assembling the data base that was used to modify the Brazil relationships on roughness to vehicle operating costs that made them applicable to Bolivian driving conditions and vehicle use.

Discussion

Richard Robinson*

The work described in the paper mirrors studies carried out by the Transport and Road Research Laboratory Overseas Unit in conjunction with Crown Agents in Papua New Guinea and with John Burrow and Partners in Thailand. In both cases, field data were collected and used to determine optimum grading frequencies to minimize the sum of maintenance cost and road user cost for a traffic range of 25 to 200 vehicles per day. The results are plotted in Figure 8.

Figure 9 shows the results from Papua New Guinea and Thailand compared with those of Butler et al. from Bolivia. It is clear that differences in material types, climate, and unit costs in the three

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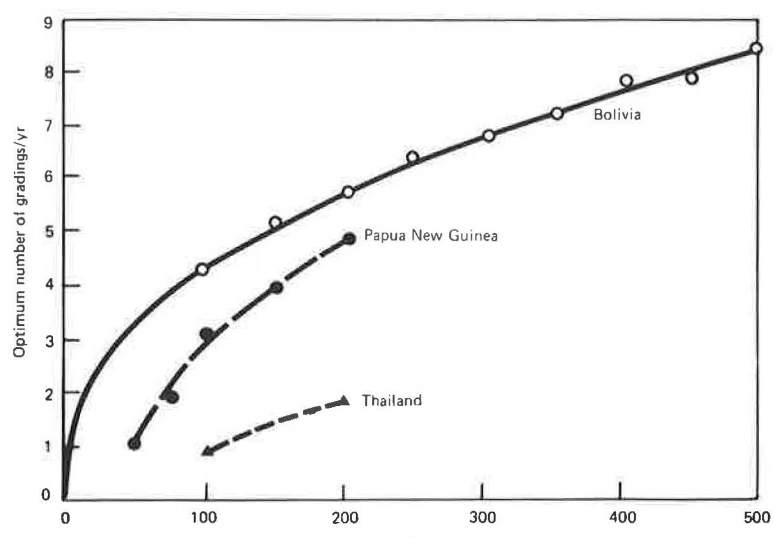

FIGURE 9 Optimum frequency of grading for Papua New Guinea and Thailand compared with that for Bolivia.

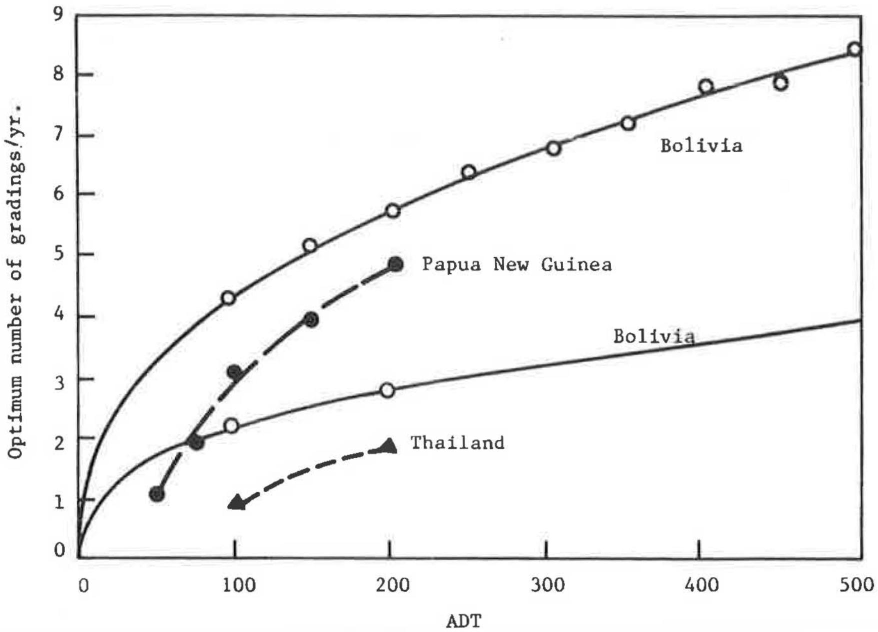

FIGURE 10 Optimum frequency of grading for Papua New Guinea and Thailand compared with that for Bolivia (modification of Figure 9).

countries have resulted in quite different recommendations being made about optimum grading frequencies for each case. This illustrates the danger of assuming that findings from one country will apply elsewhere in the world and emphasizes the need to carry out specific studies for the different conditions found in individual countries.

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Authors' Closure

In our paper equations predicting road roughness in Bolivia and those from Brazil and Kenya were compared. The performance equations were substantially different. This provided the impetus to conduct studies to determine performance equations for Bolivian conditions.

As Robinson suggested, different materials, climate, and unit costs do produce different results and, therefore, we agree with Robinson's note of caution, suggesting the danger of assuming that findings from one country can be applied elsewhere. In regard to Figure 9, if we select rehabilitation as the treatment, we get a curve that falls between his Papua New Guinea and Thailand curves (Figure 10).

Finally, our Bolivian curves reflect a minimum grading frequency of once a year.

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Optimal Rehabilitation Frequencies for Highway Pavements

MICHAEL J. MARKOW and WAYNE S. BALTA

ABSTRACT

The maintenance and rehabilitation of existing, mature facilities are becoming increasingly important components of highway activity. Yet, although the planning, budgeting, evaluation, and management of maintenance and rehabilitation are different from corresponding actions for new construction, comparatively little work has been devoted to the development of planning and management tools intended specifically for repair programs. For a number of reasons, the optimization of maintenance and rehabilitation policy is difficult, and new concepts and analytic approaches need to be formulated to address this problem. Recently, the usefulness of dynamic control theory for optimizing transport investment decisions has been demonstrated. Control theory structures a problem in terms of a dynamic (i.e., time varying) objective function (e.g., maximize total transport-related benefits over time) subject to dynamic constraints (e.g., equations describing changes in pavement condition due to deterioration and repair or variations in traffic levels responding to current pavement condition). The several factors that influence the problem are structured in terms of state variables (over which decision makers have no control, such as traffic, weather, and soil) and control variables (over which decision makers exercise judgment, such as maintenance and rehabilitation policy). Dynamic control theory thus presents an attractive analytical tool for management of highway infrastructure; it encompasses all the key variables of interest, allows technically correct engineering and economic relationships to be expressed in problem formulation, and leads directly and efficiently to solution of optimal maintenance and rehabilitation policy. The tenets of dynamic control theory are described, and a numerical example of the use of dynamic control theory to optimize the overlay frequency on highway pavements in the United States is given.

The maintenance and rehabilitation of existing, mature facilities are becoming increasingly important components of highway activity. However, comparatively little work has been devoted to the development of planning and management tools intended specifically for repair programs. Yet, decisions regarding the planning, budgeting, evaluation, and management of maintenance and rehabilitation are different from corresponding actions for new construction:

1. Planning and managing maintenance and rehabilitation programs require an understanding of concepts underlying facility performance, as opposed to facility design.
2. There is a need to understand the role of

maintenance and rehabilitation in influencing facility performance.

3. The planning of maintenance programs implies an ability to evaluate life-cycle performance and costs, with trade-offs measured in economic as well as technical terms.

4. Decisions to repair existing facilities are complicated by the wide range of activities possible (ranging from minor routine maintenance to major rehabilitation or reconstruction), problems in spatial and temporal allocation of resources throughout a network, and choices between investment and noninvestment policies (e.g., pavement strengthening versus adjustments in size and weight limits).

5. For those facilities that do not fail catastrophically (e.g., highway pavements, rail track), it is difficult to define the point of failure, which, in turn, complicates the specification of standards governing facility performance, safety, and cost.

As a result of these characteristics, the optimization of maintenance and rehabilitation policy is difficult, and new concepts and analytic approaches need to be introduced to address this problem. A first step in this direction was taken by Fernandez (1), who demonstrated the applicability of dynamic control theory to transportation investments. Fernandez used this approach to solve for several general case studies in transportation, including stage construction and routine maintenance, but not rehabilitation.

From an engineering point of view, rehabilitation is a major concern of highway administrators and a key element of pavement 3R programs across the nation. From an analytic point of view, however, rehabilitation poses the problem of discontinuities in the pavement condition history, as shown in Figure 1. These discontinuities arise through cycles of deterioration and major repair or renewal and signal the expenditure of significant sums of money. These discontinuities complicate the representation of facility performance and costs over time, as well as the determination of the optimal time to rehabilitate.

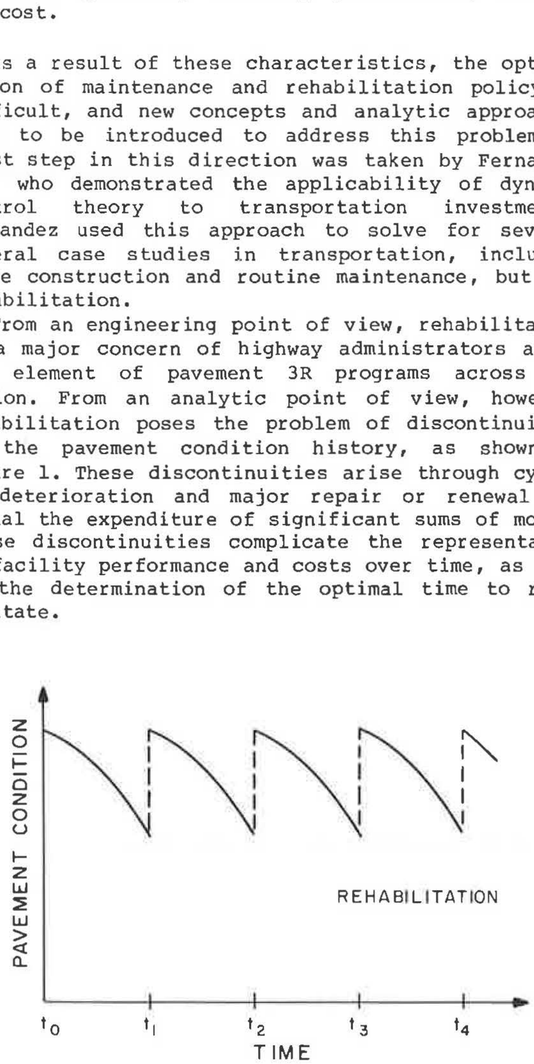

FIGURE 1 Effect of rehabilitation on pavement deterioration.

Recently, the necessary assumptions and mathematical solutions needed to address rehabilitation were successfully formulated by Balta (2). Balta developed his model, based on principles of control theory for discontinuities in state variables and system equations at interior points, to solve for the optimal timing of rehabilitation of a highway pavement. It is this solution, applied to pavement overlays, which is described in this paper.

The control theory approach enjoys several distinct advantages:

- It provides a unified conceptual and methodological framework for addressing rehabilitation policy;
- It features a closed-form optimization procedure;
- It encompasses all of the relevant variables pertaining to the demand-responsive approach to maintenance, including an interaction between demand for use and quality of pavement; and
- Although it has been developed as a policy model based in part on elements of economic and utility theory, it is intuitively appealing from an engineering point of view.

Although the mathematics of control theory may appear intimidating, the derived solution is elegant and leads to a surprisingly practical set of curves for optimal rehabilitation intervals that can be easily applied in the office or in the field. The formulation of the control theory model will be described first. Then, application of the model to the optimal timing of rehabilitation, using pavement overlays as examples, will be illustrated.

MODEL FORMULATION

Problem Description

The mathematical framework of optimal control theory can be used to model any number of stages between discontinuities for any length of time desired. For simplicity, however, the rehabilitation investment decision model developed in this paper will address only two such stages: before and after an investment is made. Given that the investment occurs at time t^*, Stage 1 then occurs between the beginning of the analysis period (time t_0) and the moment just before the investment (time t^{*-}). Similarly, Stage 2 occurs between the moment just after the investment (time t^{*+}) and the end of the planning horizon (time T). It is nonetheless clear that the model can be generalized for any number of investments. Figure 2 shows a summary of the pavement behavior that is analyzed in this model.

Like other optimization procedures, an optimal control model is specified with system variables, an objective function, and constraints. The state variables describing the system are defined by ordinary differential equations; these equations describe the state of the system at any time t. In this paper the system described is a highway pavement. The variables that characterize the system are

- $S(t) \equiv$ quality or condition of the pavement and
- $q(t) \equiv$ demand or traffic volume on the pavement.

Pavement condition, or S(t), is measured in the model in terms of the present serviceability index (PSI) as defined by AASHTO (3). Traffic demand, or q(t), is measured in the model in terms of 18-kip equivalent single axle loads.

Pavement Condition

Mathematically, pavement condition, S(t), is continuous during each stage between investments but it is discontinuous at the time of the investment. Conse-

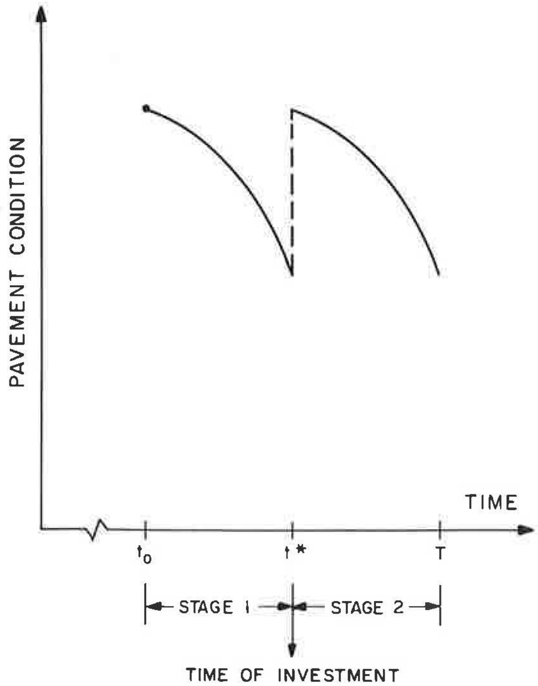

FIGURE 2 Pavement behavior analyzed in rehabilitation model.

quently the change in pavement condition over time, $\dot{S}(t)$ (where the dot denotes a time derivative), is also continuous during each stage between investments but discontinuous at the time of the investment. For the rehabilitation investment decision model it is assumed that $\dot{S}(t)$ decays from its initial value in a nonlinear fashion as shown in Figure 3. $S(t)$ will behave similarly. (Note that either concave or convex relationships may be used to represent different mechanisms of pavement damage.) It is further assumed that $S(t)$ and $\dot{S}(t)$ depend explicitly on time. Appropriate equations that capture this behavior are

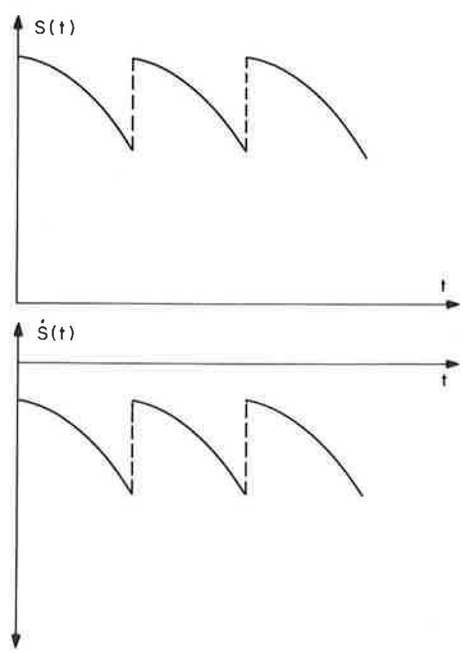

FIGURE 3 Pavement deterioration functions for rehabilitation model.

$$\dot{S}_1(t) = -\beta_1 k_1 e^{\beta_1 t} \qquad t\epsilon(0,t^{*-}) \tag{1}$$

and

$$\dot{S}_2(t) = -\beta_2 k_2 e^{\beta_2 (t-t^*)} \qquad t\epsilon(t^{*+},T) \tag{2}$$

where the subscripts 1 and 2 refer to stages one and two, respectively, and t = 0 corresponds to the beginning of the analysis period t_0. $S(t)$ is equal to the integral of $\dot{S}(t)$ and can be expressed as

$$S_1(t) = k_1 + S_0 - k_1 e^{\beta_1 t} \qquad t\epsilon(0,t^{*-}) \tag{3}$$

and

$$S_2(t) = k_2 + S_A - k_2 e^{\beta_2 (t-t^*)} \qquad t\epsilon(t^{*+},T) \tag{4}$$

In these equations, β_1 and β_2 are parameters that control how quickly pavement condition deteriorates; as β becomes larger, the condition decays at a faster pace. Values for β depend on several factors including pavement design, quality of construction, traffic volume and composition, weather, soil and drainage conditions, and extent of routine maintenance performed. It can be argued that $S(t)$ should depend explicitly on each of these factors. For mathematical simplicity in the development of this model, however, all of these factors are assumed to be implicit in a given value of β. Under these circumstances, these factors should be accounted for when formulating pavement damage as a time-dependent function. The benefit of mathematical simplicity will become evident as the model unfolds. Increased analytical complexity is really the only price to be paid for a more explicit approach.

The terms k_1, k_2, S_0, and S_A in Equations 1-4 are constants. S_0 and S_A represent the initial level of pavement condition and the level of pavement condition immediately following rehabilitation, respectively. (It is assumed that $S_A \leq S_0$.) The constants k_1 and k_2 interact with β_1 and β_2 to further affect the extent to which the deterioration function deviates from a linear equation. Note that if t = 0 in Equation 3, $S_1(0) = S_0$. If $t = t^*$ in Equation 4, $S_2(t^*) = S_A$. Because they are all constants, k_1 and S_0 as well as k_2 and S_A could be combined into one value. The reason for expressing them separately is to preserve the identity of S_0 and S_A along with the interaction between β_1 and k_1 or β_2 and k_2.

This discrimination permits straightforward sensitivity analyses. For example, it may be thought that a pavement deteriorates very little over t years and then suddenly decays very quickly. Alternatively, it may be thought that a pavement deteriorates at a more constant pace over the same time period. The functional form for $S(t)$ in this model is easily adaptable to different combinations of k and β.

Traffic

The other variable characterizing the system of a highway pavement is $q(t)$, traffic demand. Fernandez (1) asserted that it is important to account for an interaction between the quality of a pavement and the demand for its use. Mathematically this relationship is represented by a differential equation of the form:

$$\dot{q}(t) = a(t)\,S(t) + b(t) \qquad (5)$$

meaning that demand changes over time due to the condition of the pavement as well as to some external growth factor. In the first term, a(t) is a function that indicates how the number of road users changes due to a unit change in pavement quality. The function b(t) represents that part of the traffic growth rate that is independent of pavement quality. For mathematical simplicity, once again, the functions a(t) and b(t) are assumed to be constants in the development of this model.

Objective Function

The next step is to establish the objective function. The objective of the rehabilitation investment decision model is to maximize the present value of the net benefits derived from operation of the road over some planning period. Mathematically this is stated as follows:

$$\text{Maximize } J = \int_0^{t^{*-}} [U(t) - C(S,q)]\; q(t) \exp(-\rho t)$$

$$- I[S(t^{*-}), S(t^{*+}), t^*] \exp(-\rho t^*)$$

$$+ \int_{t^{*+}}^{T} [U(t) - C(S,q)]\; q(t) \exp(-\rho t) + \psi\,[q(T), T] \qquad (6)$$

U(t) is the benefit that each user perceives from using the road at time t. C(S,q) is the cost that each user experiences from using the road. It is assumed that user costs depend on the quality of the pavement and the number of road users. Because user utility and user cost are specified on a per user basis, multiplying their difference by the total number of users q(t) yields the total private net benefit obtained at any time. The term $\exp(-\rho t)$ is a discount factor that converts all values to present value; ρ is the appropriate discount rate. Integrating from 0 to t^{*-} represents a summation of the net benefits during Stage 1.

The term $I[S(t^{*-}), S(t^{*+}), t^*]$ represents the cost of the investment. The magnitude of this investment may be a function of many different factors. Here it is shown as a function of the pavement condition just before the investment, the desired new pavement condition following the investment, and the time of investment itself. Having $I \sim f[S(t^{*+})]$ is particularly relevant for the case of reconstruction where the investment would depend on the desired reconstructed quality of the pavement. Having $I \sim f(t^*)$ makes it possible to account for any real price changes in the cost of rehabilitation. For the case of overlays, it is reasonable and sufficient to use $I \sim f[S(t^{*-})]$. Existing condition affects overlay cost at least in accordance with required pavement preparation, which depends on the extent of crack sealing, patching, and localized pavement repair that is needed before the new wearing course is placed. Furthermore, because pavement condition is represented by AASHTO's present serviceability index in this model (where the rehabilitation is an overlay), the surface condition will be essentially the same following most overlays regardless of cost. The model is therefore explicitly developed using $I[S(t^{*-})]$ as the functional form for investment cost.

The third term of the objective function, $\int_{t^{*+}}^{T}$ $[U(t) - c(S,q)]\; q(t) \exp(-\rho t)$, is identical to the first except that it represents a summation of the net benefits during Stage 2. The final term, $\psi[q(T),T]$, represents the salvage value of the pavement at the end of its useful life. This form implies that salvage value of the pavement depends in some manner on the number of users at the terminal time T. Later in this paper it is specifically assumed that ψ depends on an infinite stream of user benefits beginning at T. This assumption is largely for convenience. Highway planners may justly assert that salvage value should depend on terminal pavement condition. The only consequence of incorporating this consideration into the model would be increased mathematical complexity.

There are two additional points to make about the objective function. First, it is predicated on the concept of maximizing net benefits. (The model could also be reformulated with the objective of minimizing net costs.) Second, the objective function rests on microeconomic principles of maximizing the net social benefits associated with a public facility. This is highlighted by the inclusion of user costs, which play a significant role in shaping the model's investment policy.

Solution

The problem to be solved may be summarized as follows: to determine the optimal time of rehabilitation, t*, that maximizes the objective function, Equation 6, subject to the technical constraints represented by Equations 1, 2, and 5. The solution to this problem involves computing the Hamiltonian function. The relationship of the Hamiltonian to dynamic control models is analogous to the relationship of the Lagrangian to static optimization models and involves the specification of adjoint variables analogous to Lagrangian multipliers. The details of the solution become quite involved, and are explained fully elsewhere (2). Moreover, Balta has developed individual solutions for different assumptions of traffic and investment cost (i.e., whether these are constant or variable over time). The focus in this paper will be on the most general solution obtained, considering variable traffic (i.e., allowing for nonzero rates of traffic growth and decline) and variable investment cost (i.e., implying a relationship between overlay thickness or cost and time-varying pavement condition).

The solution is expressed in the form of a decision rule, evaluated at t*, balancing marginal costs and marginal benefits. To obtain an explicit rule, however, it is necessary to specify relationships governing pavement deterioration over time, user costs (as functions of both traffic volume and pavement condition), traffic growth over time, rehabilitation cost over time (as a function of pavement condition when overlaid), and salvage value. Some of these relationships have already been defined in the problem description: pavement deterioration is characterized by Equations 1 and 2 or 3 and 4, and traffic growth is governed by Equation 5. Following are explanations of the remaining relationships used in the solution.

User Costs

This model considers two components of user costs:

- Vehicle operating costs and
- Travel time costs.

These components are both based on the research reported in the EAROMAR-2 simulation program (4). Vehicle operating costs depend on fuel, oil, and tire consumption. Travel time costs depend on the user's trip purpose and income level.

The initial form of the user cost function tested in the model was

$$C[S(t),q(t)] = c - \epsilon S(t) + \delta q(t)^n \qquad (7)$$

The first term of the user cost function, c, represents the user cost associated with low levels of traffic volume at a pavement condition of zero. The second term, $-\epsilon S(t)$, shows user cost decreasing linearly by ϵ for a unit increase in pavement quality. The third term, $\delta q(t)^n$, implies that the rise in user cost due to increased traffic (congestion) is a power function.

After applying this model in some test runs, however, it was noted that some of the results were unrealistic in light of normal practices of transportation and highway departments: rehabilitation was occurring too early in the pavement's life, after only a small decline in S(t). It was concluded that the most likely element causing these results was the assumed linear relationship between user costs and pavement surface condition and the magnitude of its slope, as shown in Figure 4. These values were determined using successive EAROMAR-2 simulations. What makes this relationship unrealistic is the magnitude of the increased cost experienced for a unit decline in PSI within the range of PSIs normally encountered on pavements in service (from approximately 4.5 to 2.0 PSI). As a result, the potential savings in user costs is so great that this marginal benefit will equal the marginal cost of undertaking the investment at a very slight degradation in surface condition.

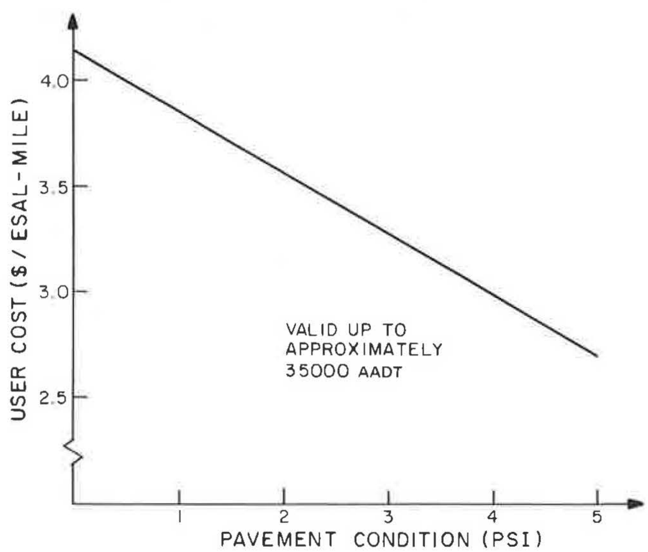

FIGURE 4 Linear C(S) relationship used to obtain initial results.

However, recent research demonstrates that the change in user cost (specifically in vehicle operating cost) experienced due to a unit change in PSI within the range of PSIs normally encountered on pavements in the United States is not much at all and is, indeed, significantly lower than the values shown in Figure 4 (5,6). Moreover, Ross (6) asserts that the true relationship between user costs and pavement surface condition is nonlinear.

The optimal rehabilitation investment decision model therefore employs a revised, nonlinear C versus S relationship as shown in Figure 5. This curve is expressed by the following natural antilogarithmic function:

$$C(S) = \mu C_0 e^{-\omega S(t)} + (1-\mu) C_o \qquad (8)$$

where

C_0 = user cost for travel on the worst possible pavement ($S \rightarrow 0_{PSI}$) without traffic congestion and

ω,μ = parameters controlling the shape of the function.

The parameter μ plays a role similar to that of k in the pavement deterioration functions, Equations 3 and 4. Note that when S = 0, C(S) = C_0. With this function it is possible to model user costs as rising at a slow, roughly linear pace between about 5 PSI and 2 PSI and then rising rapidly thereafter.

Rehabilitation Cost

In this section a relationship for I as a function of S^{*-} is developed. The approach used combines principles from the AASHTO Interim Guide for Design of Pavement Structures (3) along with a hypothesized relationship between the pavement's surface layer coefficient and the present serviceability index. It should be understood from the outset that the specific function for $I(S^{*-})$ developed herein is primarily a proposal or suggestion and should not be construed as representing the definitive relationship between overlay cost and current pavement condition. It is intended more as an example used to illustrate the broad capabilities of this model.

Although overlays have been chosen to illustrate the mathematical formulation of the model and its application to example solutions, other classes of rehabilitation (e.g., recycling, major sublayer stabilization, surface restoration) could also be treated. Different types of rehabilitation would be represented in the model through the particular form of the pavement deterioration function assigned to Stage 2 in Figure 2, the values assigned to the parameters of this function, and the unit costs input.

Two rehabilitation cases are developed as examples in this paper: flexible overlays of flexible pavements and flexible overlays of rigid pavements. Because the functions relating design procedures (and hence investment cost) to pavement condition are somewhat different for these cases, the appropriate equations will be developed for each case separately. The general methodology is to determine an overlay thickness based on structural condition and future traffic predictions and then to determine cost from the required overlay thickness.

FLEXIBLE OVERLAYS OF FLEXIBLE PAVEMENTS

The first task is to find the overlay thickness that is needed to restore the pavement's structural number to its original design value in the as-constructed state. Assuming that the flexible pavement consists of a subbase, a base, and a surface layer, the overlaid pavement will have four layers: subbase, base, original surface, and overlaid surface. Equating the structural number of the original pavement to that of the newly overlaid pavement:

$$a_1 D_1 + a_2 D_2 + a_3 D_3 = a_1 D_1 + a_2 D_2 + a'_3 D_3 + a_4 D_4 \qquad (9)$$

where

a_1, a_2, a_3, a_4 = layer coefficient for subbase, base, original surface, and overlaid surface layers, respectively;

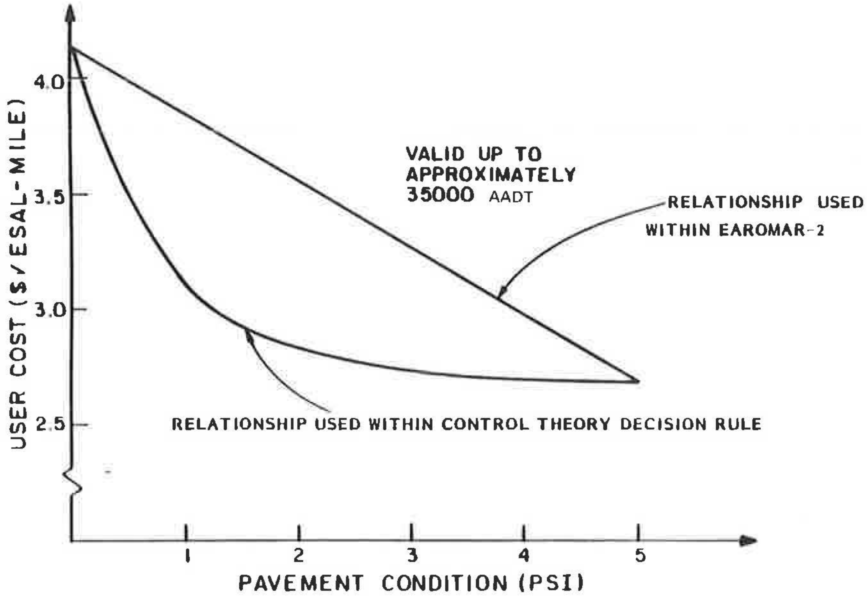

FIGURE 5 Comparison of nonlinear C(S) function to linear C(S) function.

D_1, D_2, D_3, D_4 = thickness of subbase, base, original surface, and overlaid surface layers, respectively; and

a_3' = decayed value of layer coefficient corresponding to current condition of original surface layer.

Equation 9 implies that for design purposes the strength of the existing pavement due to be overlaid is reduced from its as-constructed value. In accordance with AASHTO (3,7) this is represented by lowering the value of the original surface layer coefficient from a_3 to a_3'. Assuming that the layer coefficients and thicknesses of the subbase and base do not change over time, Equation 9 yields

$$D_4 = [(a_3 - a_3')/a_4]\, D_3 \tag{10}$$

Recognizing that the layer coefficient of the overlay usually equals the layer coefficient of the original surface layers, Equation 10 becomes

$$D_4 = D_3\, [1 - (a_3'/a_3)] \tag{11}$$

The fact that the surface layer coefficient of a pavement due to be overlaid is reduced from its as-constructed value can be used to establish a relationship between the surface layer coefficient and the PSI. It is hypothesized that the surface layer coefficient experiences an exponential decay during pavement life. This assumption is similar to the research reported for the EAROMAR-2 program (4). Mathematically,

$$a_3' = a_3 e^{\lambda[S_1(t) - S_0]} \tag{12}$$

Combining Equations 11 and 12 with Equation 3 results in

$$D_4^S = D_3 \left[1 - e^{\lambda k_1 (1 - e^{\beta_1 t^*})}\right] \tag{13}$$

where D_4^S represents the overlay thickness needed to restore the pavement's structural number to its original design value.

The overlay thickness specified by Equation 13, however, does not account for the additional traffic loading beyond the original design level that the pavement might experience during the next t years. This additional thickness, to be labeled D_4^Q, can be accounted for by taking the difference between the structural number as specified by AASHTO for the design of a flexible pavement (with t* as the beginning of the design period) and the structural number specified is Equation 9 and then dividing this difference by the overlay surface coefficient. The resulting differential thickness due to traffic growth is

$$D_4^Q = (1/a_4)\, [SN_{TOT} - a_1 D_1 - a_2 D_2 - a_4 D_3] \tag{14}$$

SN_{TOT} can be determined using the AASHTO design procedure for flexible pavement structures (3). This procedure provides a relationship between weighted structural number and total equivalent 18-kip single axle load applications (the model's measure of traffic volume). The relationship between axle loads over a 20-year life, Q_{20}, and structural number, SN, can be approximated as follows (2):

$$SN = (Q_{20}/310)^{1/6} \tag{15}$$

with $R^2 = 0.99$.

The value of Q_{20} can be obtained by integrating Equation 5 from t* to some time τ:

$$Q_\tau = \int_{t^*}^{t^*+\tau} [A_2 t - B_2 e^{\beta_2 (t - t^*)} + C_2]\, dt \tag{16}$$

where (for brevity) $A_i \equiv q_0\,(ak_i + aS_A + b)$, $B_i \equiv q_0\, ak_i/\beta_i$, and $C_2 \equiv A_1 t^* - B_1 \exp\,(\beta_1 t^*) + C_1 + B_2 - A_2 t^*$. The resulting solution is

$$Q_\tau = \tau\left\{A_2[t^* + (1/2)\tau] + C_2\right\} - (B_2/\beta_2)(e^{\tau\beta_2} - 1) \tag{17}$$

where τ can be set equal to 20 years (or any other design life). The required thickness for a flexible overlay of a flexible pavement may be written as

$$D_4 = D_3 \left[1 - e^{\lambda k_1 \left(1 - e^{\beta_1 t^*}\right)}\right] + (1/a_4) \left\{[(Q_\tau/310)^{1/6}] - a_1 D_1 - a_2 D_2 - a_4 D_3\right\} \tag{18}$$

where Q_τ is given by Equation 17; the first term of Equation 18 represents D_4^S and the second term represents D_4^Q. Now the cost of the overlay can be found by multiplying the thickness by a factor of cost per unit of thickness:

$$I = \sigma_F(D_4) \tag{19}$$

As a final note, it is possible for Equation 14 to give a negative value for D_4^Q if the original pavement design had a higher structural number than that required by the AASHTO design method applied at t*. (Such a case might arise, for example, if the original pavement design procedure differed from that of AASHTO.) In this case D_4^Q should simply be set equal to zero. Moreover, the overlay thickness determined by Equation 18 will be subject to a minimum thickness constraint required by construction procedures.

FLEXIBLE OVERLAYS OF RIGID PAVEMENTS

A rigid pavement given a flexible overlay becomes a composite pavement. This analysis treats a composite pavement as a flexible pavement with a relatively strong base (the former rigid surface layer). In this case, however, a single overlay thickness, which accounts for both restoring the pavement to its original strength and allowing for future traffic loadings, is determined.

For the composite pavement,

$$SN_{TOT} = a_s D_s + a'_R D_R + a_F(D_0) \tag{20}$$

where

a_s, a'_r, a_F = layer coefficient for subbase, rigid slab, and flexible overlay, respectively;
D_s, D_R = thickness of subbase and rigid slab, respectively; and
D_0 = required flexible overlay thickness.

In accordance with AASHTO (7) the layer coefficient of the rigid slab about to be overlaid is reduced from its original value. As in the previous section, this can be used to hypothesize a relationship between the layer coefficient and surface condition:

$$a'_R = a_R\, e^{\lambda[S_1(t) - S_0]} \tag{21}$$

where in general λ for flexible overlays of rigid pavements will not equal λ for flexible overlays of flexible pavements. In addition, Equation 15 can be used to determine the required structural number from the AASHTO procedure. Therefore, substituting Equations 15 and 21 along with 3 into Equation 20 and solving for D_0 yields

$$D_0 = (1/a_F)\left\{[(Q_\tau/310)^{1/6}] - a_s D_s - a_R D_R e^{\lambda k_1 (1-e^{\beta_1 t^*})}\right\} \tag{22}$$

where Q_τ is given by Equation 18.

The cost of the overlay can be found by multiplying the thickness by a cost factor (σ_R) as defined in the previous section. Once again, Equation 22 may produce a negative value if the original rigid pavement was overdesigned compared to AASHTO criteria. A minimum overlay thickness may be used to override this event.

Salvage Value

The salvage value in this model is assumed to depend on an infinite stream of user benefits beginning at T. [This assumption was originally made by Fernandez (1).] Mathematically this can be expressed as

$$\psi(T) = \int_T^\infty -G_2(t)q(T)\exp(-\rho t)dt \tag{23}$$

where $-G_2(t)$ is defined as $\{U-[C(S,q) + (\partial C/\partial q)\, q_2(t)]\}$ and represents net benefits per user during Stage 2. The term q(T) represents the number of users at terminal time T; for simplicity it is assumed that it becomes constant at this time. Multiplying the infinite stream of per user net benefits by the traffic volume at T yields the infinite stream of total user net benefits.

Resulting Decision Rule

The complete decision rules for flexible and for rigid pavements are as follows:

For Flexible Overlays of Flexible Pavements

$$\begin{aligned}\rho\sigma(D_4) = {} & \mu C_0 \left[e^{-\omega(k_1 + S_0 - k_1 e^{\beta_1 t^*})} - e^{-\omega S_A}\right] x\, (A_1 t^* - B_1 e^{\beta_1 t^*} + C_1) \\ & + \left[S_A - S_0 + k_1 (e^{\beta_1 t^*} - 1)\right] \pi(t^*) a q_0 \\ & + \sigma D_3 \lambda \beta_1 k_1 e^{\lambda k_1 (1-e^{\beta_1 t^*}) + \beta_1 t^*}\end{aligned} \tag{24}$$

where $\pi(t^*)$ is given by Equation 26 and D_4 is given by Equation 18.

For Flexible Overlays of Rigid Pavements

$$\begin{aligned}\rho\sigma(D_0) = {} & \mu C_0 \left[e^{-\omega\left(k_1 + S_0 - k_1 e^{\beta_1 t^*}\right)} - e^{-\omega S_A}\right] x\, (A_1 t^* - B_1 e^{\beta_1 t^*} + C_1) \\ & + \left[S_A - S_0 + k_1 (e^{\beta_1 t^*} - 1)\right] \pi(t^*) a q_0 \\ & + (a_R/a_F)\, \sigma D_R \lambda \beta_1 k_1 e^{\lambda k_1 (1-e^{\beta_1 t^*}) + \beta_1 t^*}\end{aligned} \tag{25}$$

where $\pi(t^*)$ is given by Equation 26 and D_0 is given by Equation 22.

$$\begin{aligned}\pi(t^*) = (U/\rho) - \Big\{ & -[\mu C_0/(\beta_2 \omega k_2 - \rho)]e^{-\omega S_A} + [(1-\mu)C_0/\rho] \\ & + (n+1)\delta[1/\rho\, (A_2 t^* + C_2)^n] + (nA_2/\rho^2)(A_2 t^* + C_2)^{n-1} \\ & + [n(n-1)A_2^2/\rho^3]\,(A_2 t^* + C_2)^{n-2} \\ & + [n(n-1)(n-2)A_2^3/\rho^4]\,(A_2 t^* + C_2)^{n-3} \\ & + [n(n-1)(n-2)(n-3)A_2^4/\rho^5]\,(A_2 t^* + C_2)^{n-4} \\ & + [n(n-1)(n-2)(n-3)(n-4)A_2^5/\rho^6]\,(A_2 t^* + C_2)^{n-5} + (n!A_2^6/\rho^7) \Big\}\end{aligned} \tag{26}$$

Although Equations 24 and 25 appear complex, their interpretation yields some intuitive insights into the structure of the solution.

The rules are marginal rules, balancing marginal costs (on the left side of the equations) and marginal benefits (on the right side of the equations). The term on the left denote the capitalized costs of undertaking the rehabilitation investment: in this case, the overlay of either flexible or rigid pavement. The first term on the right side of Equations 24 and 25, respectively, denotes the benefits of the rehabilitation accruing to the traffic stream, resulting from reductions in user costs due to the improved quality of the pavement surface. The second term in each equation quantifies the benefits of attracting additional traffic (and thereby providing the advantages of transportation to more users) because of the improved quality of the pavement. (Of course, additional traffic also causes increased rates of pavement deterioration and of congestion; these effects can be captured in the deterioration and user cost equations discussed earlier. Also note that if the variable "a" defined in Equation 5 is zero, this "generated traffic" effect is eliminated.) The third term in each equation captures the monetary benefit associated with preservation of investment (i.e., if rehabilitation is performed earlier, more substantial rehabilitation is avoided later). This term, in effect, justifies the avoidance of "deferred maintenance."

APPLICATIONS TO EXAMPLES

General Information

The decision rules in Equations 24 and 25 were applied to a series of examples of flexible and rigid pavement rehabilitation. The general approach was to define, for each pavement type, five arbitrary designs of different strengths: F1 through F5 for flexible pavements and R1 through R5 for rigid pavements. Each pavement design was subjected to four different traffic levels, corresponding to 5,000 AADT, 15,000 AADT, 25,000 AADT, and 35,000 AADT at the start of the analysis period (i.e., before growth). For each combination of pavement design and traffic level, the optimal rehabilitation time, t^*, was computed by solving either Equation 24 or Equation 25. The results were plotted to assess the general trends of the solution and to provide an easy way for engineers and administrators to apply these results in practice.

Given the design of this approach, not all the pavement-traffic combinations represent desirable situations. For example, some combinations impose heavy traffic on weak pavements, and others test light traffic on strong pavements. Nevertheless, including such combinations along with the more closely matched traffic-pavement design pairs has two advantages. First, it allows development of the solution function over a wide domain of traffic and pavement design possibilities and investigation of the behavior of the solution at the boundaries of typical situations. Second, it recognizes that, in their focus on existing pavements, rehabilitation decisions are different from those of design and new construction. (Refer to the several points at the beginning of this paper.) It is plausible that a pavement, once built, will be subjected to traffic levels much lighter or much heavier than that for which it was designed. Therefore, it should be possible to consider at least the possibility of some unforeseen combinations of design thickness and traffic.

Description of Examples

The numerical examples involved a two-lane, one-directional roadway with a design speed limit of 70 mph. The environmental region simulated was the northeastern United States.

The structural designs of the five rigid pavements and five flexible pavements tested are given in Tables 1 and 2. The portland cement concrete pavement is a plain jointed slab over a granular subbase; the asphalt pavement consists of an asphalt concrete surfacing over a granular base and subbase.

TABLE 1 Characteristics of Rigid Pavement Designs

Design	Subbase Thickness (in.)	Surface Thickness (in.)
R1	6	8
R2	6	9
R3	8	10
R4	9	11
R5	10	12
Layer coefficient	0.11	0.50

TABLE 2 Characteristics of Flexible Pavement Designs

Design	Subbase Thickness (in.)	Base Thickness (in.)	Surface Thickness (in.)
F1	12	6	4
F2	16	9	6
F3	24	12	8
F4	24	12	10
F5	24	12	12
Layer coefficient	0.11	0.14	0.44

In developing the examples, values had to be assigned to the parameters in Equations 24 and 25. These parameters can be grouped into subsets corresponding to the model's basic functional categories: pavement deterioration, $S(t)$; traffic demand, $q(t)$; user cost, $C(S,q)$; and investment cost, $I(S^{*-})$. To obtain these calibrations, the EAROMAR-2 simulation program was used to quantify general behavioral trends for these categories, using the same pavement designs, traffic loads, environmental conditions, maintenance policies, and other factors defined for the examples. From these results, specific values were assigned to the parameters in the model's detailed equations. Again, overlays were used to illustrate model application; however, other types of rehabilitation can also be represented under this approach.

The list of calibrated values is extensive and is presented elsewhere (2). Following are summary data that highlight key elements of the examples.

Pavement Deterioration

Values of k and β were determined to represent pavement deterioration for the various combinations of design and traffic level. To illustrate the results obtained, Figure 6 shows the predicted deterioration of the five rigid pavements tested for an initial traffic of 25,000 ADT; Figure 7 shows corresponding curves for flexible pavement.

The curves in Figures 6 and 7 correspond to Stage 1 (before the overlay), and rates of deterioration are quantified by β_1 and k_1 in Equation 3. After the overlay, the rigid pavement was treated as a compos-

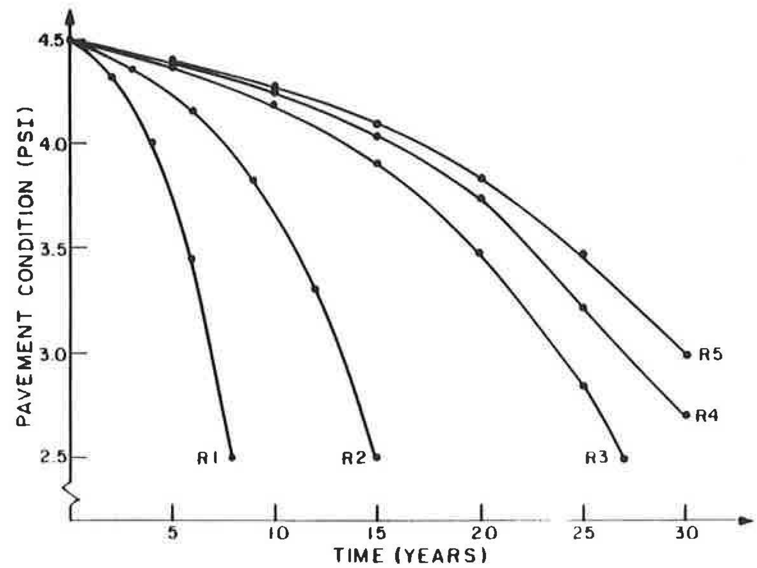

FIGURE 6 Rigid pavement deterioration curves for 25,000 AADT.

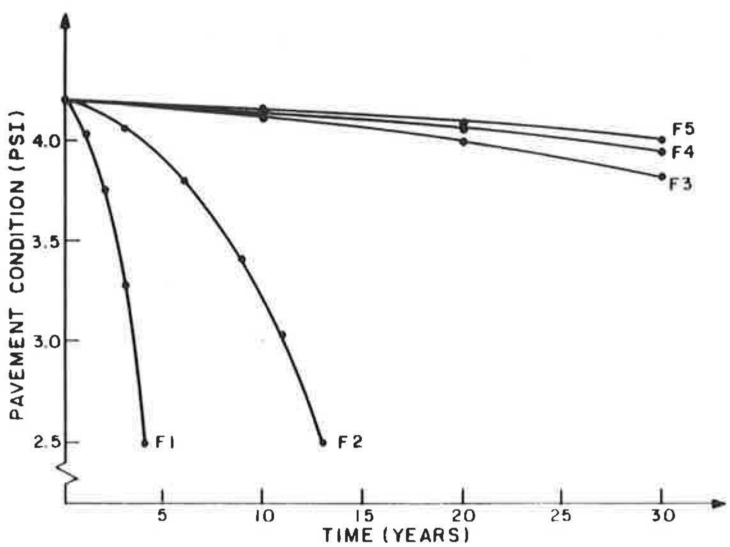

FIGURE 7 Flexible pavement deterioration curves for 25,000 AADT.

ite pavement, as described earlier; therefore, new values were determined for k_2 and β_2 in Equation 4, for each composite design C1 through C5 (corresponding to R1 through R5 after overlay), and for each traffic level (2). Also note that for 25,000 ADT, flexible designs F3 through F5 are simulated to act as premium pavements, showing only slight loss in PSI over a 30-year period. (Again, recall that a range of arbitrary designs was selected for the examples.)

Traffic Demand

Four cases of traffic demand were defined as discussed earlier, ranging from 5,000 to 35,000 AADT at the start of the analysis (t = 0). Because the model represents traffic in 18-kip equivalent single axle loads (ESALs), the AADT values were converted to ESALs. For the particular traffic stream simulated in EAROMAR-2, the conversion factor used was 0.1099 ESAL per vehicle. (Notwithstanding this conversion, traffic will continue to be described by AADT in this paper to retain clarity.)

Traffic growth is represented by the parameters a and b in Equation 5. Either positive or negative values for a and b are permissible; this example assumed a = 0.001 [or 0.1 percent per year per unit change in pavement condition S(t)] and b = 0.01 (or 1 percent per year).

User Costs

User costs were separated into a congestion effect and a pavement-related effect. The congestion functions simulated by EAROMAR-2 and used in the decision rule are shown in Figure 8.

The important point to observe from Figure 8 is that traffic volume itself has virtually no effect on user cost up to a certain threshold where congestion begins. As congestion increases, user costs

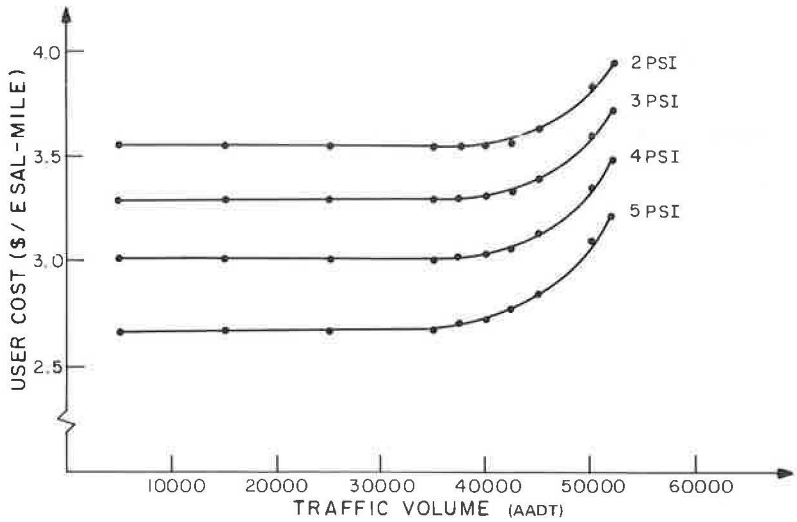

FIGURE 8 User cost versus traffic volume.

rise very quickly, due primarily to the dramatic rise in travel time costs. For the case of a two-lane, one-directional high-speed roadway, the EAROMAR-2 simulations indicate that user costs begin to be affected by traffic volume at approximately 37,500 AADT. The value of 37,500 AADT is henceforth referred to as the congestion threshold. The curves in Figure 8 are captured by the following relationship:

$$C(q) = \begin{cases} \text{constant}, 0 < q < q' \\ \delta(q-q)^n + K, q' \leq q < q_{max} \end{cases} \quad (27)$$

where q' is the congestion threshold.

The relationship of user costs to pavement condition was the subject of some review, as discussed earlier with respect to Figure 5. Based on findings of Zaniewski (5) and Ross (6), values were estimated for the parameters given in Equation 8, resulting in the following relationship used in the model:

$$C(S) = 1.4528 \exp(-1.25\ S) + 2.6972 \quad (28)$$

User Utility

User utility may be regarded as equal to the largest cost that a user will tolerate and still choose to use the roadway. In other words, it must be worth at least this much for a user to occupy the roadway; otherwise, he or she simply would not use it. This is true even for captive traffic that suffers high cost while occupying a roadway only because there exist no alternative links. This traffic still chooses to use the road and hence the utility to these users of doing so must be at least equal to the cost that they absorb.

In the development of the rehabilitation investment decision model, it has been assumed that utility is constant and the same for each user (a simplifying assumption to reduce mathematical complexity and field calibration requirements). For the current example, it was assumed that user utility is equal to the user cost associated with 1.0 PSI and 50,000 AADT. For this traffic level, a nonlinear C(S) curve similar to the one in Figure 5 indicates that this value is about \$3.45/ESAL·mile, or roughly \$0.28/vehicle·mile.

Overlay Costs

Building Construction Cost Data 1981 (8) indicates that \$1.62/yd^2·in. is a representative cost estimate for placing a bituminous wearing course for asphalt priced at \$19.95 per ton. (The figure of \$1.62/yd^2·in. can be adjusted to reflect price changes.) This estimate includes materials, installation, and contractor's overhead and profit.

The 1-mi length of roadway assumed for the case study encompasses 23,466.67 yd^2/mile; the cost of placing a new bituminous wearing course therefore becomes approximately \$38,000/in.·mile. However, this does not represent the only cost associated with the investment. There are also base preparation, mobilization, and line painting, as well as the public highway department's design, inspection, and general overhead costs. To account for these costs, 16 percent has been added to the estimate. The investment cost parameter, labeled σ, used in the case studies therefore equals \$44,100/in.·mile; thus, a 2-in. overlay would cost \$88,200/mile, and a 3-in. overlay would cost \$132,300/mile.

To represent the variability in overlay costs over time, λ in Equations 21 and 22 was estimated to equal 0.2554. A minimum overlay thickness of 2 in. was specified.

Discount Rate

A discount rate of 7 percent was used (assuming constant dollar estimates excluding inflation).

Optimal Rehabilitation Times

The solutions for the optimal time of rehabilitation investment are most easily presented in graphic form. The solutions for rigid pavements developed for the case study are shown in Figures 9 and 10. These two figures represent the same set of solutions plotted in different ways. Similarly, Figures 11 and 12 show the solutions for flexible pavements. The trends portrayed by these curves may be used in making a number of decisions.

The most direct application is in the programming of rehabilitation expenditures. For a given pavement and traffic, the optimal time to rehabilitate may be

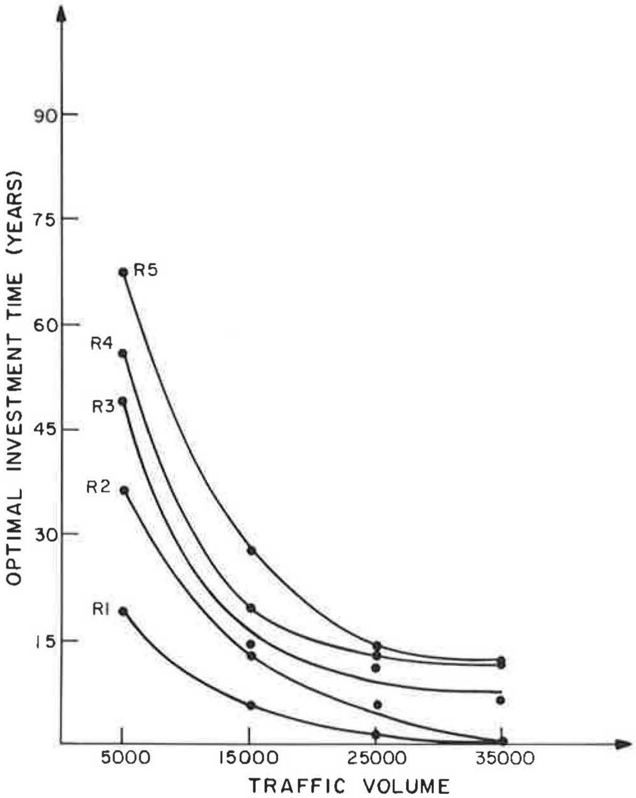

FIGURE 9. Optimal investment time versus traffic volume for variable traffic and variable investment cost (rigid pavement); note: traffic volumes represent initial values.

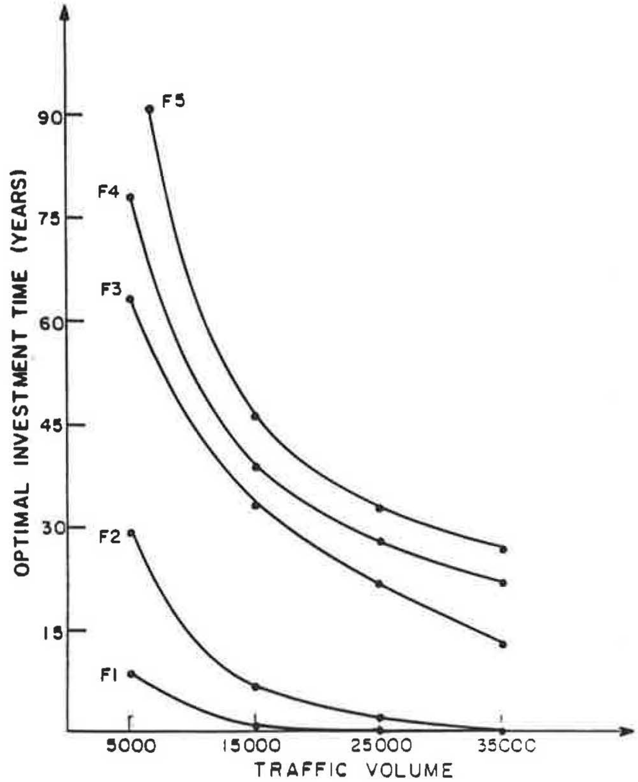

FIGURE 11 Optimal investment time versus traffic volume for variable traffic and variable investment cost (flexible pavement); note: traffic volumes represent initial values.

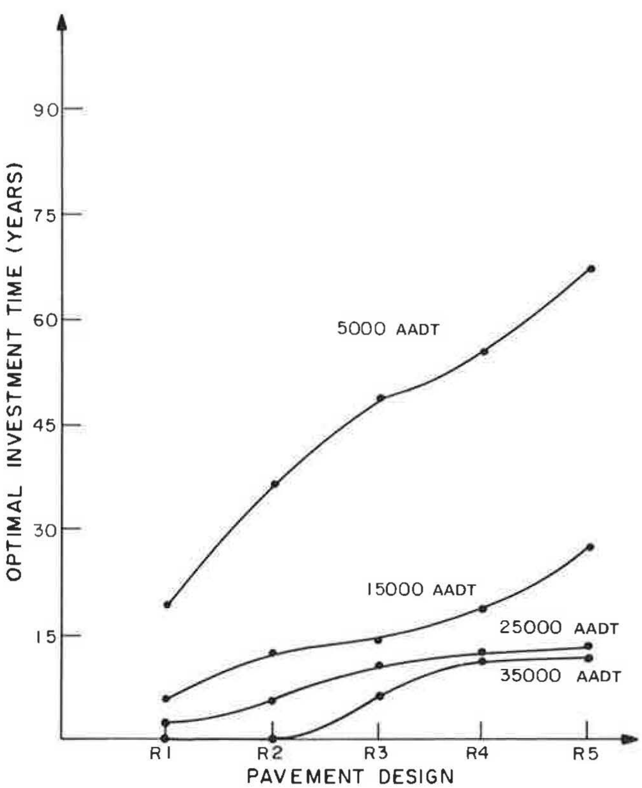

FIGURE 10 Optimal investment time versus pavement design for variable traffic and variable investment cost (rigid pavement); note: traffic volumes represent initial values.

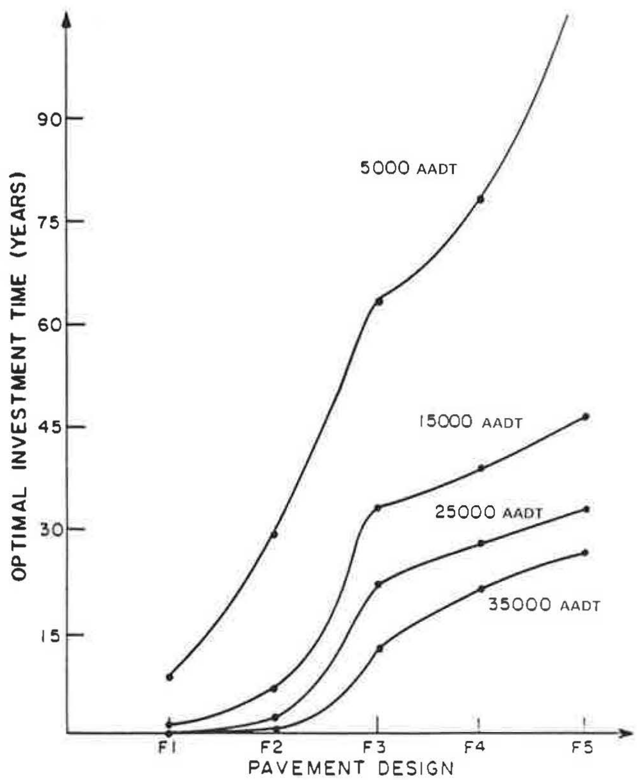

FIGURE 12 Optimal investment time versus pavement design for variable traffic and variable investment cost (flexible pavement); note: traffic volumes represent initial values.

determined and used as the basis for scheduling work and allocating funds. Either Figure 9 or Figure 10 for rigid pavements, or Figure 11 or Figure 12 for flexible pavements may be used for this purpose. It is also possible to organize a series of such calculations for a pavement network and to develop rehabilitation programs on either an open-ended or a budget-constrained basis.

The second application is in the evaluation of design-rehabilitation trade-offs. For example, design procedures such as AASHTO's (3) are fixed by an assumed 20-year pavement life. However, Figures 9 and 11 indicate that, for a given traffic projection, a number of designs and design lives are possible. For example, assuming an initial traffic of 15,000 AADT for rigid pavements, designs with optimal rehabilitation times of about 15 years (e.g., R2 or R3) or stronger pavements with longer optimal lifetimes (e.g., 19 years for R4, or 28 years for R5) could be selected. Note, however, that Figures 9 and 11 simply indicate the optimal investment time for a given set of circumstances (in this case, pavement design and traffic level); they do not indicate which design-rehabilitation combination has the lowest life-cycle cost. This cost information can be obtained, however, by evaluating the objective function, Equation 6, at t* for each pavement design. (In this case, the costs of pavement initial construction must be included in the objective function.)

The third application is the conduct of sensitivity analyses of pavement design and rehabilitation with respect to traffic volume. For example, consider the curves for the five rigid designs in Figure 10, and assume that the best available traffic load projection for a pavement corresponds to 25,000 AADT, but is subject to considerable uncertainty. Observe from Figure 10 that, at this level of traffic, designs R1 and R2 are relatively sensitive to changes in traffic, whereas R3 through R5 are less so. Again, only sensitivity is indicated by the curves in Figures 10 and 12; to understand the cost impacts, the objective function, Equation 6, at the solution t* must be evaluated.

One remaining point that is important to reiterate is that the solutions defined by Equations 24 and 25, and illustrated by Figures 9-12, are based on both economic and technical criteria, as opposed to the purely technical criteria traditionally applied to pavements. For example, a common standard derived from the AASHTO Road Test is that high-type pavements be overlaid at a PSI of 2.5. However, in the results indicated by Figures 9-12, the pavements, for the most part, were overlaid at PSI values higher than 2.5. This trend appears to be consistent with results of a survey of highway departments conducted by AASHTO (Summary of Selected State Practices Collected in 1980 Through AASHTO for the Truck Size and Weight Study, Section 161, Surface Transportation Assistance Act of 1978, memorandum, Federal Highway Administration, December 1982), showing considerable variability in the actual threshold for overlays used by different states. (Bear in mind also that the model is predicting a desired, not the actual, threshold.) Moreover, note that of the three cases defined by Balta (2), the case reported herein is the one most likely to drive pavement rehabilitation earlier, because its assumptions favor to a greater degree the benefits to both highway agency and road users of a high-quality pavement that is sustained by more frequent overlays. The adoption of one of the alternative assumptions investigated by Balta (i.e., the case of constant traffic over time, or the case of constant rehabilitation cost over time) would tend to defer the optimal rehabilitation time.

CONCLUSION

This paper began with the premise that maintenance and rehabilitation are inherently different from new construction because they involve an existing facility and require an understanding of facility performance as opposed to design. Moreover, the optimization of maintenance and rehabilitation policy is difficult because of the different options available in the choice and the timing of activities because pavement performance and related costs change over time, with no definitive point of failure. Methods of evaluating maintenance and rehabilitation policy cannot be based solely on technical grounds but must also include life-cycle costs and benefits.

As an example of one promising avenue, pavement rehabilitation, has been investigated and an analytic procedure to determine the optimal time to overlay for both flexible and rigid pavements has been developed and investigated. The procedure is based on principles of dynamic control theory and yields results that can be organized within sets of curves that are easy to understand and use. The control theory approach is predicated on an objective function rooted in engineering and economic principles and subject to constraints representing the detailed technical performance of the pavement-traffic interaction.

A series of examples has been presented to illustrate the nature of the solution and demonstrate the practicality of the results. Data for these examples were obtained from the EAROMAR-2 simulation model. However, to be a truly effective tool, the control theory solution should be calibrated by relationships validated in the field. This is true for all categories of parameters identified earlier in the paper but is especially true for user costs, which play a strong role in driving the solution of t*, the optimal time to rehabilitation.

Several potential applications of the control theory solutions have been discussed, including the programming of pavement rehabilitation, the investigation of design-rehabilitation trade-offs, and sensitivity analyses. More generally, the control theory solution for rehabilitation, coupled with an analogous solution for routine maintenance, could play an important role in many aspects of pavement management.

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Pavement Routine Maintenance Cost Prediction Models

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ABSTRACT

In this paper a methodology is presented for using the available data on pavement routine maintenance from the Indiana Department of Highways (IDOH) to develop models relating the cost of pavement routine maintenance to pavement system characteristics on a network level. The results showed that total pavement routine maintenance costs are affected by traffic level and by climatic zone (weather effect). Furthermore, the analysis of costs of individual activities showed that the extent of patching work (amount of pothole repair that is done after winter) is negatively correlated to the amount of sealing activity that takes place before winter. The implication of this result is that a higher level of service (fewer potholes) may be achieved by increasing sealing activity.

One of the common shortcomings of most current highway maintenance management systems (MMS) is that they are primarily designed for managing available resources (labor, materials, and equipment) and not geared to managing pavement facilities (1). The focus in this paper is on the use of available maintenance management data to provide information that can be directly employed in highway pavement management. In particular, models for pavement routine maintenance costs, which can be effectively used in preparing annual maintenance programs as well as in making decisions about resurfacing and rehabilitation, particularly on a network level, were developed.

As is the case in many other states, the maintenance management system in Indiana is designed for resource management and the necessary data are recorded on an aggregated unit representing a subdistrict. However, other pavement-related information is recorded on the basis of a contract. On the average, a subdistrict may include more than 100 contracts. The nonconformity between the maintenance data and the pavement data makes it difficult to use MMS information effectively in pavement management. For the purpose of this study a system was developed to represent all available data in terms of a highway section that was defined as the part of a highway within a county limit. This system allowed the maximum use of both the MMS data and the pavement management data as a unified information base. The

Indiana state highway system was divided into 820 highway sections.

DATA BASE

The state highway system of Indiana is divided into two categories: Interstate and other state highways (OSH). In this study the two highway systems were further subdivided by geographic location (climatic zone) and by pavement type. Two geographic locations, north and south, were included to reflect the major climatic differences in Indiana (2). The pavement types considered were flexible pavement, rigid pavement, and resurfaced pavement.

For each of the 820 sections, four major groups of information were summarized: traffic, pavement characteristics, climatic zone, and pavement maintenance records. Traffic information included average annual daily traffic (AADT), percentage of trucks, and equivalent axle load (EAL). The EAL was used as the common traffic index to account for different vehicle types and weights (2). Pavement characteristics included pavement type, layer thickness, and age. Climatic zones included geographic areas with similar climate in terms of snowfall, rainfall, temperature difference, and so on. Finally, pavement maintenance records included total production units, total manhours, and types and quantities of materials. Pavement maintenance information was summarized for each highway section by activity and by fiscal year. Pavement routine maintenance activities consisted of the following: shallow patching, deep patching, premix leveling, seal coating, sealing longitudinal cracks and joints, sealing cracks, cutting relief joints, joint and bump burning, and others. The unit costs and different resource consumption rates for these activities were obtained from previous studies (3,4) in addition to available information from IDOH (5-9). An important feature of the study was that it used only those data that are routinely collected by the state.

RESULTS

With network-level management in mind, the information included in the data base was used to develop statistical models that express expected pavement maintenance costs as a function of highway system, pavement type, traffic level, and climatic zone.

Study Unit

As indicated earlier, a highway section was considered as the study unit for this analysis. However, not all highway section data could be used because some of the sections included more than one pavement type (more than one contract). Only those highway sections that had the same pavement type and other characteristics along their entire length were considered.

In Indiana rigid pavement is the major type of pavement on the Interstate system (about 70 percent of total Interstate lane-miles). On the other hand, flexible pavement and resurfaced pavement are the major pavement types on the OSH system (about 90 percent of total OSH lane-miles). In summary, these three categories constitute about 85 percent of the total lane-miles of the state highway system in Indiana. Homogeneous sections were found in the three categories as follows: 26 sections of Interstate rigid, 213 sections of OSH flexible, and 84 sections of OSH resurfaced. The other three categories, Interstate flexible, Interstate resurfaced, and OSH rigid, which represent about 15 percent of the total lane-miles, were found to have a small number of homogeneous sections (five sections for Interstate flexible, eight sections for Interstate resurfaced, and five sections for OSH rigid). Moreover, most of these sections were located in the southern part of the state. These limitations were considered in the statistical analysis.

Total Pavement Maintenance Cost Prediction

Multiple regression analysis was performed to develop pavement maintenance prediction models. Each of the six categories was considered separately and 10 different regression models were tested to fit the data. Five criteria were considered in selecting the best regression model: (a) the general goodness of fit represented by the coefficient of multiple determination (R^2), (b) the general linearity test for the model through the application of the general F-test, (c) the significance of individual coefficients of the model through the t- or F-tests, (d) testing for the presence of autocorrelation problems through the Durbin-Watson test, and (e) the percentage of outliers. For each model within each of the six categories, the five criteria were applied and the best model was specified. An attempt was made during the analysis to have the same model type for the six categories to facilitate comparison of the effects of different factors.

After an intensive search, one model appeared to satisfy all required conditions for the six categories. This model is

$$\log_{10}(TC) = a \log_{10}(\Sigma EAL) + b \log_{10}(\Sigma EAL) \cdot (Z) + c\,(Z) \qquad (1)$$

where

TC = total pavement maintenance cost in dollars per lane-mile per year;
ΣEAL = accumulated equivalent axle load applications (in thousands) during the entire age of the pavement section (number of years since last major activity); and
Z = dummy variable to represent the zone in which the section is located; this variable takes the value of 1 when the pavement section is located in the northern zone of Indiana and the value of 0 if it is located in the southern zone.

The term $\Sigma EAL \cdot Z$ was introduced to measure the effect of the interaction between traffic level and climatic zone.

In almost all cases, and particularly in the type of models presented in Equation 1, the main effect of the variable Z was found to be insignificant. This can be explained by the fact that the dummy variable Z in reality measures the effect of the difference between the two zones and not the direct effect of the climatic zone variable. However, it was found in all cases that the interaction between traffic level and climatic zone was significant. Consequently, the model in Equation 1 was reduced to

$$\log_{10}(TC) = a \log_{10}(\Sigma EAL) + b \log_{10}(\Sigma EAL) \cdot Z \qquad (2)$$

The models for the six categories follow.

For Interstate flexible pavement

$$\log_{10}(TC) = 0.61 \log_{10}(\Sigma EAL) \qquad R^2 = 0.87 \qquad (3)$$

For Interstate rigid pavement

$$\log_{10}(TC) = 0.530 \log_{10}(\Sigma EAL) + 0.032 \log_{10}(\Sigma EAL) \cdot (Z) \quad R^2 = 0.89 \tag{4}$$

For Interstate resurfaced pavement

$$\log_{10}(TC) = 0.590 \log_{10}(\Sigma EAL) \quad R^2 = 0.81 \tag{5}$$

For OSH flexible pavement

$$\log_{10}(TC) = 0.974 \log_{10}(\Sigma EAL) + 0.24 \log_{10}(\Sigma EAL) \cdot (Z) \quad R^2 = 0.85 \tag{6}$$

For OSH rigid pavement

$$\log_{10}(TC) = 0.681 \log_{10}(\Sigma EAL) \quad R^2 = 0.87 \tag{7}$$

For OSH resurfaced pavement

$$\log_{10}(TC) = 0.850 \log_{10}(\Sigma EAL) + 0.040 \log_{10}(\Sigma EAL) \cdot (Z) \quad R^2 = 0.80 \tag{8}$$

A comparison of actual maintenance expenditures and estimated values for Interstate rigid pavement is shown in Figure 1.

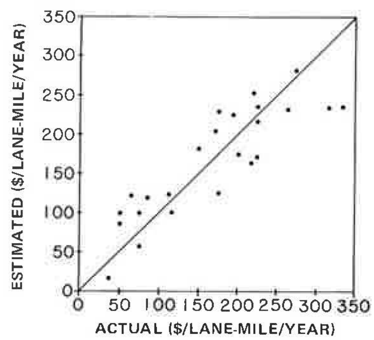

FIGURE 1 Estimated versus actual pavement routine maintenance costs for Interstate rigid pavement.

As may be noticed, Equations 3, 5, and 7 do not include the second term (interaction between traffic level and climatic zone). This is because the number of available sections in the northern zone for the corresponding categories was either small or non-existent as indicated earlier. It should be noted that the model presented in Equation 4 for Interstate rigid pavement included both jointed reinforced concrete and continuously reinforced concrete pavements. However, the preliminary work on this study, reported elsewhere (10), indicated a higher cost for continuously reinforced concrete sections than for jointed reinforced concrete sections. It was observed that the maintenance cost for continuously reinforced sections was about 15 to 35 percent higher than that for jointed reinforced sections subject to the same traffic level.

In general, it is believed that the choice of the model presented in Equation 2 was appropriate for all categories. It should be mentioned here that, during the analysis, different models were tested for those categories that had enough homogeneous sections (OSH flexible pavement, OSH resurfaced pavement, and Interstate rigid pavement). When an acceptable model had been obtained for these categories, as shown in Equation 2, the general form was simply applied to the other categories for which the available data were not sufficient.

Implications of Total Maintenance Cost Prediction Models

The effect of traffic and its interaction with geographic location can be best demonstrated through the examination of Tables 1-4. In these tables, the average pavement maintenance costs at typical traf-

TABLE 1 Estimated Pavement Routine Maintenance Cost for Typical Traffic Levels on Interstate Flexible, Interstate Resurfaced, and OSH Rigid Pavements in the South

	Estimated Maintenance Cost ($/lane-mile/year)		
Accumulated EAL (10^3)	Interstate Flexible[a]	Interstate Resurfaced[b]	OSH Rigid[c]
1,000	68	59	110
2,000	103	88	177
3,000	132	113	233
4,000	157	133	284
5,000	180	152	330
6,000	202	169	374
7,000	222	186	415
8,000	240	201	456
9,000	258	215	492
10,000	275	229	530

[a] From Equation 3.
[b] From Equation 5.
[c] From Equation 7.

TABLE 2 Estimated Pavement Routine Maintenance Cost for Typical Traffic Levels on Interstate Rigid Pavement

	Estimated Maintenance Cost[a] ($/lane-mile/year)		
Accumulated EAL (10^3)	South	North	Ratio (north to south)
5,000	91	120	1.31
10,000	132	177	1.34
15,000	163	222	1.36
20,000	190	261	1.37
25,000	214	296	1.38
30,000	236	328	1.39
35,000	256	358	1.40
40,000	275	386	1.40
45,000	293	412	1.41
50,000	309	437	1.41

[a] From Equation 4.

TABLE 3 Estimated Pavement Routine Maintenance Cost for Typical Traffic Levels on OSH Flexible Pavement

	Estimated Maintenance Cost[a] ($/lane-mile/year)		
Accumulated EAL (10^3)	South	North	Ratio (north to south)
50	45	50	1.10
100	88	99	1.10
150	131	149	1.14
200	174	198	1.14
250	216	247	1.15
300	258	297	1.15
350	300	346	1.15
400	342	395	1.16
450	383	445	1.16
500	425	494	1.16

[a] From Equation 6.

TABLE 4 Estimated Pavement Routine Maintenance Cost for Typical Traffic Levels on OSH Resurfaced Pavement

Accumulated EAL (10^3)	Estimated Maintenance Cost[a] ($/lane-mile/year) South	North	Ratio (north to south)
100	50	60	1.20
200	90	112	1.24
300	127	160	1.26
400	163	207	1.28
500	196	252	1.29
600	230	297	1.29
700	262	341	1.30
800	293	383	1.31
900	324	426	1.31
1,000	354	468	1.32

[a]From Equation 8.

fic levels (ΣEAL) for appropriate combinations of highway system, pavement type, and climatic zone (north and south) are presented. It is clear that, for the same pavement type and traffic level, the average maintenance cost of the Interstate sections is less than that of the OSH. This is because the Interstate system received a higher rate of major maintenance activities, particularly in the last few years, which may have reduced the need for higher levels of routine maintenance. In general, it can also be expected that maintenance levels differ significantly by pavement type within a particular highway system. For example, in both the Interstate and OSH systems, at the same traffic levels, the highest unit pavement maintenance cost was observed in flexible pavement followed by resurfaced pavements and then rigid pavement. However, an important finding of this analysis was that the effect of traffic level on the difference in maintenance costs is not constant. To illustrate, the Interstate rigid and resurfaced pavements can be considered. At a traffic level of 5 million accumulated EAL, the ratio between maintenance cost of resurfaced pavement and rigid pavement is 1.67, whereas this ratio becomes 1.75 at a traffic level of 10 million accumulated EAL. Similarly, for the OSH system, at traffic levels of 500,000 and 1 million accumulated EAL, the corresponding cost ratios are 2.8 and 3.2, respectively.

An important application of these models is in assessment of the effect of climatic zone and of the interactive effect of traffic level and climatic zone on pavement maintenance cost. The effect of climatic zone on maintenance costs can be easily seen from Equations 4, 6, and 8 for the Interstate rigid pavement, OSH flexible pavement, and OSH resurfaced pavement, respectively. It is clear that, in general, pavement maintenance costs in the northern part are higher than in the southern part. The models, however, not only confirm the geographic difference in maintenance costs, they also point out that this difference increases as the traffic level increases due to interaction effect. For example, for the Interstate rigid pavement, at the relatively low traffic level of 1 million accumulated EAL, the pavement maintenance cost in the northern zone is about 24 percent higher than that in the southern zone; at a higher level of traffic of 45 million accumulated EAL, the northern zone average cost is 40 percent higher than that of the southern zone. The difference between 40 and 24 percent at traffic levels of 1 million and 45 million could be attributed to the interaction effect between traffic level and climatic zone. For the OSH flexible pavement, the ratio between average cost in the northern zone and that in the southern zone ranges from 1.10 at relatively low traffic levels to 1.16 at higher traffic levels. Similarly, for the OSH resurfaced pavement, the corresponding ratios are 1.20 and 1.32, respectively. The main conclusion that can be drawn from these results is that, at higher traffic levels, the effect of climatic zone (weather effects) tends to be more severe. However, the degree of interaction is significantly dependent on pavement type. For example, the unit maintenance cost for OSH flexible pavement in the northern zone is about 16 percent higher than that in the southern zone at a traffic level of 400,000 EAL (Table 3) and about 31 percent for OSH resurfaced pavement at the same traffic level (Table 4). This trend is consistent at all levels of traffic, and it can be concluded that the effect of climatic zone (weather factor) on maintenance cost is more pronounced for resurfaced pavement.

Maintenance Group Cost Prediction Models

The next phase of the study involved the development of cost models for individual maintenance activity groups, namely patching and sealing. The patching group included shallow patching and deep patching. The sealing group included sealing longitudinal cracks and joints and sealing cracks. Models for the

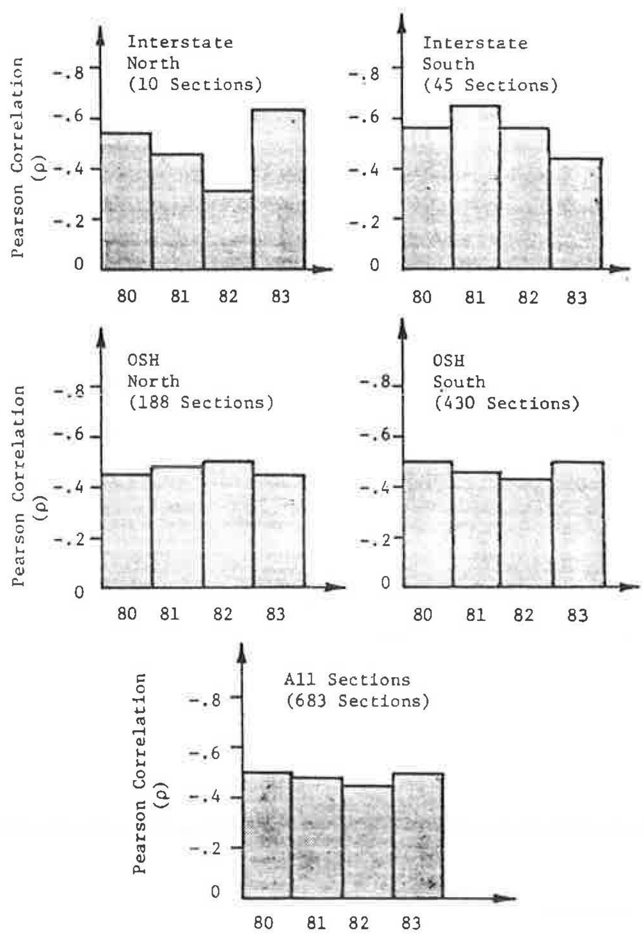

FIGURE 2 Correlation analysis of sealing and patching activities; analysis is based on percentages of total cost allocated to sealing and patching.

prediction of individual maintenance activity group costs can provide a tool for estimating the portions of total maintenance cost that can be attributed to different activity groups such as patching and sealing. In addition, these models can be used to gain insight into the interaction of various maintenance activities under different levels of traffic.

Patching and sealing activities comprise about 85 percent of the total pavement maintenance cost (10) and there is a high correlation between patching and sealing performed in the same fiscal year. Figure 2 shows the results of a detailed correlation analysis performed on the portions of total cost allocated to patching versus those allocated to sealing for different highway categories and fiscal years. A correlation value as high as -0.6 between portions of total cost allocated to sealing and patching was found. The scheduling of different maintenance activities in a fiscal year adds a particular characteristic to the correlation between patching and sealing. This is because sealing activities usually precede patching activities within a fiscal year. Sealing activities take place in the late summer and fall, and patching usually takes place during the spring season after the winter. Although there might be some variation in scheduling of these activities, the majority of sealing and patching jobs occurs during the periods mentioned.

A high correlation between patching and sealing in a fiscal year is a one-way correlation that indicates that the amount of patching done in a year is generally dependent on the extent of sealing performed before the winter. However, sealing activity does not depend on patching activity.

The general type of regression models for patching and sealing follows the form presented in Equation 2. Equations 9-20 are the models developed for sealing and patching for each highway category.

For Interstate flexible pavement

$$PS = 0.185 \cdot \log_{10} (\Sigma EAL) \quad R^2 = 0.87 \tag{9}$$

$$PP = 0.182 \cdot \log_{10} (\Sigma EAL) - 0.670 \cdot PS \quad R^2 = 0.83 \tag{10}$$

where

PS = percentage of total pavement maintenance cost allocated to the sealing group and
PP = percentage of total pavement maintenance cost allocated to the patching group.

For Interstate rigid pavement

$$PS = 0.098 \cdot \log_{10} (\Sigma EAL) - 0.015 \cdot \log_{10} (\Sigma EAL) \cdot Z \quad R^2 = 0.81 \tag{11}$$

$$PP = 0.206 \cdot \log_{10} (\Sigma EAL) - 0.023 \cdot \log_{10} (\Sigma EAL) \cdot Z - 0.998\ PS \quad R^2 = 0.95 \tag{12}$$

For Interstate resurfaced pavement

$$PS = 0.115 \cdot \log_{10} (\Sigma EAL) \quad R^2\ 0.91 \tag{13}$$

$$PP = 0.186 \cdot \log_{10} (\Sigma EAL) - 0.621 \cdot PS \quad R^2 = 0.85 \tag{14}$$

For OSH flexible pavement

$$PS = 0.22 \cdot \log_{10} (\Sigma EAL) - 0.074 \cdot \log_{10} (\Sigma EAL) \cdot Z \quad R^2 = 0.78 \tag{15}$$

$$PP = 0.346 \cdot \log_{10} (\Sigma EAL) + 0.025 \log_{10} (\Sigma EAL) \cdot Z - 0.786\ PS \quad R^2 = 0.89 \tag{16}$$

For OSH rigid pavement

$$PS = 0.1075 \cdot \log_{10} (\Sigma EAL) \quad R^2 = 0.82 \tag{17}$$

$$PP = 0.150 \cdot \log_{10} (\Sigma EAL) - 0.135\ PS \quad R^2 = 0.92 \tag{18}$$

$$PS = 0.196 \cdot \log_{10} (\Sigma EAL) - 0.0617 \cdot \log_{10} (\Sigma EAL) \cdot Z \quad R^2 = 0.81 \tag{19}$$

$$PP = 0.228 \cdot \log_{10} (\Sigma EAL) + 0.011 \cdot \log_{10} (\Sigma EAL) \cdot Z - 0.55 \cdot PS \quad R^2 = 0.84 \tag{20}$$

Implications of Maintenance Group Cost Prediction Models

The models presented in Equations 9-20 are for sealing and patching costs in terms of the percentage of total pavement maintenance cost required at different traffic levels and zones (north and south). As may be seen, sealing prediction models are functions of traffic level (accumulated EAL) and zone (north and south), and patching prediction models are functions of traffic level and zone and also of the level of sealing performed in the same fiscal year. The reason for this is that although patching level is highly correlated with sealing level, sealing activity does not show significant dependence on patching level. This is mainly because most sealing jobs are scheduled before patching jobs within the same fiscal year. In Figures 3-5, graphic presentations of sealing and patching percentages of the total cost at typical traffic levels are shown.

The first implication of Figures 3-5 is that both sealing and patching shares of the total pavement maintenance cost increase as the traffic level increases. This is expected because an increasing traffic level accelerates the pavement distress process, and this, in turn, requires an increased level of pavement maintenance, primarily sealing and

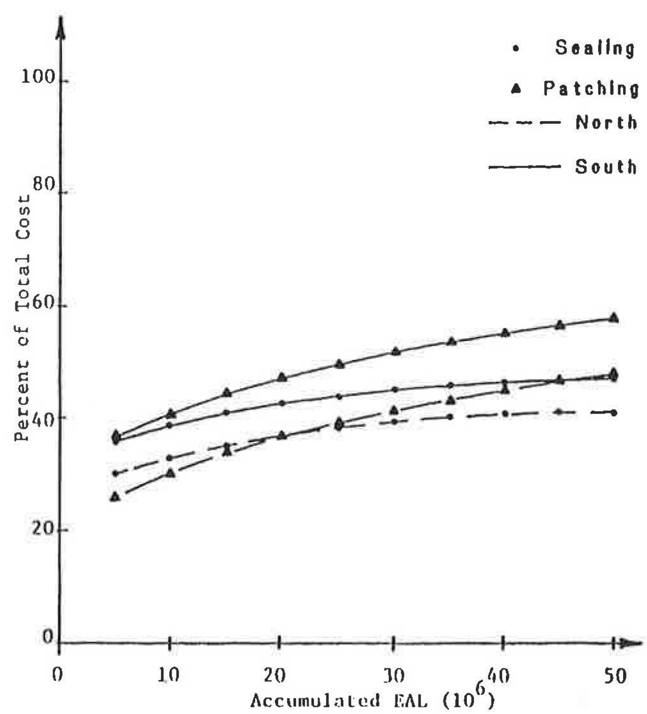

FIGURE 3 Estimated patching and sealing percentages for Interstate rigid pavement.

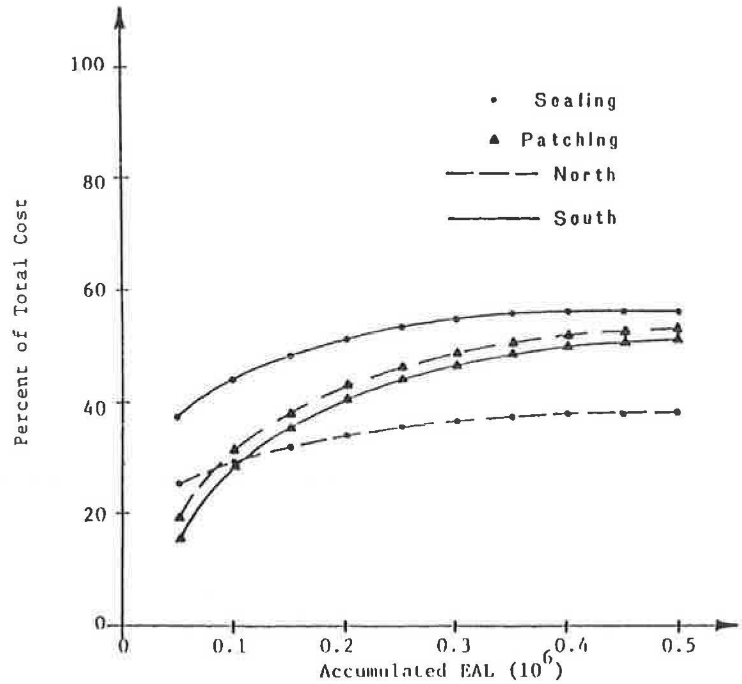

FIGURE 4 Estimated patching and sealing percentages for OSH flexible pavement.

patching. However, the rate of increase in the patching share as traffic level increases is higher than that in sealing.

Second, both rigid and resurfaced pavements show a similar trend in terms of higher patching and sealing shares in the south. This trend, however, was found to be different in the case of flexible pavement (Figure 3) where patching shares are higher in the north than in the south. This could be due to the relatively low level of sealing in the northern part because of the short season available for sealing activity. Because sealing is a type of preventive maintenance, a low level of sealing activity causes a high level of corrective maintenance, patching.

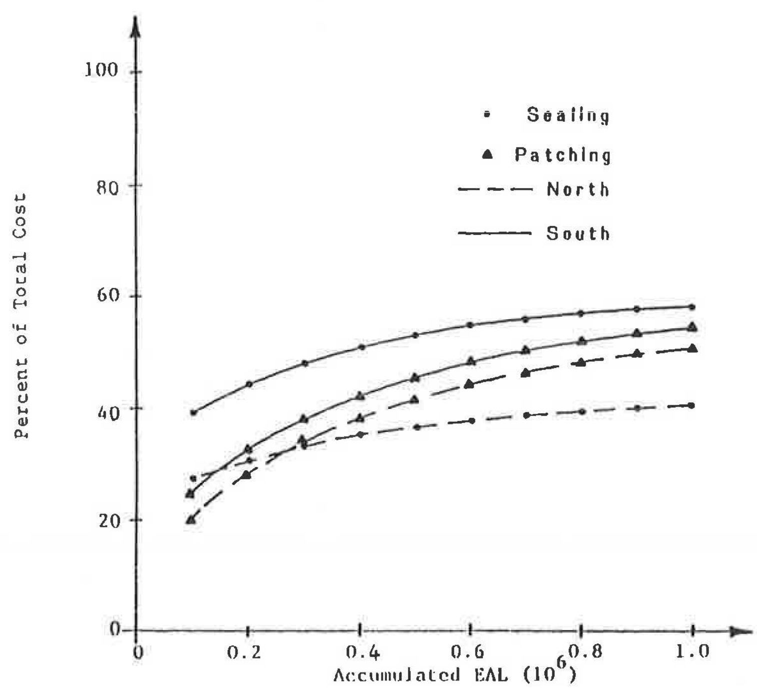

FIGURE 5 Estimated patching and sealing percentages for OSH resurfaced pavement.

CONCLUSIONS

On the basis of the findings of this study, the following major conclusions can be drawn:

1. Total pavement routine maintenance costs for a particular pavement type were found to be significantly affected by traffic level and geographic location (climatic zone). However, it was found that the interaction effect of traffic level and climatic zone (weather factor) is more significant than is that of climatic zone alone.
2. Portions of total cost allocated to major routine maintenance activity groups (sealing and patching) were found to be functions of the same factors. However, it was found that the patching level (amount of patching activities taking place after winter) is negatively affected by the level of sealing (amount of sealing activities taking place before winter). That is, the more sealing of cracks a highway section receives before wintertime, the less pothole patching is required after winter.

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Flow Characteristics at Freeway Lane Closures

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ABSTRACT

The findings of a limited study aimed at examining the basic characteristics of freeway traffic flow at construction zones are presented. The intent is to expand the scope of previous research efforts in this area, which have focused on the determination of point estimates of work zone capacity, under a variety of freeway lane configurations upstream of and in the vicinity of the work area. Field studies conducted in Illinois, encompassing more than 21,000 vehicle observations, were used to examine the entire range of the speed-flow relationship in the open lane of traffic. A normalizing procedure was devised to isolate and quantify the impact of work zone activity descriptors, such as the location of work relative to the traveled lanes, crew size, equipment, and other pertinent parameters, on the observed traffic speed. It was found that the effect of work activity on traffic flow is significant in periods of (a) high approach flow rates, (b) high truck percentages, and (c) intense work activity near the traveled lanes.

Estimating capacity and level of service (LOS) at freeway construction zones is essential to planning and scheduling work zone traffic control. A comprehensive evaluation of alternate control strategies, including their respective traffic impacts, must be carried out before a particular procedure is recommended for implementation. Traffic performance measures such as delays, stops, fuel consumption, and operating costs are directly related to the capacity and operating speed on the roadway segment. Thus a thorough examination of the basic speed-flow relationship at freeway lane closures is warranted.

It may be stated that traffic speed through a lane closure is primarily governed by the following factors:

- Geometrics including lane configuration before and at the work area; grades and curvatures; effective lane width and lateral clearance; sight distance and proximity to on and off ramps;
- Traffic stream including flow rates past the work area and truck occurrence in the traffic stream;
- Traffic control including signing, arrow board, and channeling devices; speed zoning; presence of flagmen or patrolmen, or both; and
- Work activity descriptors including activity location, crew size, equipment type, noise and dust level generated and length of the work area.

Although considerable research has focused on the problem of speed control at work zones, little is known about the correlation between traffic speed and work zone activity. A procedure is thus required to isolate the effect of work zone activity from geometrics, traffic stream, and traffic control impacts. This study is a first attempt to address these issues. Specifically, the following objectives are sought:

1. Review previous work related to speed-volume-capacity relationships at freeway lane closures.
2. Conduct uncontrolled field studies to generate a data base for developing speed-flow models at the visited work sites.
3. Estimate the magnitude and direction of work activity impact on observed traffic speed; segregate by flow rate, truck occurrences, and work activity levels to study the contribution of each parameter.

LITERATURE REVIEW

In the relatively few years since the problem of work zone safety has received national recognition (e.g., 1975 FHWA coordinated research program project 1Y Traffic Management of Construction and Maintenance Zones), speed control at work zones has been a persistent problem. Traffic engineers have yet to agree on whether speed reduction is desirable or whether traffic speed should be maintained throughout the work zone by means of higher geometric standards. There is a consensus, however, that posted regulatory or advisory speed limits are ineffective in reducing speeds (1). Speed measurements collected to test the effectiveness of speed control measures were essentially limited to free-flowing vehicles (headways $\geq$ 5 sec). The impact of construction activity was not specifically analyzed in these studies (2-4).

A majority of the work reviewed in the course of this study dealt with the analysis of flow rates approaching capacity at lane closures (i.e., in situations in which a queue developed and was sustained upstream of the work zone). Dudek and Richards (5) have studied the impact of lane geometry on capacity at a number of urban freeway lane closures in Texas, and Kermode and Myrra (6) attempted to correlate observed capacity on the San Diego Expressway in California with the type (rather than the intensity) of construction activity (e.g., resurfacing, stripping). Their results, however, are based on 3-min observations of maximum flow rates past the work area and therefore tend to overestimate the hourly capacity expected at the sites.

It is interesting to note that the Texas study included limited observations of work zone capacity with no construction activity under way. The study found average lane capacity to be approximately 20 percent higher than that observed at similar sites with the work crew present. Although this result is based on data from a single site, it clearly demonstrates the degree to which work zone activity affects capacity and level of service at lane closures. In an earlier study by Butler (7), volume-to-capacity ratios and corresponding speeds measured at work zones were overplotted on the typical Highway Capacity Manual (HCM) curves (8). The author stated that the field data show tremendous scatter at particular volumes (unspecified) around HCM values but that they could still be approximated by the HCM curves. However, no numerical justification was provided for that argument. This approach was also followed in a recent study by Wang and Abrams (9) in developing a rational planning process for the selection of the most effective work zone traffic control strategy for a given project.

In a study by Dudash and Bullen (10), traffic speed and flow rates on the Penn-Lincoln Parkway in Pennsylvania were measured in order to estimate single-lane capacity during reconstruction. Their estimates were not substantially different from those observed in California and Texas. Although this study used speed-flow "envelopes" in a before-and-during construction comparison, their value is limited to a qualitative assessment of the lane closure impact under three types of control.

In brief, the literature review revealed two unresolved issues related to flow characteristics at freeway lane closures:

1. The applicability of the typical HCM speed-flow curve to traffic flow in construction zone lane closures and
2. The degree to which the work zone activity parameters influence speed for the entire range of volume-to-capacity (V/C) ratios.

FIELD STUDIES

Rationale and Scope

An exhaustive field investigation of all factors affecting operating speed at freeway lane closures requires an extensive data collection effort far beyond the resources of this study. In reference to geometric impact, it was decided to confine the sampling effort in this study to four-lane Interstate-type facilities with a single lane closure and no crossovers, for the following reasons:

• The effect of lane geometry is likely to be confounded with the impact of worksite activity, thus jeopardizing one of the principal objectives of this study. Moreover, lane geometry impact on capacity appears to be adequately documented in the literature.
• The study was originally designed to investigate rural freeway lane closures, where the four-lane divided roadway configuration is most prevalent.

Traffic control devices at the visited sites were in general conformance with the standards of the Illinois Manual of Traffic Control Devices (11). No advisory or regulatory speed limit signs were posted at any of the sites.

Site Description

Four projects were covered in the course of this study. The study sites were located within 60 mi of the Chicago area and all but one can be characterized as rural.

Site 1 was located on the northbound lanes of I-57, 1/4 mi north of US-30, at the southern boundary of Cook County, Illinois. At that site, a bridge repair project consisting primarily of steel joist replacement was under way. A crew of three workers, a foreman, and a flagman was present at the site. The work activity consisted of (a) breaking up concrete deck, (b) dusting, and (c) removing and replacing steel joists. Traffic control included advanced warning signs, a flashing arrow panel, and 18-in. cones for channelization in the lane closure area.

Site 2 was located on the eastbound lanes of I-80, east of the Illinois Route 7 interchange. The construction activity was similar to that of Site 1, except that relatively smaller drilling equipment and a larger crew size were present at this site. Also, a flagman was located just next to the work area; the flagman was located near the arrow panel at Site 1. No major differences in geometrics or traffic control devices were noted between the two sites.

Site 3 was located on the eastbound lanes of I-290, 3/4 mi east of Illinois Route 53. This section of I-290 was heavily congested during the data collection period, with queues extending to the Route 53 on-ramp. Flow rates past the work area were thus indicative of the lane closure capacity. Work activity at this site consisted of concrete pavement patching, which at times infringed on the open lane of traffic. Traffic control devices included advanced warning signs, arrow board, and vertical posts in the taper and lane closure areas.

Site 4 was located on the northbound lanes of I-55, 1 mi south of I-80 near Joliet, Illinois. Work at this site consisted of bridge deck repair. This site was characterized by a physical separation of work activity and vehicular traffic by means of a portable concrete barrier.

A summary of other pertinent site characteristics is given in Table 1.

TABLE 1 Summary of Site Characteristics

Site	Lane Closed	Length of Closure (ft)	Channelizing Device	Average Hourly Volume
I-57	Left	3,000	18-in. cones	535
I-80	Left	1,035	Type I barricades	1,193
I-290	Right	530	Tubular posts	435[a]
I-55	Left	435	Portable concrete barrier	760

[a] Measured in queuing conditions; does not reflect traffic demand.

Data Gathering

Three data elements were collected at each site:

1. Traffic speed and composition upstream of the work zone,
2. Simultaneous 5-min counts of speeds and flow rates at the beginning and end of the lane closure section, and
3. Work area activity descriptors for the intervals indicated in part 2.

Traffic speed upstream of the work zone was collected on a random sample of approaching vehicles via a radar gun. Vehicle types were recorded by a time-lapse camera located at a vantage point at each site. The recording interval varied from 1 to 3 sec, depending on the approach speed prevalent at the site. Speed and flow rate counts were collected using two Stevens PPRII print-punch classifiers located at both ends of the closure. All data were collected for a period of approximately 4 hr per site, except for Site 1 where equipment problems limited the data collection to 1 hr.

Because construction activity does not lend itself to numerical description, it was necessary to devise an ordinal-level scale to quantify the intensity (in terms of its vehicular impact) of the work activity. Six activity indicators, which have a potential impact on traffic flow characteristics, were selected:

1. Proximity of work activity to travel lane (PL). A numerical code from zero to four denotes the

location of the activity. An assigned zero code implies no work activity (e.g., lunch break), and a code of four describes work activity carried out at the lane edge (e.g., stripping). PL increases by one unit for each 3-ft shift in construction activity closer to the travel lane.

2. Crew size (CS). Active crew size in the work area.

3. Equipment code (EC). A numerical code from zero to three denotes the relative size of equipment operating at the site. A zero code implies no operating equipment, and a code of three indicates heavy equipment usage.

4. Flagman code (FC). Binary code indicating the presence (FC = 1) or absence (FC = 0) of a flagman in a particular time interval.

5. Noise level code (NL). A numeric code from zero to three designating the relative noise level at the site.

6. Dust level code (DL). A numeric code from zero to three designating the relative dust level at the site.

The sum of the numerical codes is termed the activity index (AI). It serves to identify those intervals during which construction activity interferes with traffic flow in an attempt to test the hypothesis that traffic speed is directly correlated to the intensity of the construction activity at the visited sites.

It should be noted that the work activity data were collected manually in 5-min intervals that corresponded to the speed-flow observations obtained by the traffic classifiers. A sample plot of the activity index history for Site 3 is shown in Figure 1.

Results

The results of data analysis are presented in three parts:

1. Speed distribution upstream of and at the lane closure area,
2. Speed-flow relationships at each site and comparison with HCM, and
3. Impact of work zone activity on traffic flow parameters.

Analysis of Speed Distribution

Speed distributions observed upstream of the work zone were tested for normality. Except for Site 3, speeds followed the normal distribution. It should be emphasized that at Site 3 traffic upstream of the work zone was operating in stop-and-go conditions; this made it impossible to derive an estimate of approach speeds before joining the queue because free-flow conditions did not occur within the instrumented segment of the road. It is interesting to note that mean speeds for the remaining sites were all within 2 mph of each other, as indicated in Table 2. Further testing by approach lane and vehicle type showed no significant differences.

Speed measurements taken at the beginning and end of the lane closures exhibited a consistent pattern of skewness, as shown in the sample histogram in Figure 2 for Site 1. A statistical test for skewness of speed distribution was borrowed from a study by Bleyl (12) in which the sampling distribution of the spot speed skewness index (SI) was found to be normal with a mean skewness index of 1.00 and a standard

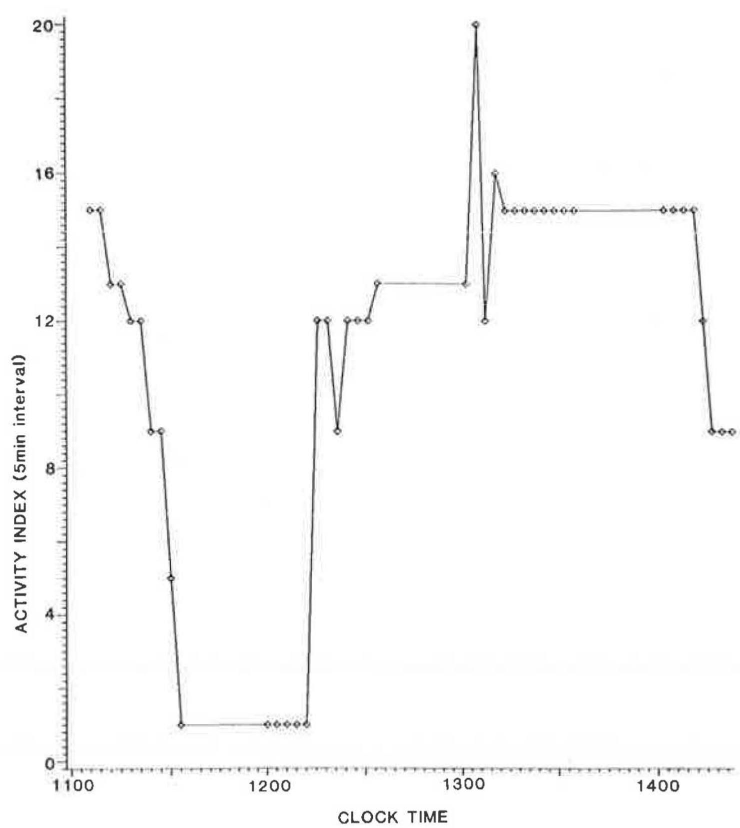

FIGURE 1 History of activity index at Site 3.

TABLE 2 Observed Speed Distribution Upstream of Lane Closure

Site	Sample Size	Mean Speed (mph)	Standard Deviation	χ^{2a}	Level of Significance
I-57	125	54.78	4.695	3.88	0.700
I-80	144	52.33	5.854	7.25	0.123
I-290	N/A	N/A[b]	N/A	N/A	N/A
I-55	157	53.25	4.971	1.38	0.636

[a] χ^2 goodness of fit statistic for normal distribution.
[b] Approach speeds upstream of lane closure could not be measured because of queue buildup.

error (SE), provided that the parent population is normal, where

$SI = 2 * (P_{93} - P_{50})/(P_{93} - P_7)$,
P_c = ith percentile speed,
$SE = (0.002 + 0.949)/SQRT(N)$, and
N = sample size.

The preceding test was applied to the pair of speed distributions observed at each end of the lane closure. The results given in Table 3 led to the rejection of the null hypothesis (i.e., parent population is normal) in all but one location at Site 4. This site exhibited higher speeds than other sites despite the presence of a concrete barrier at the edge of the traveled lane and a considerable reduction in lateral clearances on both sides of the lane.

It is interesting to note that, for skewed speed distributions with a left skew (SI < 1), the geometric condition (e.g., lane closure) is more restrictive than is perceived by the driver; that situation occurs at Sites 2 (start of closure) and 4 (end of closure). On the other hand, a right skew (SI > 1) indicates that the geometric condition appears worse than it actually is; this results in a large number of drivers slowing down to an "apparent" physical speed limited (13). This situation occurred at Sites 1, 2 (end of closure), and 3.

TABLE 3 Observed Speed Distributions at Lane Closure

Site Location	Mean Speed (mph)	Standard Deviation (mph)	Skewness Index[a]	Level of Significance
I-57				
Start of closure	38.18	6.81	1.056	<0.01
End of closure	37.63	5.43	1.115	<0.01
I-80				
Start of closure	38.11	7.82	0.99	<0.01
End of closure	38.49	7.85	1.157	<0.01
I-290				
Start of closure	26.75	9.81	1.45	<0.01
End of closure	31.51	4.10	1.30	<0.01
I-55				
Start of closure	51.36	7.33	1.004	0.389
End of closure	51.14	7.22	0.987	<0.01

[a] $SI = 2(P_{93} - P_{50})/(P_{93} - P_7)$.

Speed-Flow Relationship

Speed-flow patterns were analyzed at each site by aggregating the speed observations in each 5-min interval into a space-mean speed and corresponding mean flow rate. The time interval was selected such

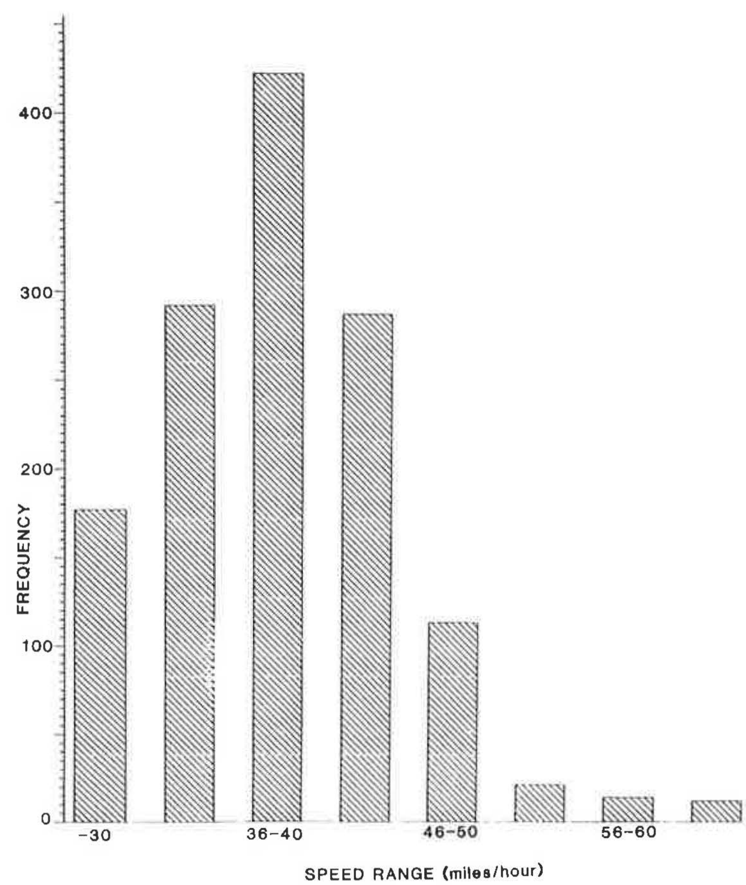

FIGURE 2 Speed distribution, Site 1, start of lane closure.

that the uniform flow assumption stated in the HCM was met and traffic fluctuations associated with short counting intervals were avoided.

A graphic representation of the observed speed-flow data is shown in Figure 3. The general shape formed by the data is surprisingly similar to the typical speed-flow curve in the HCM; that is, nonlinear in the high service volume regime and flow-independent speed values at the lower end of the service volumes. Site 3 data were concentrated in the congested and forced-flow regimes of the speed-flow curve.

The maximum observed flow rate was approximately 130 vehicles/5-min interval or 1,560 vehicles per hour, and the highest sustained hourly flow rate was 1,507 vehicles per hour, which corresponds to a peak-hour factor of 0.97.

A second degree polynomial fitted to the data is

$$V = -13.2 + 4.571S - 0.055S^2 \tag{1}$$

where V and S refer to the observed flow rates and corresponding space-mean speed, respectively. A capacity estimate can be derived from Equation 1 by setting the conditions

$$dV/dS = 0,\ d^2V/dS^2 < 0 \text{ at } V = V_{max} \tag{2}$$

From Equations 1 and 2, V_{max} = 975 vph and S_{opt} = 41.3 mph.

The regression model in Equation 1 thus gives unrealistic estimates of optimum speed and capacity, a very poor fit to the observed data (R^2 = 0.068) and, therefore, would have limited value for capacity estimation purposes.

The preceding analysis indicates that the derivation of capacity and level of service without regard to individual site variations, especially the extent of construction activity, the impact of truck traffic, the site geometrics, and so forth, will generally result in a tremendous scatter of the data points, as pointed out in an earlier study by Butler (7). To eliminate intersite variations, individual site models were generated using the linear form:

$$S = a + bV \tag{3}$$

where a and b are the regression coefficients and S and V are as defined earlier. These models were developed at locations adjacent to the work area (either start or end of the lane closure). A total of 146 sample points were included in this analysis.

The models are given in Table 4 and shown in Figure 4. It is observed that the flow rates at Sites 1, 2, and 4 varied from 40 (480 vph) to 120 (1,440 vph), with some overlap between sites. The regression coefficients were quite realistic in that as volume increased the slope b in Equation 3 consistently decreased and the corresponding intercept a consistently increased. However, slopes that were statistically significant (i.e., $b \neq 0$) occurred in the volume range of 50 to 100 vehicles per interval, or at an average hourly volume of 900 vph (i.e., about 57 percent of maximum observed flow). In contrast, HCM speed-flow curves start to exhibit a substantial reduction in speed at a V/C ratio of approximately 0.80. Thus it appears that a lane closure will result in a greater speed reduction for a given volume or V/C ratio than would be expected on the full cross section.

At Site 3, forced-flow conditions prevailed as speeds were clustered at the bottom half of Figure 3.

A final observation in Figure 4 is the discontinuity in the speed-flow lines from site to site.

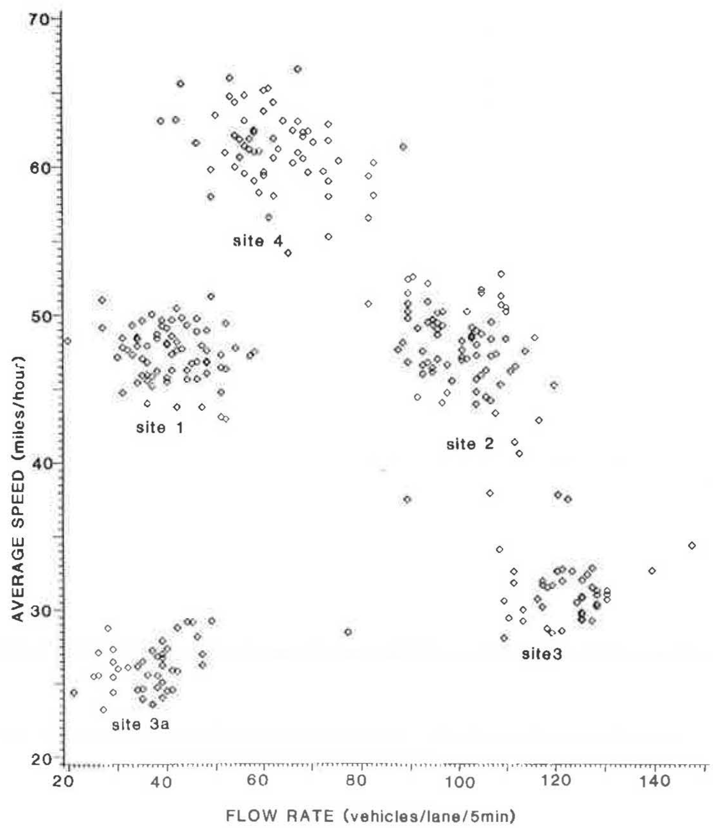

FIGURE 3 Observed speed-flow data.

TABLE 4 Derived Linear Speed-Flow Models Near Work Area

Site	Range of 5-Min Flow Rates Observed	Intercept	Slope	Correlation Coefficient Level of Significance
I-57	34-58	49.04	−0.018	0.85
I-80	88-122	72.50	−0.24	<0.01[a]
I-290	109-147	25.90	+0.04	0.23
I-290[b]	26-49	23.16	+0.08	0.11[c]
I-55	39-88	56.72	−0.087	<0.01[a]

[a] Significant at the 1 percent level.
[b] Observations at start of closure under forced-flow conditions.
[c] Marginally significant at the 10 percent level.

For example, Site 1 had the lowest observed volumes, yet the space-mean speed for that site was consistently lower than that observed at Sites 2 and 4. This is precisely the discrepancy that remains to be explained. This requires the normalization of the speed data to account for variations in geometrics, traffic composition, and demand volume. The presence of any significant residual differences in traffic speed after the normalization procedure is carried out is then attributed to the presence (and intensity) of the construction or maintenance activity itself. The procedure is covered in the next section.

Determination of Construction Activity Effect on Speed

Development of Procedure

The following speed model is assumed at a freeway lane closure site:

$$S_{ot} = S_{pt} - S_t \quad (4)$$

where

S_{ot} = observed space-mean speed in time interval (t) at the lane closure area;
S_{pt} = predicted space-mean speed in time interval (t) for given geometrics, traffic, lane width, and clearance restrictions with work activity; and
S_t = speed reduction in time interval (t) due solely to the presence and intensity of the work activity.

The hypothesis tested is whether (a) S_t is indeed significantly different from zero and (b) the degree to which S_t is functionally correlated to the activity index or to one of its components (e.g., PL, NL). Furthermore, the analysis will determine whether S_t is independent of flow rate (i.e., speed-flow curve parallel to HCM curve) and truck occurrence in the traffic stream. The procedure is carried out in four steps:

1. The observed 5-min flow rates are converted into the equivalent service volume in passenger cars per hour per lane (pcph/lane), with proper adjustment factors for trucks (Q_w) and lane width restrictions (W_w) based on capacity studies of lane closures.
2. Estimates of W_w and Q_w are derived from observations of work zone operation in a study by Wang and Abrams (9). In that study, a computed work zone capacity in vph/lane is defined as

$$C_c = 2000\ V_h Q_h \quad (5)$$

where the h subscript refers to adjustment factors taken from the HCM. The observed capacity in the field is denoted as C_o. A stepwise regression analysis with (C_o - C_c) as a dependent variable and a set of independent variables related to lane geometrics,

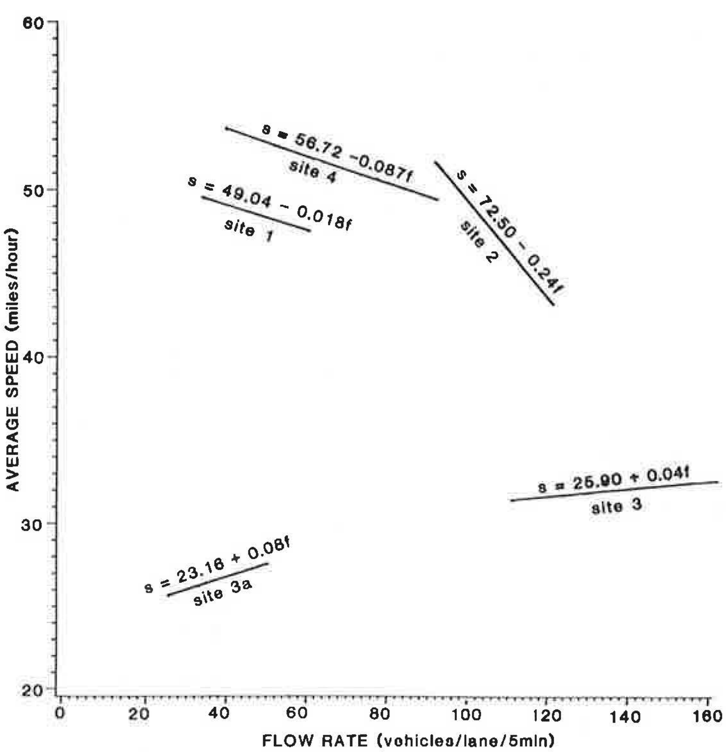

FIGURE 4 Site-specific speed-flow models.

traffic, and type of work activity gave the following equation:

$$C_o - C = 1262 - 228.6N_T - 1230Q_h + 167.4A + 90N_o, \quad R^2 = 0.7 \tag{6}$$

where

N_T = total number of lanes before closure, per direction;
A = work activity type, 1 = long term, 2 = short term; and
N_o = number of open lanes in work zone.

For the sites considered in this study, $N_T = 2$, $A = 1$, and $N = 1$. Equation 6 can be rewritten as

$$C_o = 2000Q_hW_h + 1062.2 = 1230Q_h \tag{7}$$

If a similar definition of C_c is applied for C_o, then

$$C_o = 2000Q_wW_w \tag{8}$$

where the w subscript refer to work zone adjustment factors for trucks, lane width, and lateral clearances. Hence:

$$2000Q_wW_w = 2000Q_hW_h + 1062.2 - 1230Q_h \tag{9}$$

For a stream of traffic consisting entirely of passenger cars, $Q_w = Q_h = 1$ and Equation 9 can be solved for W_w. This gives

$$W_w = W_h - 0.084 \tag{10}$$

It is important to note that the stepwise procedure incorporated the type of channelizing device as an independent variable. Temporary (cones, posts) and permanent devices (concrete barrier) were tested but none was significant for inclusions in the model, except of course for implicit impact on lane width and lateral clearance in the parameters W_w and W_h. Substituting for W_w in Equation 9 and solving for Q_w gives

$$Q_w = 0.531 + Q_h(W_h - 0.615)/(W_h - 0.084) \tag{11}$$

Equations 10 and 11 give the required adjustment factors for lane width restrictions and trucks in single lane closure work zones.

3. The service volume in Step 1 of the procedure is now calculated as

$$SV_t(\text{pcph/l}) = 12f_t/(W_w * Q_w) \tag{12}$$

where f_t is the observed 5-min flow rate in interval t. Subsequently, the predicted space-mean speed in each interval was derived on the basis of the HCM speed-flow curve in TRB Circular 212 (14). The resulting regression equations are

$$S_{pt} = 54.41 - 0.0029SV_t \text{ for } SV_t \leq 1600 \text{ pcph/l} \tag{13}$$

and

$$S_{pt} = 501.9 - 61.24LnSV_t \text{ for } 1600 < SV_t \leq 2000 \text{ pcph/l} \tag{14}$$

4. Substituting into Equation 4, the interval estimates of S_t that reflect speed reduction due to work activity are derived and analyzed.

Results

The data set used in this analysis consisted of all speed-flow observations taken in the vicinity of the work area itself, where the impact of construction activity on traffic flow would be most significant. After discarding observations with missing data or due to equipment malfunction, a total of 103 5-min observations were identified at all four sites. It was found that on the average the observed mean speed at lane closures was 3 mph lower than the predicted speed under the given volume, lane width, clearance, and truck occurrence in the traffic stream from the HCM. Individual differences varied from 10 mph higher to 18 mph lower than the predicted speeds.

A series of t-tests was performed on the difference in mean values for S_t that are associated with various levels of work zone descriptors. A brief summary follows.

Activity Index

The original data set was divided into two subsets. Subset A included all observations with AI $\leq$ 8 and Subset B the remaining observations. As the data in Table 5 indicate, the difference in speeds was found to increase as AI increased; this pattern was consistent at three of the four sites. Overall, however, the difference was less than 1 mph and not statistically significant. It should be noted that the high

TABLE 5 Predicted Difference in Speed[a] (mph) Versus Activity Index (AI)

	Intensity of Construction Activity				
	Low (AI < 8)		High (AI > 8)		
Site	Mean	Standard Deviation	Mean	Standard Deviation	Level of Significance
I-57	N/A	N/A	4.27	2.13	N/A
I-80	−0.317	2.47	2.01	2.56	0.074[b]
I-290	6.44	2.25	10.94	3.95	0.17
I-55	−10.19[c]	N/A	−8.15	2.27	N/A
All sites combined	2.34	5.04	3.03	7.17	0.19
Sites 1, 2, 3	3.17	2.25	5.83	2.76	<0.05

[a] Positive values indicate predicted speed (HCM) greater than observed and vice versa.
[b] Significant at the 10% level.
[c] Only one observation fell into this category.

speeds observed at Site 4 affect Subset B more than A because only one observation at Site 4 fell into Subset A. Omitting Site 4 data, the overall difference in means increases to 2.5 mph and becomes statistically significant at the 5 percent level.

Proximity of Work Activity to Lane of Travel

Observations were categorized as those occurring while construction activity was within 6 ft of the edge of the traveled lane (Subset A, PL > 2) and all remaining observations (Subset B, PL $\leq$ 2). As the data in Table 6 indicate, the predicted difference in speed increased significantly as the work activity moved to within 6 ft of the lane edge. This conclusion held true even after Site 4 data were removed. It is also apparent that because of the precise definition of PL, as opposed to the activity index (AI) that contains a number of subjective

TABLE 6 Predicted Difference In Speed[a] (mph) Versus Activity Location (PL)

	Activity Location with Regard to Travel Lane				
	Close (PL > 2)		Far (PL < 2)		
Site	Mean	Standard Deviation	Mean	Standard Deviation	Level of Significance
I-57	4.43	0.69	2.69	2.18	0.467
I-80	1.66	1.83	1.69	3.04	0.454
I-290	12.27	3.54	7.85	3.54	0.0165[b]
I-55	N/A	N/A[c]	−7.98	2.39	N/A
All sites combined	6.37	5.43	0.563	6.79	<0.001[b]

[a] Positive number indicates predicted speed (HCM) greater than observed speed and vice versa.
[b] Significant at the 1% level.
[c] Site 4 activity physically separated from travel lane by means of portable concrete barrier.

components (e.g., EC, NL, DL), the former parameter is superior in predicting speed changes due to the presence of construction. A stepwise regression model on the original data indicated that with all other variables fixed, a 3-ft shift of construction activity toward the travel lane (i.e., a unit increase in PL) results in an average speed reduction of 2 mph.

5-Min Flow Rate

This test was intended to verify whether drivers in free- and congested-flow conditions react equally to the presence of construction. The original data set was divided into two subsets. Subset A contained flow rates $\leq$ 100 (1,200 vph) and Subset B all the remaining observations. The 100 figure represents a V/C ratio of approximately 0.65 for the observed truck traffic. As the data in Table 7 indicate, the difference in speeds increased substantially as the flow rate past the work site increased. This implies

TABLE 7 Predicted Difference in Speed[a] (mph) Versus 5-Min Flow Rate (TOT)

	Observed Flow Rate				
	Low (TOT < 100)		High (TOT > 100)		
Site	Mean	Standard Deviation	Mean	Standard Deviation	Level of Significance
I-57	4.91	2.14	N/A[b]	N/A	N/A
I-80	1.12	1.96	2.11	2.98	0.2
I-290	22.8[c]	N/A[b]	11.21	5.07	N/A
I-55	−8.93	2.14	N/A[b]	N/A	N/A
All sites combined	−2.51	7.16	7.99	6.22	<0.01[d]

[a] Positive values indicate predicted speed (HCM) greater than observed speed and vice versa.
[b] No observations in category.
[c] Only one observation fell into this category.
[d] Significant at the 1% level.

that the speed-flow curve in a work zone exhibits a steeper slope than its HCM counterpart for a given service volume. The stepwise regression technique discussed earlier was applied for determining the best two-variable formulation. It was found that 52 percent of the variation in speed differences is attributed to flow rates and proximity of work to travel lane. The model form is

$$S_t = -14.17 + 2.07PL_t + 0.14V_t \qquad (15)$$

Equation 15 shows that the impact of flow rates on speed differences is greater when the work activity is within 6 ft of the edge of the lane (PL > 2) at approximately 1,000 vph flow rate (V_t).

Trucks

The original data set was divided into two subsets, one with truck percentages less than 10 percent and the other containing all remaining observations. The results indicated that speed differences increased as the level of trucks increased in the traffic stream. The difference was significant at the 1 percent level. Note that S_t had already been adjusted for trucks; thus the test establishes the additional impact due to the presence of construction.

Others Parameters

The remaining work activity descriptors were not statistically significant in terms of their impact on speed. One interesting exception is the effect of the type of channelizing device. At Site 4, which had a portable concrete barrier at the taper and lane closure areas, speed was virtually independent of work zone activity. It appears that because the barrier effectively separated and visually shielded the construction activity from traffic, the observed speeds consistently exceeded predicted speeds, despite the restrictive lane geometry (width and clearances) at the closure.

SUMMARY AND CONCLUSIONS

This study represents a first attempt at a systematic analysis of freeway traffic flow at single-lane closures, including the effect of work activity interference on observed traffic speed. The following conclusions, based on the limited field observations in this study, are presented:

1. The distribution of traffic speed upstream of the work zone follows a normal distribution when no queuing conditions exist. In the closure area, however, the speed distribution shows significant skewness regardless of the quality of flow upstream of the closure.
2. Speed-flow models at the observed lane closures in this study are considerably different from HCM curves under similar volumes, truck levels, lane width, and lateral clearance restrictions. On the average a difference of 3 mph between HCM and observed speed was noted.
3. The difference in speeds noted in Conclusion 2 is quite sensitive to the location of the work activity. It was found that for every 3-ft shift in construction activity closer to the edge of the traveled lane, a drop of 2 mph in observed speed can be expected.
4. The sensitivity of traffic speed to work zone activity increases as traffic or truck volumes, or both, increase. This may help explain the considerable variations in observed lane capacities at work zones, especially when short counts of 3-6 min are expanded into an hourly flow. In this study, the observed speed at Site 3, which operated at or near capacity in the lane closure section, was on the average 10 mph lower than the corresponding HCM value.

ACKNOWLEDGMENTS

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Operational and Safety Impacts on Freeway Traffic of High-Occupancy Vehicle Lane Construction in a Median

NANA M. KUO and JOHN M. MOUNCE

ABSTRACT

In this paper are presented the results of a study by the Texas Transportation Institute to evaluate the operational and safety impacts associated with the retrofit construction of an authorized high-occupany vehicle lane in the median of the Katy Freeway (I-10W) in Houston, Texas. Because the Katy Freeway transitway is the first of a 70-mi network of transitways to be retrofitted in an existing high-volume freeway cross section in Houston, it is important to assess the traffic impacts associated with this type of construction. Operational impacts studied include travel speeds as a measure of travel time delay, traffic volumes as a measure of travel demand served, and lane distributions as a measure of driver reaction to reduced lane widths and restricted lateral clearances. Safety was assessed through an analysis of reported accidents associated with various work area segments and time periods of construction. Results indicate that a detailed traffic control plan can minimize the possible adverse effects of transitway retrofit construction.

Many of the metropolitan areas in the southwestern United States have experienced unprecedented population growth in the last decade (1). This growth has not only taxed, it has exhausted, the peak-period capacity of the freeway systems within these cities. Degradation of mobility has progressed to critical stages and, in many instances, is threatening future growth and economic vitality. Houston represents an extreme example (2).

Working within both physical and fiscal constraints, transportation officials have turned to an ambitious and innovative plan for highway-transit priority treatment (3). This approach calls for the construction of exclusive, barrier-separated, authorized, high-occupancy vehicle lanes within the medians of existing cross sections of urban freeways. These transitways would provide a high level of service as an incentive for motorists to use such authorized high-occupancy modes as buses, vanpools, and carpools. This plan effectively uses the existing urban transportation infrastructure in a cost-effective manner (4).

The incorporation of a high-occupancy vehicle facility into the median requires special retrofit construction processes that constrain freeway sections already serving high volumes of traffic. Minimizing the adverse traffic impacts associated with this type of construction is a primary concern. Construction on the first such median transitway in Houston was begun in May 1983 on a 5.0-mi section of the Katy Freeway (I-10W) and is scheduled to be completed in October 1984. Because this was the initial effort in a planned 70-mi network of transitways to be constructed in a similar fashion, it is important to measure and understand the operational and safety impacts on mixed-flow traffic resulting from the Katy Freeway transitway project (5). This paper is a report on the study conducted by the Texas Transportation Institute to evaluate the aforementioned impacts and to make recommendations for future project implementation.

PROJECT DESCRIPTION

The Katy Freeway is a major Interstate highway (I-10W) serving travel demands from western Harris County to various parts of Houston (Figure 1). Extensive residential and commercial development has occurred and is continuing to occur along this corridor as far west as Brookshire, a distance of 35 mi from downtown Houston.

Before it was designated I-10W in 1957, the Katy Freeway was known as US-90. Throughout the 1960s much of the Katy Freeway was upgraded to Interstate standards. Today, I-10W is a 10-lane freeway from downtown to Loop I-610. For a short distance of 2 mi (to Antoine) outside I-610, an 8-lane cross section exists. Beyond Antoine, the Katy Freeway is a 6-lane facility.

Traffic volumes on the Katy Freeway increased at an annual rate of approximately 5 percent from 1970 to 1980. Weekday traffic volumes per lane currently approach 25,000. Peak direction flow rates exceed 1,900 vehicles per hour per lane, and travel time delays average 18 min throughout a 4- to 6-hr period each day (6).

Increasing development combines with depressed levels of mobility to justify the need for a high-occupancy priority transportation facility within the Katy Freeway corridor. The Texas State Department of Highways and Public Transportation and the Metropolitan Transit Authority of Houston-Harris County jointly initiated technical and funding efforts to expedite implementation of the Katy Freeway transitway.

The Katy Freeway transitway was designed to be operated in two phases. Construction of Phase 1 began in May 1983 between Post Oak (near I-610) and West Belt, a distance of 5 mi, and will be completed in October 1984. Phase 2 will extend the transitway another 5 mi from West Belt to Highway 6 (Figure 2). Both phases will be constructed in the median of the freeway and will be separated from general traffic lanes by concrete median barriers. The facility will be reversible (operating inbound in the morning and outbound in the evening); will include an emergency breakdown shoulder along most portions; and will be designed to accommodate buses, vanpools, and other authorized high-occupancy vehicles. Typical existing freeway and proposed transitway cross sections are shown in Figure 3.

Construction of the Katy Freeway transitway was combined with the rehabilitation of the freeway pavement to minimize traffic disruption and project cost. The individual segment limits and correspond-

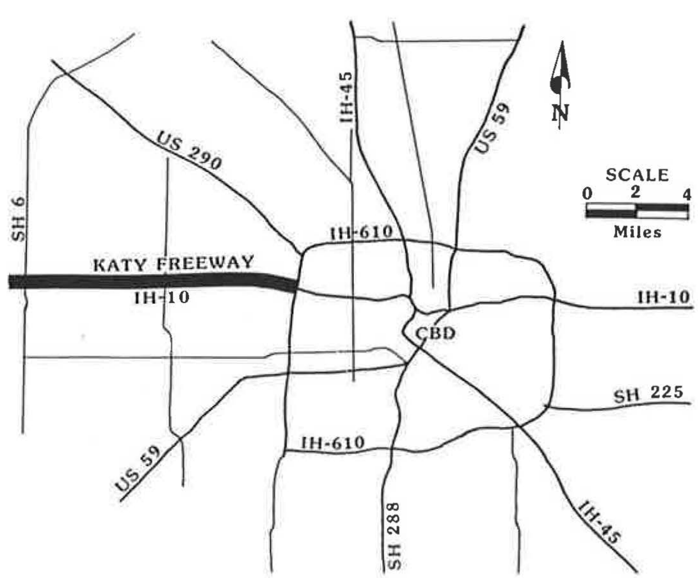

FIGURE 1 Katy Freeway (I-10W) geographic location.

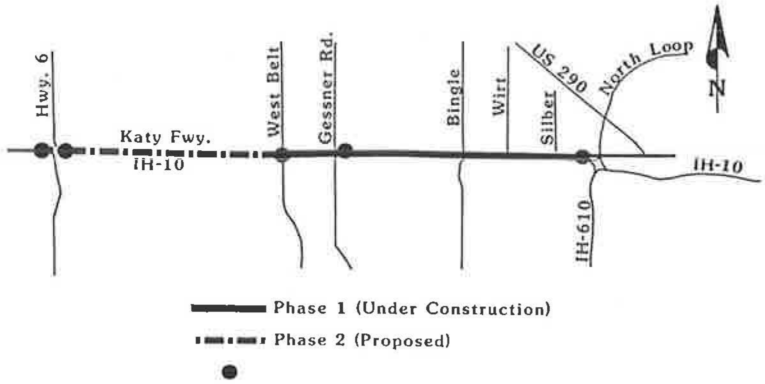

FIGURE 2 Katy transitway construction project phases.

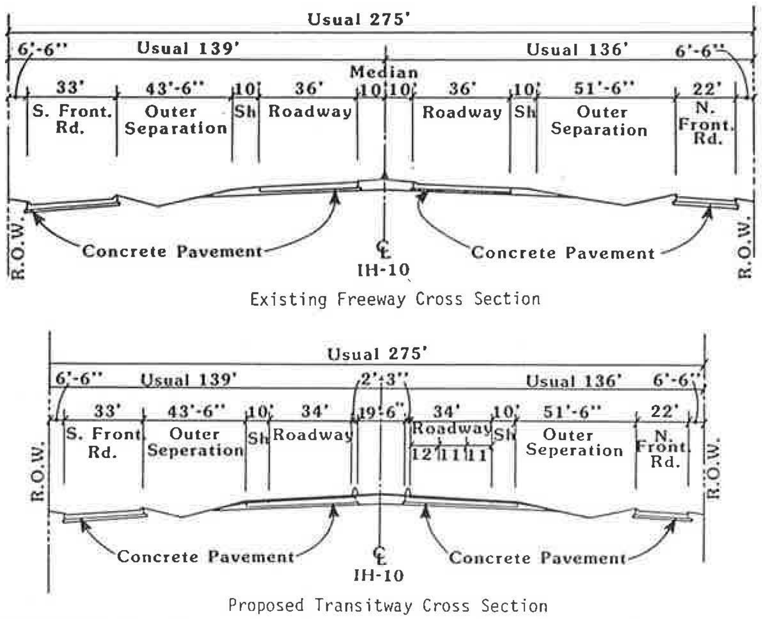

FIGURE 3 Katy Freeway transitway project existing and proposed cross sections.

ing lengths are given in Table 1, as taken from the construction plans (7). Also presented are measured 1981 to 1983 average daily traffic (ADT) for each section (8). Work was independently sequenced by plan and traffic was diverted within each segment as shown in Figure 4. To allow retrofit construction of the transitway, work areas were developed in the median and to the inside and outside areas of the freeway main-lane cross section. Traffic was then routed around the work areas in narrow lanes varying from 10 to 11 ft in width with no shoulders on either the inside or the outside. Temporary concrete median barriers protected and separated the work areas from freeway traffic (Figure 5).

TABLE 1 Construction Segments and Average Traffic Volumes, Katy Transitway Project *(7,8)*

Construction Segments (7)			Average Daily Traffic (8)		
No.	Length (mi)	Limits	1981	1982	1983
1	1.26	West Belt to Gessner	118,000	135,000	136,270
2	1.44	Gessner to Blalock	156,000	167,000	161,090
3	1.95	Blalock to Bingle/Wirt	156,830[a]	161,050[a]	165,270
4	0.89	Bingle/Wirt to Antoine	140,410[a]	143,975[a]	147,540
5	0.83	Antonine to I-610	179,000	186,000	192,190

[a]Estimated—no data available.

DATA COLLECTION METHODOLOGY AND ANALYSIS

The impacts of retrofitting the transitway to the Katy Freeway were categorized as either operational or safety related. Operational measures studied included (a) speeds as a measure of travel time delay, (b) traffic volumes at sites along the length of the Katy Freeway transitway project representative of the various construction segments as a measure of demand served, and (c) lane distributions as a measure of driver reaction to reduced lane widths and lateral clearances. Safety was assessed through an analysis of reported accidents associated with various work segments and time periods of construction.

All operational data were collected manually during both peak periods (morning and evening) and during off-peak periods (midday and nighttime). The

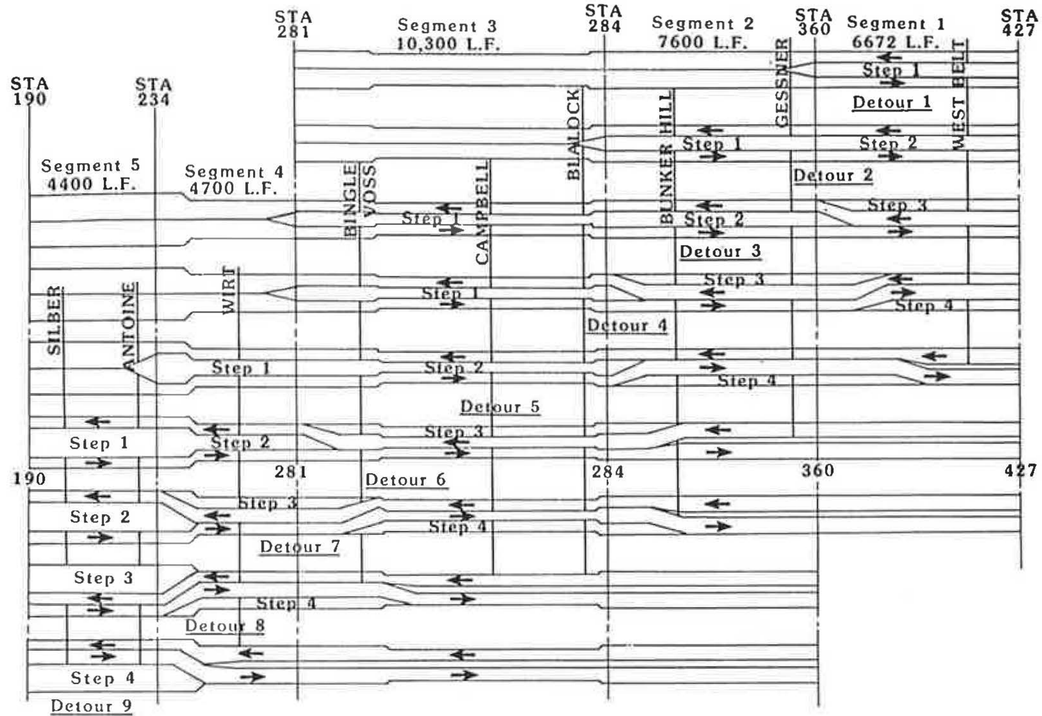

FIGURE 4 Katy Freeway transitway project construction sequence.

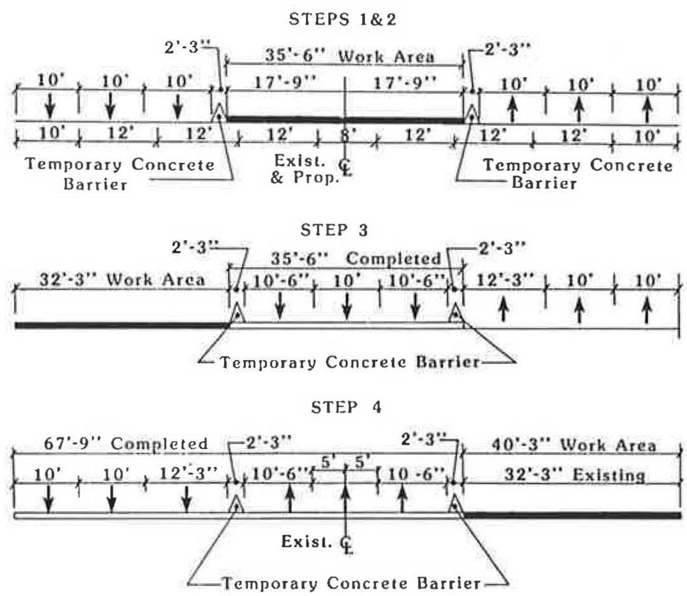

FIGURE 5 Katy Freeway transitway project typical wide area cross sections (looking west).

data were sorted by direction--either eastbound (a.m. peak direction) or westbound (p.m. peak direction). Standard measuring techniques for recording vehicular volumes and speeds were employed. No data were recorded under aberrant operating (accident, breakdown) or environmental (rain, fog) conditions.

The operational and safety data for each segment under construction were compared to the data for each segment 1 year before construction. The changes were then evaluated using a paired t-test. The speed and accident data were compared for identical segments and for equal time periods before and during construction. The chi-square test for independence was applied to the variables associated with free-flow lane volume distribution conditions to determine the statistical significance of the observed by-lane volume distribution changes between full width and narrowed lane cross sections.

The differences between speed profiles before and during construction were tested for statistical significance. Segment 5 in the morning, Segments 2 and 4 in the evening, and the overall peak-hour peak-direction differences between preconstruction and during-construction travel speeds are signifi-

TABLE 2 T-Tests of Differences in Speeds for Preconstruction Versus During-Construction Conditions

	Segment	Size	Difference[a]	Deviation	Error	T	PR>|T|
Morning eastbound	1	8	-1.02	2.58	0.91	-1.12	0.300
	2	5	-1.27	5.82	2.60	-0.49	0.651
	3	3	+1.16	2.71	1.56	0.74	0.534
	4	2	-2.42	6.47	4.58	-0.53	0.690
	5	3	-13.70	2.86	1.65	-8.31	0.014[b]
Overall		21	-2.71	5.90	1.29	-2.11	0.048
Afternoon westbound	1	8	+6.24	11.29	3.99	1.56	0.162
	2	5	+6.02	3.07	1.37	4.38	0.012[b]
	3	3	+2.58	2.80	1.62	1.60	0.251
	4	2	+4.33	0.28	0.20	22.05	0.029[b]
	5	3	+3.95	5.11	2.95	1.34	0.312
Overall		21	+5.16	7.2	1.57	3.28	0.004[b]

[a]During-construction speeds minus preconstruction speeds.
[b]Statistically significant at the 5 percent level.

cant at the 5 percent level (Table 2). However, of these five statistically significant differences, only the morning speed differences indicate a negative impact due to construction. On average, Segment 5 speeds decreased by almost 14 mph with a standard error of 1.65 mph in the morning during construction as opposed to 1 year earlier. Overall, the morning eastbound speeds decreased an average 3 mph with a standard error of 1.29 mph during construction; however, this small decrease is practically nonsignificant.

Average peak-period speeds during the first stages of narrow lane construction were compared to observations made during the later stages of narrow lane construction. Because only two of the segments have undergone more than one construction step, only two of the five segments may be tested. Neither of the differences in operating speed in each segment or throughout the construction length is significant at the 5 percent level (Table 3).

Finally, operating speeds before construction (with full-width lanes plus emergency shoulders) were compared to initial construction operating speeds as well as to later construction operating speeds (both with reduced lane widths and no emergency shoulders). The results are given in Tables 4 and 5. Only one difference in operating speed between preconstruction and beginning-construction speeds was statistically significant at the 5 percent level. The traffic in Segment 5 in the morning eastbound direction experienced an average decrease of more than 15 mph during the first stages of narrow lane

TABLE 3 T-Tests of Differences in Speeds for Beginning- Versus Ending-Construction Conditions

	Segment	Size	Difference[a]	Deviation	Error	T	PR>|T|
Morning eastbound	1	4	1.54	4.89	2.45	0.632	0.575
	2	2	-5.44	17.38	12.29	-0.44	0.735
	3	1	-2.89				
	4	1	-7.20				
	5	1	2.42				
Overall		9	-1.38	7.84	2.61	-0.53	0.612
Afternoon westbound	1	4	3.12	15.60	2.80	1.11	0.346
	2	2	3.73	5.15	3.64	1.02	0.492
	3	1	1.73				
	4	1	-0.73				
	5	1	-0.32				
Overall		9	2.29	4.24	1.41	1.62	0.143[b]

[a]Ending-construction speeds minus beginning-construction speeds.
[b]Statistically significant at the 5 percent level.

TABLE 4 T-Tests of Differences in Speeds for Preconstruction Versus Beginning-Construction Conditions

	Segment	Size	Difference[a]	Deviation	Error	T	PR>|T|
Morning eastbound	1	4	-0.42	3.50	1.75	0.242	0.826
	2	3	-2.36	7.95	14.59	-0.51	0.658
	3	2	2.34	2.53	1.79	1.30	0.417
	4	1	2.15				
	5	2	-15.31	0.84	0.59	-25.81	0.0247[b]
Overall		12	-2.71	7.28	2.10	-1.29	0.224
Afternoon westbound	1	4	4.26	11.71	5.86	0.73	0.519
	2	3	5.20	3.54	2.04	2.55	0.126
	3	2	2.10	3.78	2.67	0.79	0.576
	4	2	1.97	5.36	3.79	0.52	0.694
	5	12	3.78	6.74	1.94	1.94	0.078
Overall		9	2.29	4.24	1.41	1.62	0.143[b]

[a]Beginning-construction speeds minus preconstruction speeds.'
[b]Statistically significant at the 5 percent level.

TABLE 5 T-Tests of Differences in Speeds for Preconstruction Versus Ending-Construction Conditions

	Segment	Size	Difference[a]	Deviation	Error	T	PR>|T|
Morning eastbound	1	4	-1.63	1.55	0.77	-2.10	0.126
	2	2	0.36	0.38	10.27	1.36	0.404
	3	1	-1.18				
	4	1	-7.00				
	5	2	-10.47				
Overall		9	-2.72	3.74	1.25	-2.18	0.061
Afternoon westbound	1	4	8.21	12.25	6.12	1.34	0.272
	2	2	7.26	2.77	1.96	3.70	0.168
	3	3	12.10	3.55			
	4	1	4.13				
	5	9	7.00	7.78	2.59	2.70	0.027[b]
Overall		9	2.29	4.24	1.41	1.62	0.143[b]

[a]Ending-construction speeds minus preconstruction speeds.
[b]Statistically significant at the 5 percent level.

construction. Overall, operating speeds did not change significantly during the initial institution of narrow lane work areas. As for the differences between preconstruction and ending-construction operating speeds, no negative speed differentials were statistically significant at the 5 percent level.

Table 6 gives average measured total volumes at sites with representative construction cross sections for morning, evening, noon, and nighttime periods. Volumes at capacity during morning peak periods approach an average of 1,750 vehicles per hour per lane at the locations sampled. This exceeds the theoretical capacity service volume of 1,680 vehicles per hour per lane for level of service E as calculated to reflect the influence of the geometric restrictions (10.5-ft lane widths, minimal lateral clearances) on basic capacity (9).

TABLE 6 Katy Freeway Transitway Project Observed Work Area Volumes (vehicles/hour/lane)[a]

	Location		
	Campbell (10.5', 10', 10.5')	Wirt (10', 10', 10')	Antoine/Silber (11', 10.5', 10.5')
A.M. peak	1,708	1,686	1,841
P.M. peak	1,689	1,733	1,747
Day off peak	1,350	1,424	1,507
Night off peak	828	796	856

[a]Date of Observation–May 1984.

Table 7 gives a summary of information regarding free-flow vehicle lane distribution sorted by (a) inside, middle, and outside lane; (b) daytime or nighttime period; (c) narrowed or full lane widths; and (d) total and truck-only vehicles. Table 7 also gives the measured sample frequency of vehicles and the calculated chi-square statistical information. All chi-square tests for trucks only as well as for total vehicles indicate that lane distribution is not independent of either time period (day versus night) or cross-sectional width (narrowed versus full) at the 7 percent level. The following effects are noteworthy:

1. During daytime off-peak operation there is little difference in lane distribution of total vehicles. However, there is a shift of approximately 20 percent from the inside lane to the middle lane by trucks within the narrow lane construction cross section over that observed in the full-width cross section.

2. During nighttime operations there is a shift of approximately 13 percent from the inside to the middle lane by total vehicles within the narrow lane construction cross section as opposed to lane distribution in the full-width cross section. There was also a shift of approximately 10 percent from the inside to the outside lane by trucks within the narrow lane construction cross section over a full-width normal cross section.

3. There was little difference in middle lane distribution of trucks between cross sections.

TABLE 7 Katy Freeway Transitway Project Free-Flow Distribution Statistical Analysis

	Day (inside/middle/outside)	Chi-Square DF = 2	Night (inside/middle/outside)
Off-Peak, Hourly Total Vehicle Volumes			
Construction narrowed lanes[a]	2668/3235/2210	$X^2 = 168.72$	1214/2414/1038
	(33%/40%/27%)[b]	$P < 0.0001$	(26%/52%/22%)[b]
Chi-square	$X^2 = 17.55$		$X^2 = 208.59$
DF = 2	$P < 0.0004$		$P < 0.0001$
Normal full-width lanes[c]	1034/1332/1059	$X^2 = 61.87$	1197/1099/763
	(30%/39%/31%)[b]	$P < 0.0001$	(39%/36%/25%)[b]
Off-Peak, Hourly Truck Volumes			
Construction narrowed lanes[a]	87/369/169	$X^2 = 8.12$	27/102/24
	(14%/59%/27%)[d]	$P < 0.0172$	(18%/67%/16%)[d]
Chi-square	$X^2 = 48.86$		$X^2 = 5.32$
DF = 2	$P < 0.0001$		$P < 0.0684$
Normal full-width lanes[c]	75/84/52	$X^2 = 16.84$	20/47/5
	(36%/40%/25%)[d]	$P < 0.0005$	(28%/65%/7%)[d]

[a]Sample location–Campbell, Wirt (May 1984).
[b]Percentage of total vehicle volume.
[c]Sample location–N. Wilcrest (May 1984).
[d]Percentage of total truck volume.

Total accident experience was noted within the limits of the construction project by segment for equal time periods before and during construction. These recorded values were related to segment length as given in Table 1 and to the measured average daily traffic also given in Table 1. This allowed the data to be converted to accident rates (accidents per 100 million vehicle-miles) that lend themselves to statistical analysis for significance of change (10). Tables 8-11 give the impact on safety of the transitway construction as measured by the changes in accident rates. Three changes in accident rates were statistically significant at the 5 percent level. The overall accident rate increased between the preconstruction and the during-construction periods by 49 accidents per 100 million vehicle-miles with a standard error of 22 accidents per 100 million vehicle-miles. The Segment 3 accident rate increased by 80 accidents per 100 million vehicle-miles between the preconstruction and the during-construction time periods with a standard error of 27 accidents per 100 million vehicle-miles. Finally, between the preconstruction and the beginning-construction time periods, the overall accident rate increased by 82 accidents per 100 million vehicle-miles with a standard error of 3 accidents per 100 million vehicle-miles. It is also important to notice one difference that was not statistically signifi-

TABLE 8 T-Tests of Differences in Accident Rates for Preconstruction Versus During-Construction Conditions

Segment	Sample Size	Mean Difference[a]	Standard Deviation	Standard Error	T	PR>|T|
1	9	15.05	66.20	22.07	0.68	0.515
2	7	54.27	133.43	50.43	1.08	0.323
3	5	79.52	60.82	27.20	2.92	0.043[b]
4	3	131.76	110.33	63.70	2.07	0.175
5	6	25.35	204.34	83.42	0.30	0.774
Overall	30	48.68	121.62	22.20	2.19	0.037[b]

[a]During-construction accident rates minus preconstruction accident rates.
[b]Statistically significant at the 5 percent level.

TABLE 9 T-Tests of Differences in Accident Rates for Beginning- Versus Ending-Construction Conditions

Segment	Sample Size	Mean Difference[a]	Standard Deviation	Standard Error	T	PR>|T|
1	5	-14.11	97.03	43.39	-0.33	0.761
2	4	-44.50	78.00	39.00	-1.14	0.337
3	3	-66.60	34.02	19.64	-3.39	0.077
4	2	164.36	34.79	24.60	6.68	0.095
5	3	-71.98	39.07	22.56	-3.19	0.086
Overall	17	19.74	96.06	23.30	-0.85	0.409

[a]Ending-construction accident rates minus beginning-construction accident rates.

TABLE 10 T-Tests of Differences in Accident Rates for Beginning- Versus Ending-Construction Conditions

Segment	Sample Size	Mean Difference[a]	Standard Deviation	Standard Error	T	PR>|T|
1	4	44.75	43.72	21.86	2.05	0.133
2	3	49.37	202.64	117.00	0.42	0.714
3	2	117.64	64.90	45.90	2.56	0.237
4	1	21.14				
5	3	159.45	128.12	73.97	2.16	0.164
Overall	13	81.68	114.71	31.81	2.57	0.025[b]

[a]Beginning-construction accident rates minus ending-construction accident rates.
[b]Statistically significant at the 5 percent level.

TABLE 11 T-Tests of Differences in Accident Rates for Preconstruction Versus Ending-Construction Conditions

Segment	Sample Size	Mean Difference[a]	Standard Deviation	Standard Error	T	PR>|T|
1	5	-8.71	75.79	33.90	-0.26	0.810
2	4	57.94	90.49	45.25	1.28	0.290
3	3	54.10	53.57	30.93	1.75	0.220
4	2	187.08	77.40	54.73	3.42	0.181
5	3	-108.75	184.44	106.49	-1.02	0.415
Overall	17	23.44	124.01	30.08	0.78	0.447

[a]Ending-construction accident rates minus preconstruction accident rates.

cant: the mean difference in accident rates between the preconstruction and the ending-construction time periods was not significant at the 5 percent level.

SUMMARY OF RESULTS

The results of this study suggest the following conclusions:

1. Transitway construction, as instituted with a detailed traffic control plan often involving many ramp closures, has not appreciably decreased operating speeds.
2. The geometric restrictions imposed by transitway construction have not adversely affected freeway volumes to the extent that current highway capacity theory would predict.
3. The institution of narrowed lane cross sections and reduced lateral clearances on the inside and the outside lanes along the transitway construction areas has resulted in a higher percentage of trucks as well as total vehicles using the middle and outer freeway lane.
4. Traffic safety was adversely affected during the beginning of each step in the transitway construction sequence. However, as time passed, drivers were able to adjust to the traffic diversions and highway geometric restrictions that accompanied transitway construction.

RECOMMENDATIONS AND CONCLUSIONS

Retrofitting an HOV facility into the median of an existing freeway is a difficult and potentially hazardous task. In Houston, the narrow lane cross sections and reduced lateral clearances that were instituted along the transitway construction areas raised fears of drastically reduced speeds and volumes and increased accidents. In response, a detailed traffic control plan was developed for the management of freeway main-lane traffic during transitway construction. This traffic control plan has confronted the potential problems and minimized the operational and safety impacts that could have resulted from transitway retrofit construction.

Although the institution of narrowed freeway lanes with little or no lateral clearances does not produce ideal conditions for optimal freeway operation, it does allow the freeway to continue operation during extensive retrofit construction with only minimal operational and safety impacts. Reducing lane widths and using emergency shoulders for through traffic is much preferable to the more traditional strategy of reducing the number of through lanes available to peak-period traffic.

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Field Evaluation of Work Zone Speed Control Techniques

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ABSTRACT

The results of field studies conducted in Texas to evaluate selected methods of slowing work zone traffic to acceptable speeds are presented. The studies were performed at six work zone sites, including two rural freeway sites, one urban freeway site, one urban arterial site, and two rural highway sites. The following work zone speed control methods were studied: flagging, law enforcement, changeable message signs (CMSs), effective lane width reduction, rumble strips, and conventional regulatory and advisory speed signing. The study results indicate that flagging and law enforcement are effective methods for controlling speeds at work zones. The best flagging treatment tested reduced speeds an average of 19 percent for all sites, and the best law enforcement treatment reduced speeds an average of 18 percent. In contrast, the best changeable message sign and effective lane width reduction treatments tested each reduced speeds by only 7 percent. An innovative flagging procedure, a police traffic controller, and a stationary patrol car were found to be the most effective treatments on most highway types. A circulating patrol car and rumble strips were found to be ineffective treatments for controlling work zone speeds. Although conventional regulatory and advisory signing was found to be ineffective in reducing work zone speeds, conventional speed signs are an essential component of any work zone speed control effort.

Speed control through highway work zones has been a topic of concern for several years (1,2). Recent studies have indicated that excessive work zone speeds can adversely affect the safety of the work crew and motorists. For example, in reviewing rural work zone accidents in Ohio, Nemeth and Migletz (3) found that excessive speed was cited 5 1/2 times more frequently than any other accident-producing factor. Richards and Faulkner (4) observed that speed violations contributed to 27 percent of the work zone accidents in Texas compared to 15 percent of non-work zone accidents. Humphreys et al. (2) visited 103 work zones in several states and concluded that unsafe speeds within work zones and ineffective attempts at speed reduction are primary causes of work zone accidents.

In an attempt to control work zone speeds, highway agencies have followed standard signing practices (i.e., posting regulatory or advisory speed signs, or both). However, work zone drivers do not always slow down in response to posted speed limits (1).

Other methods, besides signing, have been tried in an attempt to reduce work zone speeds to the desired level. Table 1 gives some of these techniques that were identified in a recent study (5). However, the overall and relative effectiveness of many of these techniques is unknown.

STUDY DESCRIPTION

Purpose and Scope

A series of field studies was conducted at several work zones in Texas to (a) determine the effectiveness of selected speed control methods in reducing speeds at work zones on different types of highways and (b) evaluate specific selected speed control treatments (5,6).

The studies evaluated the short-term (or immediate) effects of the selected speed control methods. It was not practical within the scope of the research to leave speed control treatments in place for extended time periods so that long-term effects could be studied.

Speed Control Methods

Four principal speed control methods were selected for full-scale field testing: flagging, law enforcement, CMSs, and effective lane width reduction. In addition, rumble strips were tested at two of the sites. Conventional speed signing (regulatory or advisory) was also evaluated as a base condition at all sites.

Study Sites

The studies were conducted at six work zone sites on four types of highways:

1. Undivided multilane arterial (one site)
2. Rural freeway (two sites)
3. Urban freeway (one site)
4. Rural two-lane, two-way highway (six sites)

Table 2 gives the study sites by highway and location and also summarizes prevailing site conditions including type of work activity, location of work, traffic control strategy, traffic volumes, percentage trucks, and posted and prevailing speeds. Construction or major maintenance work was in progress at all of the sites during the studies.

Study Design and Treatments

The study approach was an incomplete factorial design in which several treatments within each speed control approach were tested, but all treatments were not tested at every site. Limitations in equip-

TABLE 1 Work Zone Speed Control Methods

Method	Description
Changeable message signs	Changeable message signs are installed in the work zone to display speed advisories based on real-time conditions.
Overhead flashing signals	Flashing yellow lights are suspended over the travel lanes on span wire with or without accompanying signing to warn of a hazardous condition.
Iowa weave section	Drivers are forced to negotiate a reverse curve created by cones upstream of the hazard area.
Flagging	A flagger, equipped with a paddle or flag, signals traffic to slow.
Pacing	A special pace vehicle leads a line of vehicles through the work area at a reduced speed.
Rumble strips	A series of raised strips is installed upstream of the hazard area.
Transverse striping	A series of colored markings (usually white) is installed on the pavement upstream of the hazard area.
Effective lane width reducing ("funneling")	Lane width is effectively narrowed to create a "funneling" effect.
Law enforcement	Law officers who may issue citations to speeders are deployed at the work zone.
Speed humps and bumps	A hump or bump, designed to jolt vehicles traveling over a desired speed, is placed across the travel lanes.

TABLE 2 Site Summary

Site No.	Location	Type of Roadway	Work Activity	Location of Work Activity	Traffic Control Strategy	Directional Traffic Volume, VPH Average (range)	Trucks (%)	Mean Approach Speed (mph)	Posted Speed[a] (mph)
1	FM 1960 near Houston[b]	Urban 4-lane undivided arterial with continuous left-turn land	Construction of overhead structure	Within normal travel lanes	Detour	800 (650-950)	15	54	35 (R)
2	I-35 near Kyle[b]	Rural 4-lane freeway	Pavement overlay	Left travel lane	Left lane closure	1,150 (850-1,450)	10	60	45 (A)
3	I-35 near Selma[b]	Rural 4-lane freeway	Interchange reconstruction	Off-roadway both sides	Normal travel lanes open	1,000 (850-1,050)	10	56	45 (R)
4	I-10 in Houston[b]	Urban 6-lane freeway	Major reconstruction	Within normal travel lanes	Detour	1,550 (1,300-1,750)	20	60	40 (R)
5	FM 2818 near Bryan	Rural 2-lane highway	Widening to 4 lanes	Off-roadway adjacent to opposing lane	Shoulder use, physical separation from work activity	300 (150-450)	12	52	40 (A)
6	SH 105 near Navasota	Rural 2-lane highway	Widening to 4 lanes	Off-roadway adjacent to travel lane	Physical separation from work activity	125 (100-150)	5-10	56	45 (R)

[a] Advisory speed limit = (A); regulatory speed limit = (R).
[b] Taken from Nemeth and Migletz *(3)*.

ment availability and institutional constraints made it impractical to study all treatments at each site (5). The following treatments were studied:

1. Flagging
   • Manual on Uniform Traffic Control Devices (MUTCD) flagging (7)
   • Innovative flagging (one side)
   • Innovative flagging (both sides)
2. Law enforcement
   • Stationary patrol car
   • Police traffic controller
   • Circulating patrol car
   • Stationary patrol car--lights on
   • Stationary patrol car--radar on
3. Changeable message sign
   • CMS--speed messages only
   • CMS--speed and informational message
   • CMS--speed and informational (alternative location)
4. Effective lane width reduction
   • Lane width reduction--11.5 ft with cones
   • Lane width reduction--12.5 ft with cones
5. Rumble strips--eight strips with decreasing logarithmic spacings

Table 3 gives a summary of these treatments, and Richards et al. (5) give a detailed description and illustration of each one. Table 4 identifies which treatments were studied at each site.

All of the treatments were supplemented by an advisory or regulatory speed sign displaying the desired work zone speed. The signing was included at the request of the highway agency for liability protection. In addition to its legal function, the signing served a critical role in supporting and enhancing the intended speed message of the various treatments. The highway agency established the posted (desired) work zone speed at the sites.

Study Procedure

To perform the studies, one of the treatments was installed, the necessary data were collected, and then the treatment was removed. When the treatment had been completely removed and traffic had returned to normal, another treatment was installed and the procedure was repeated. Treatments were installed in one travel direction only. Allowing time for data collection, each treatment was in place for 1 to 2 hr. In general, two or three treatments, plus a base condition, were evaluated per day at a site. Thus the studies took 3 to 4 days to complete at each site. Studies were conducted only during daylight, off-peak periods when traffic was free flowing.

TABLE 3 Speed Control Treatments Evaluated

Speed Control Method	Treatment	Description
Flagging	MUTCD procedure	Flagger equipped with red flag and orange vest performed "Alert and Slow" signal detailed in Part VI, MUTCD.
	Innovative procedure	MUTCD "Alert and Slow" signal enhanced by two additional movements: (a) flagger motioned traffic to slow with free hand then (b) pointed with free hand to nearby speed sign.
Law enforcement	Stationary patrol car—lights and radar off	Marked patrol car parked on side of road parallel to traffic.
	Stationary patrol car—lights on, radar off	Marked patrol car parked on side of road parallel to traffic with flashing red and blue lights on.
	Stationary patrol car—lights off, radar on	Marked patrol car parked on side of road perpendicular to traffic with radar on and pointed toward traffic stream.
	Circulating patrol car[a]	Marked patrol car continuously driven back and forth through work zone without lights or radar on.
	Police traffic controller	Uniformed officer standing on side of road next to speed sign and manually motioning traffic to slow down.
CMS	Speed and informational message	One- or three-line bulb matrix sign displaying work zone information message plus a speed advisory.
	Speed message only	One- or three-line bulb matrix sign displaying speed advisory.
Effective lane width reduction	Cones (12.5 ft)	On two-lane highways, cones deployed to funnel traffic through a 12.5-ft-wide travel path. On multilane highways, cones positioned along the pavement edges leaving a 12.5-ft travel path between the cones and lane lines.
	Cones (11.5 ft)	Same as above except the travel path width decreased to 11.5 ft.
Conventional signing	Regulatory signing	Black-on-white regulatory speed sign with the desired work zone speed.
	Advisory signing	Black-on-orange advisory speed sign with the desired work zone speed.
Rumble strips[a]	8 strips—decreasing spacing	Eight 1/2-in.-high, polycarbonate strips installed across the travel lane with decreasing spacing, perpendicular to the travel direction.

[a]Tested only on 2-lane highways.

TABLE 4 Summary of Treatments Studied by Site

Treatment	Urban Arterial Site 1 FM 1960	Rural Freeway Site 2 I-35 Kyle	Rural Freeway Site 3 I-35 Selma	Urban Freeway Site 4 I-10	Rural 2-Lane, 2-Way Highway Site 5 FM 2818	Rural 2-Lane, 2-Way Highway Site 6 SH 105
MUTCD flagging	X	X (L)[a]	X	X	X	X
Innovative flagging	X		X	X	X	X
Innovative flagging, both sides		X		X		
Stationary patrol car	X	X (L)	X[b]	X	X	X
Police traffic controller	X				X	X
Circulating patrol car					X	X
Stationary patrol, lights on				X		
Stationary patrol, radar on				X		
CMS speed-only message	X	X (L)		X		
CMS speed and informational message	X	X (L)		X		
CMS speed and advisory at alternate locations				X		
Effective lane width reduction, 11.5 ft	X	X	X	X	X	X
Effective lane width reduction, 12.5 ft	X	X	X	X	X	X
No signing	X		X	X	X	X
Advisory speed signing		X				X
Regulatory speed signing	X		X	X	X	X
Rumble strips					X	X[c]

[a]All treatments were implemented on the right unless noted by (L) indicating left implementation.
[b]Both left- and right-side treatments were studied.
[c]Rumble strips would not adhere to the pavement; thus no data were collected.

Data Collection

Treatment effects on speeds were determined by evaluating speeds at three points (called "speed stations") within the work zone study sites. The first speed station at each site was located upstream and out of sight of any work zone signing or activity. The second station was immediately downstream of where the speed control treatments were implemented. This station measured initial response to the treatments. The third and final station was positioned farther downstream of the treatment location to determine if the treatments suppressed speeds beyond the point of treatment.

For each treatment, 125-vehicle speed samples were collected simultaneously at the three speed stations. Only free-flowing vehicles traveling in the treatment direction were included. Every effort was made to sample unbiasedly and randomly from the total directional flow.

Speeds were determined by measuring vehicle travel times through a marked distance on the roadway (i.e., "trap" section). A 200-ft "trap" length was used. Travel times were manually measured and recorded using digital, electronic stopwatches. This data collection method allowed individual vehicle speeds to be collected to within ±2 mph.

Data Reduction and Analysis

The travel time data, classified by treatment type, speed station, and site, were stored in computer files. Individual travel times were then converted to speeds. Using the MEANS procedure of the Statistical Analysis System (SAS), mean speed and standard deviation statistics were calculated for each station, treatment, and site combination. Speed profiles were developed from the mean speed results, and cumulative frequency distributions were generated for selected treatments at each site.

Treatments were evaluated on the basis of their effectiveness in reducing speeds at Station 2. Relative comparisons among the speed control treatments were made by performing one-way analysis of variance (ANOVA) tests and Duncan's multiple range tests using the ANOVA procedure of SAS.

FIELD STUDY RESULTS

General Results

Figure 1 shows the performance of the various speed control treatments at all six sites. The figure shows the reductions in mean speed (in mph) and percentage speed reduction attained by each treatment on a site-by-site basis. The data in the figure are based on driver responses at Station 2 to the treatments and were generated by comparing mean speeds when a treatment was in place to mean speeds during the base condition. The posted speed at each site is shown in the figure for reference.

Roadway Type

The small number of sites within each roadway category made it difficult to fully assess the influence of roadway type on speed control treatment performance. Figure 1, however, does support some basic trends related to roadway type observed during the studies. In general, the speed control treatments were less effective in reducing speeds at the urban freeway site and more effective at the two-lane, two-way highway and urban arterial sites. From Figure 1, the best treatment at the urban freeway site (Site 4) only reduced the mean speed by 6 mph. However, at the two-lane highway sites (Sites 5 and 6) and the urban arterial site (Site 1), the best treatment reduced the mean speeds by 16, 10, and 13 mph, respectively.

The data, with respect to roadway type, were inconclusive for the rural freeway sites (Sites 2 and 3). At Site 2 the best treatment reduced mean speed by 13 mph, but at Site 3 the best treatment reduced the mean speed by only 7 mph.

Site Differences

It is important to recognize that some of the variation in method and treatment performance was due to individual site differences. However, because the work zones were generally complicated and diverse in character, it is difficult to evaluate what effects site differences had on the results. Nevertheless, Figure 1 provides some evidence of the apparent site effects.

Sites 5 and 6, for example, were both on two-lane, two-way rural highways, and the type of work and traffic control strategy were the same at both sites. As shown in Figure 1, most of the speed control treatments performed significantly better at Site 5. Site characteristics that may have accounted for this better performance are a matter of speculation. Site 5 was nearer to an urban center, and it had more repeat drivers, more turning traffic, more trucks, and straighter alignment than Site 6. The posted speed limit at Site 5 was also lower than at Site 6 (40 versus 45 mph).

Posted Speed

A desired speed limit, selected by the highway agency, was displayed at each site and used as an "anchor" speed for the treatments tested at the

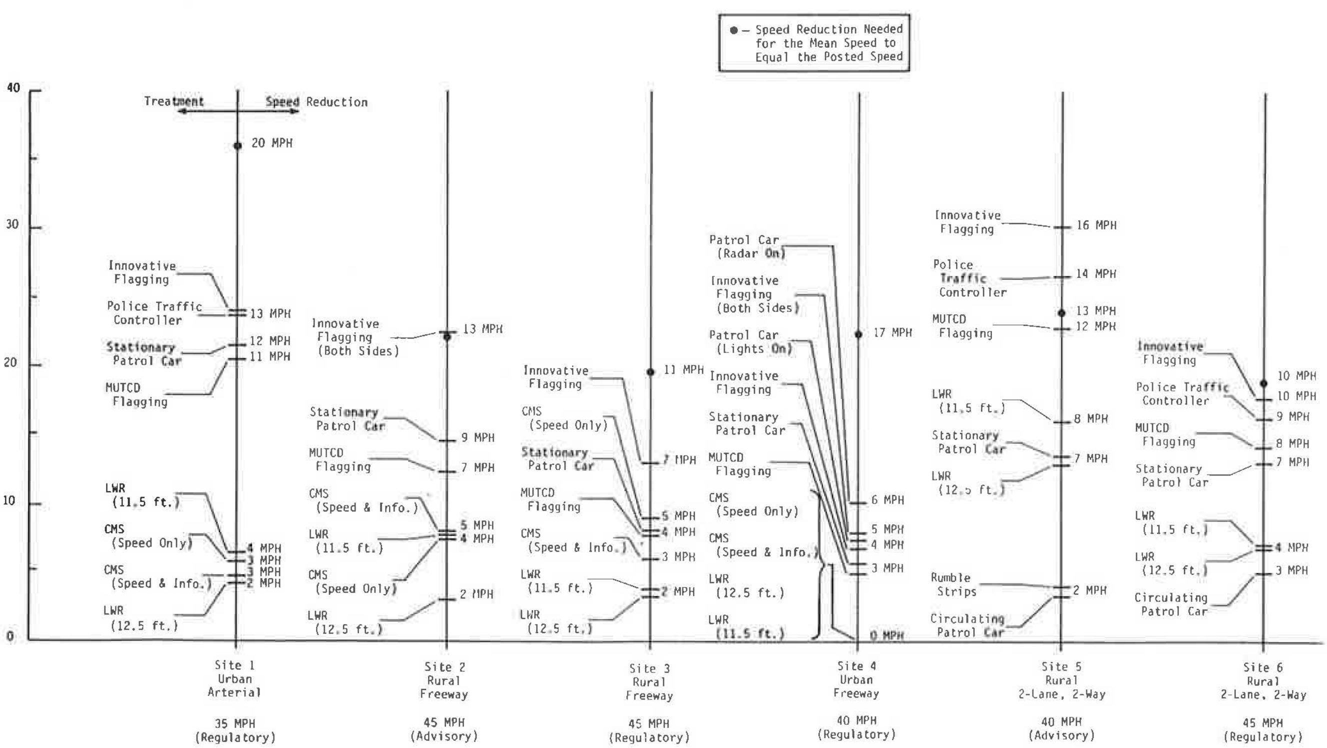


FIGURE 1 Summary of speed control treatments by site.

site. Figure 1 shows the regulatory or advisory speed limit posted at each study site. As seen in the figure, the posted speed limit varied from site to site and ranged from 35 to 45 mph.

In Figure 1, it is seen that none of the treatments tested reduced mean speeds to the posted speed limit at Site 1 (urban arterial), Site 3 (rural freeway), or Site 4 (urban freeway). Apparently the posted speed limit at these sites was simply too low for most drivers to accept under the prevailing site conditions. At the remaining sites, certain treatments did reduce mean speeds to or below the posted speed limit.

Method Performance

Table 5 gives a summary of the relative effectiveness of the four speed control methods in reducing work zone speeds. For each speed control method, the table shows the range and average percentage reduction in mean speeds observed across all sites due to the method. The data in the table are based on drivers' immediate response to the speed control methods (i.e., at Station 2) and on the best treatment within each method on a site-by-site basis.

TABLE 5 Effectiveness of Speed Control Methods[a]

Method	Percentage Speed Reduction[b] Range	Average
Flagging	8-30	19
Law enforcement	8-27	18
Changeable message signs[c]	3-9	7
Lane width reduction with cones	0-16	7

[a]Based on best treatment within each speed control method on a site-by-site basis.
[b]Reduction in mean speed at Station 2 due to speed control method.
[c]No CMS data were available for 2-lane, 2-way rural highways. The average speed reduction shown for CMSs may therefore be misleading (i.e., too low) because all the other speed control methods generally performed better at the 2-lane, 2-way highway sites.

As seen in the table, flagging was the most effective overall method. The best flagging treatment reduced speeds from 8 to 30 percent, depending on the site, and for all sites the best flagging treatments reduced speeds about 19 percent on average.

Law enforcement was also generally effective. The best law enforcement treatments reduced speeds from 8 to 26 percent, depending on the site, and reductions averaged 18 percent for all sites.

CMSs were not tested at the two-lane, two-way highway sites and thus caution should be exercised in comparing the overall performance of CMSs with that of the other methods. At the freeway and urban arterial sites, the best CMS treatments reduced speeds from 3 to 9 percent, and on average they reduced speeds 7 percent.

Effective lane width reduction using cones reduced speeds an average of 7 percent. The effectiveness of this method varied widely by site from no effect at one site up to a 16 percent speed reduction at another. It should be noted that more restrictive treatments than those tested would likely result in larger speed reductions. "More restrictive" refers to the use of narrower lanes or more formidable devices than cones (e.g., barrels or portable barriers, or both).

The effects of the four speed control methods on speed sample variance were analyzed on the basis of standard deviation statistics and cumulative distribution speed plots. The analyses revealed that none of the methods generally altered speed variance. However, certain individual speed control treatments did significantly affect speed sample variance at some sites. The effects of treatment and site on speed variance are discussed in detail in the next section.

Treatment Performance

Flagging Treatments

Table 6 gives a summary of the performance of the various flagging treatments in terms of mean speed reduction and percentage mean speed reduction. The data in the table are based on drivers' responses to the treatments at Station 2. The speed reductions and percentage speed reductions were generated by comparing the mean speed when a treatment was in place to the mean speed during the base (i.e., signing only) condition.

The data in Table 6 indicate that the innovative flagging treatment resulted in larger speed reductions than did standard MUTCD flagging at five of the six study sites. (A direct comparison between the two flagging treatments could not be made at Site 2 because the number of flaggers differed by treatment.) On one of the rural two-lane, two-way highways (Site 5), for example, the innovative flagging treatment reduced the mean speed by 16 mph (30 percent) and MUTCD flagging reduced the mean speed by 12 mph (23 percent). It should be noted that the difference between the innovative and the MUTCD flagging treatments was small at some of the sites. On the urban freeway (Site 4), for example, innovative flagging reduced speeds by 4 mph (7 percent), and MUTCD flagging reduced speeds by 3 mph (5 percent).

The results of the ANOVA and Duncan's multiple range tests indicated that the differences between innovative flagging and MUTCD flagging were statis-

TABLE 6 Performance of Flagging Treatments in Terms of Reduction in Mean Speed at Station 2

	Reduction in Mean Speed (mph)					
	Urban Arterial	Rural Freeway		Urban Freeway	Rural 2-lane Highway	
Flagging Treatment	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Innovative flagging	13 (24)[a]	-[b]	7 (13)	4 (7)	16 (30)	10 (18)
Innovative flagging—both sides	-	13 (22)	-	5 (8)	-	-
MUTCD flagging	11 (20)	7 (12)	4 (8)	3 (5)	12 (23)	8 (14)

[a]Numbers in parentheses indicate percentages.
[b]Not available.

tically significant. The differences were only of the magnitude of about 1 to 4 mph, however, and thus they may or may not be significant from a traffic safety and operational standpoint. Nevertheless, the innovative flagging treatment did produce favorable speed reduction results and allowed the flagger to direct a specific speed message to drivers. MUTCD flagging, on the other hand, displays a more general "alert and slow" message.

The data in Table 6 reveal that the various flagging treatments produced the greatest speed reductions at the two-lane, two-way highway and urban arterial sites. They generally resulted in smaller speed reductions at the freeway sites, particularly the urban freeway site (Site 4). The results suggest that flagging may not be a solution for all situations in which it is desirable to reduce speeds at work zones.

Table 6 does not clearly indicate if flagging effectiveness is improved on freeways by using flaggers on both sides of the travel lanes. At Site 2, innovative flagging on both sides reduced speeds by 13 mph (22 percent), whereas MUTCD flagging on one side reduced speeds by 7 mph (12 percent). These data suggest that using two flaggers may be beneficial; however, they do not allow a direct comparison between one and two flaggers using the same flagging approach.

Law Enforcement Treatments

Table 7 gives a summary of the performance of the various law enforcement treatments. As seen in the table, the police traffic controller treatment was effective in slowing traffic at the three sites where it was tested. At the urban arterial site (Site 1), the police traffic controller reduced mean speeds by 13 mph (26 percent) and at the two-lane, two-way highway sites (Sites 5 and 6), it reduced speeds by 14 and 9 mph (26 and 16 percent). A police traffic controller was not evaluated at any of the freeway sites because the participating police officers were reluctant to stand on the side of the road, away from their vehicles, in the freeway environment. The officers cited two reasons for their reluctance: some were concerned about their personal safety, and others believed that the speed control effort would be unsuccessful and thus an unproductive use of their talent and expertise.

A stationary patrol car was tested at all six sites. This treatment effectively reduced mean speeds between 4 and 12 mph (6 and 22 percent). It was most successful at the urban arterial site (Site 1) and least effective at the urban freeway site (Site 4). At Site 4, a stationary patrol car was evaluated with its lights on and then with its radar in operation. Both of these treatments performed slightly better than a stationary patrol car without lights or radar. The stationary patrol car reduced mean speeds at Site 4 by 3 mph (6 percent). When the patrol car's overhead flashing lights were turned on, the mean speed reduction increased only slightly to 4 mph (8 percent). When the officer turned on a hand-held radar gun and pointed it at passing motorists, the mean speed reduction increased to 6 mph (10 percent).

The circulating patrol car treatment was tested only on the two-lane, two-way highway sites (Sites 5 and 6). It proved to be the least effective of all the law enforcement treatments studied, reducing the mean speed by only 2 mph (3 percent) at Site 5 and 3 mph (5 percent) at Site 6. The circulating patrol car treatment was not evaluated at the other sites because of its relatively poor performance on the two-lane highway sites and because it would likely be even less effective on divided, multilane roadways with limited access points.

The various law enforcement treatments, with one notable exception, did not have much effect on speed sample variance. The stationary patrol car without lights or radar generally reduced speed sample standard deviation by 1 to 2 mph.

CMS Treatments

The performance of the two CMS treatments is summarized in terms of mean speed reductions and percentage mean speed reductions in Table 8. From the table, it is apparent that for a given site both treatments had approximately the same effects on speeds. Depending on the site, the "Speed-Only Message" treatment reduced mean speeds in the range of from 0 to 5 mph (0 to 9 percent), and the "Speed and Information Message" also reduced speeds in the range of from 0 to 5 mph.

The CMS treatments were least effective in slowing drivers at the urban freeway site (Site 4). Neither CMS treatment had any effect on speeds when the CMS was located in the usual treatment location (i.e., near the advance signing for the work zone). However, when the sign was relocated closer to the actual work area, the "Speed and Information Message" treatment reduced Station 2 speeds by 2 mph (3 percent).

Neither of the CMS treatments had a statistically significant effect on speed sample variance.

Effective Lane Width Reduction Treatments

Table 9 gives the performance of the two effective lane width reduction treatments by site and roadway

TABLE 7 Performance of Law Enforcement Treatments in Terms of Reduction in Mean Speed at Station 2

	Reduction in Mean Speed (mph)					
	Urban Arterial	Rural Freeway		Urban Freeway	Rural 2-lane Highway	
Law Enforcement Treatment	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Police traffic controller	13 (24)[a]	–[b]	–	–	14 (26)	9 (16)
Stationary patrol car	12 (22)	9 (15)[c]	5 (8)	3 (6)	7 (14)	7 (13)
Stationary patrol car with lights on	–	–	–	4 (8)	–	–
Stationary patrol car with radar on	–	–	–	6 (10)	–	–
Circulating patrol car	–	–	–	–	2 (3)	3 (5)

[a]Numbers in parentheses indicate percentages.
[b]Not available.
[c]Patrol car on left side of travel lanes.

TABLE 8 Performance of CMS Treatments in Terms of Reduction in Mean Speed at Station 2

CMS Treatment	Reduction in Mean Speed (mph) Urban Arterial Site 1	Rural Freeway Site 2	Rural Freeway Site 3	Urban Freeway Site 4	Rural 2-Lane Highway Site 5	Rural 2-Lane Highway Site 6
Speed-only message	3 (5)[a]	4 (7)	5 (9)	0 (0)	–[b]	–
Speed and information message	3 (5)	5 (8)	3 (6)	2 (3)[c]	–	–

[a]Numbers in parentheses indicate percentages.
[b]Not available.
[c]CMS relocated nearer to the work zones.

TABLE 9 Performance of Effective Lane Width Reduction Treatments in Terms of Reduction in Mean Speed at Station 2

Effective Lane Width Reduction Treatment	Reduction in Mean Speed (mph) Urban Arterial[a] Site 1	Rural Freeway[a] Site 2	Rural Freeway[a] Site 3	Urban Freeway[a] Site 4	Rural 2-Lane Highway[b] Site 5	Rural 2-Lane Highway[b] Site 6
11.5-ft width using cones	4 (5)[c]	5 (8)	2 (4)	0 (0)	8 (16)	4 (7)
12.5-ft width using cones	2 (5)	2 (3)	2 (3)	0 (0)	7 (13)	4 (7)

[a]Cones placed on edges of pavement only.
[b]Cones placed on edge of pavement and centerline.
[c]Numbers in parentheses indicate percentages.

type. The data in the table indicate that the two treatments, for a given site, had approximately the same effect on speeds, with observed speed reductions ranging from 0 to 8 mph (0 to 16 percent) depending on the site. The 11.5-ft lane width treatment resulted in slightly higher speed reductions at three of the six sites compared to the 12.5-ft treatment. However, the differences between treatments were not statistically or practically significant.

It is important to note that the highway agency would not allow cones to be placed on the lane lines at any of the multilane sites (i.e., Sites 1-4) in the interest of safety. Thus effective lane narrowing at these sites was accomplished by placing cones on the edges of the travel lanes. This may explain why the treatments generally did not reduce speeds as much at the multilane sites compared to the two-lane, two-way highway sites. At the two-lane highway sites, lane narrowing was accomplished by placing cones on the edge of the travel lane and on the centerline. Another important finding of the study was that cones proved to be somewhat hazardous devices for reducing lane widths to less than 12 ft.

The effective lane width reduction treatments had some interesting effects on speed sample variance. At every site except Site 6, the 11.5-ft treatment resulted in a larger speed sample standard deviation than the 12.5-ft treatment. At Site 6, the two treatments resulted in about the same standard deviation.

The studies also revealed that when a treatment was effective in slowing traffic at a site, it also produced a higher speed sample variance. For example, the 11.5-ft treatment produced an 8 mph (16 percent) reduction in mean speed at Site 5 but also increased the standard deviation of the speed sample by 2.4 mph. At Site 4, the 11.5-ft treatment had no effect on the mean speed, and the standard deviation actually decreased by 0.5 mph (i.e., the treatment had no significant effect on variance).

Work Area Speeds

Speed data were collected at the study sites downstream of the treatment location to measure the effects of the various speed control treatments on traffic within the work area. The downstream station (Station 3) was positioned 1/3 to 1/2 mi downstream of the treatment location near the work activity.

The data from Station 3 were combined with data from the upstream stations to generate speed profiles for each site. The profiles illustrate the effects of the speed control treatments upstream of and entering the work area. As an example, Figure 2 shows speed profiles for selected treatments at the urban arterial site (Site 1).

Figure 2 illustrates two important findings of the studies. First, after being exposed to a particular speed control treatment, drivers continued slowing down or at least maintained a reduced speed as they approached and entered the work area. In other words, drivers did not return to their normal speed immediately after passing the speed control treatment.

Second, most of the treatments (and especially innovative flagging and a stationary patrol car in Figure 2) reduced work area entry speeds well below normal or base entry speeds. Thus the treatments encouraged drivers to slow down much more than they would have simply in response to sighting the work activity. For example, the mean work area entry speed at Site 1 was 50 mph under base (i.e., signing-only) conditions. The innovative flagging treatment reduced the mean entry speed to 39 mph, and the stationary patrol car treatment reduced the mean entry speed to 41 mph. The 11.5-ft effective lane

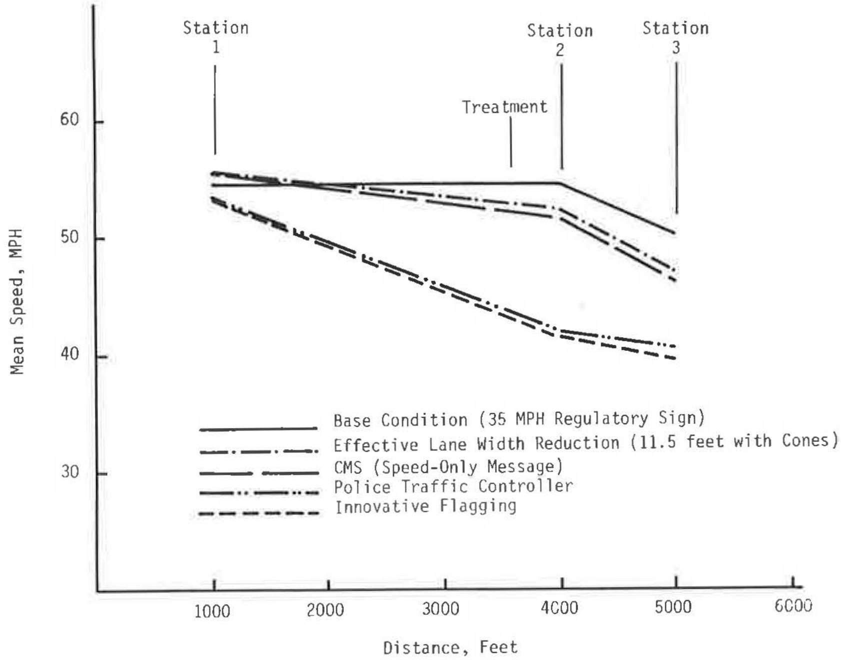


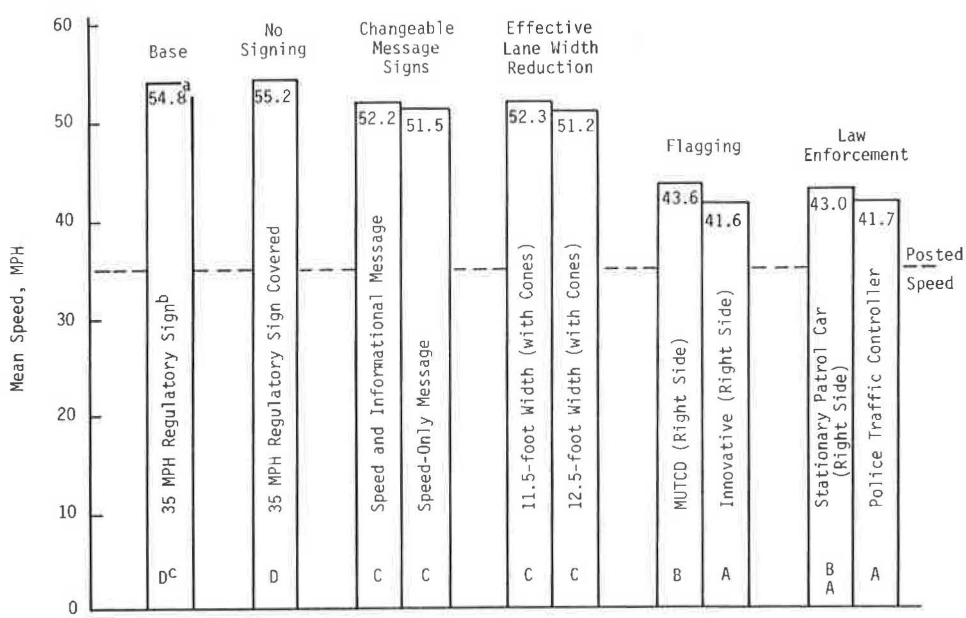
FIGURE 2 Speed profiles of selected treatments at Site 1 (urban arterial).

width reduction treatment and "Speed-Only Message" CMS treatment reduced the mean work area speeds to 46 and 47 mph, respectively.

Statistical Significance

Figures 3-8 are bar charts that summarize the mean speed data from Station 2 at each site. The figures indicate which treatments produced statistically different speeds based on the results of Duncan's multiple range tests. As seen in the figures, many of the treatments were statistically different. Because of the large sample sizes and consistent variances, however, mean speed differences of as little as 1 to 2 mph were found to be statistically significant.

From a practical standpoint, a 1- to 2-mph mean speed difference may not be significant because such a small speed difference would likely have no mea-

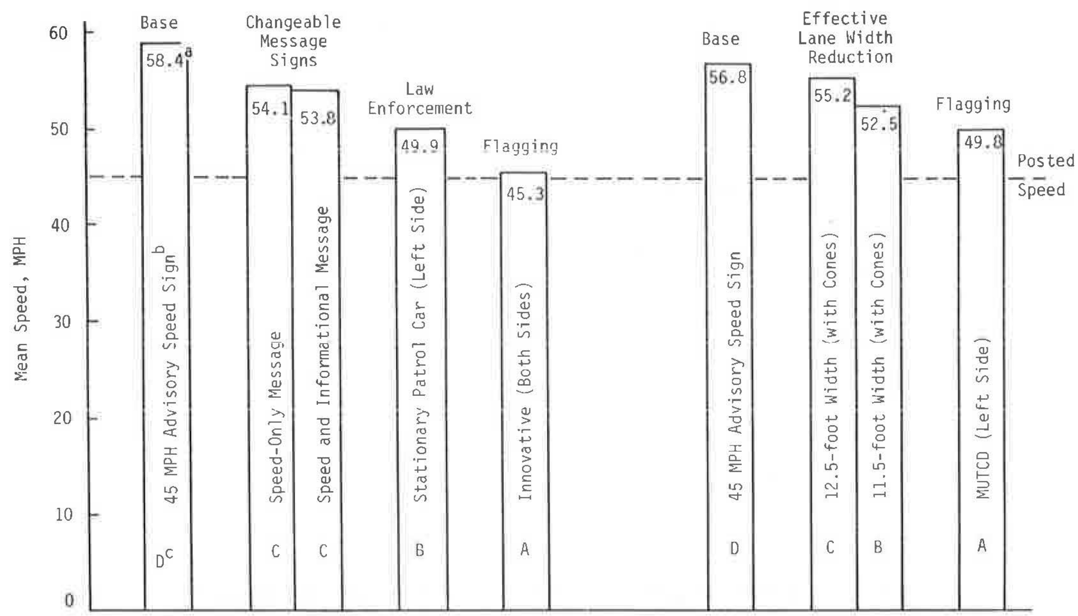


[a] Mean speed at Station 2 in miles per hour.

[b] Speed control treatment.

[c] There is no statistically significant mean speed difference between treatments with the same letter, based on Duncan's Multiple Range Tests.

FIGURE 3 Comparison of speed control treatment means at Site 1 (urban arterial).

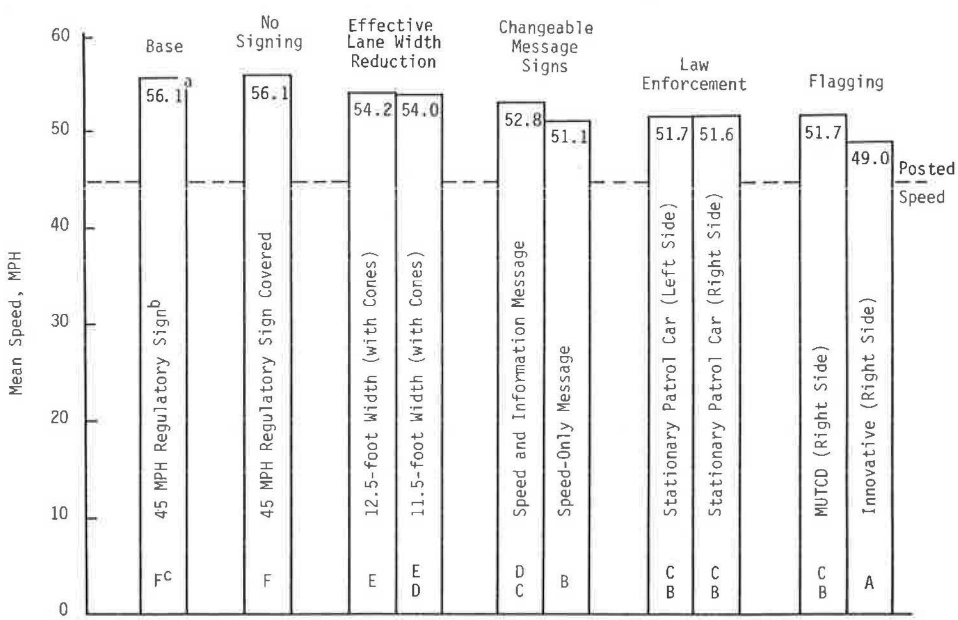


[a]Mean Speed at Station 2 in miles per hour.

[b]Speed control treatment.

[c]There is no statistically significant mean speed difference between treatments with the same letter, based on Duncan's Multiple Range Tests.

FIGURE 4 Comparison of speed control treatment means at Site 2 (rural freeway).

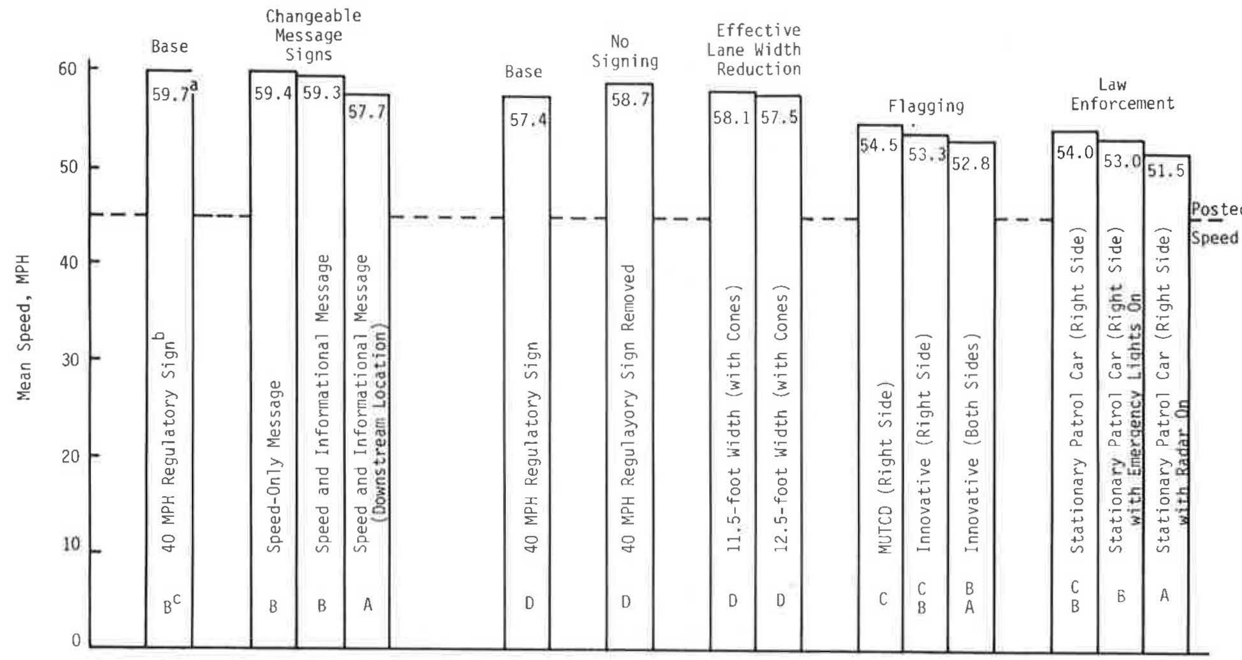


[a] Mean speed at Station 2 in miles per hour.

[b] Speed control treatment.

[c] There is no statistically significant mean speed difference between treatments with the same letter, based on Duncan's Multiple Range Tests.

FIGURE 5 Comparison of speed control treatment means at Site 3 (rural freeway).

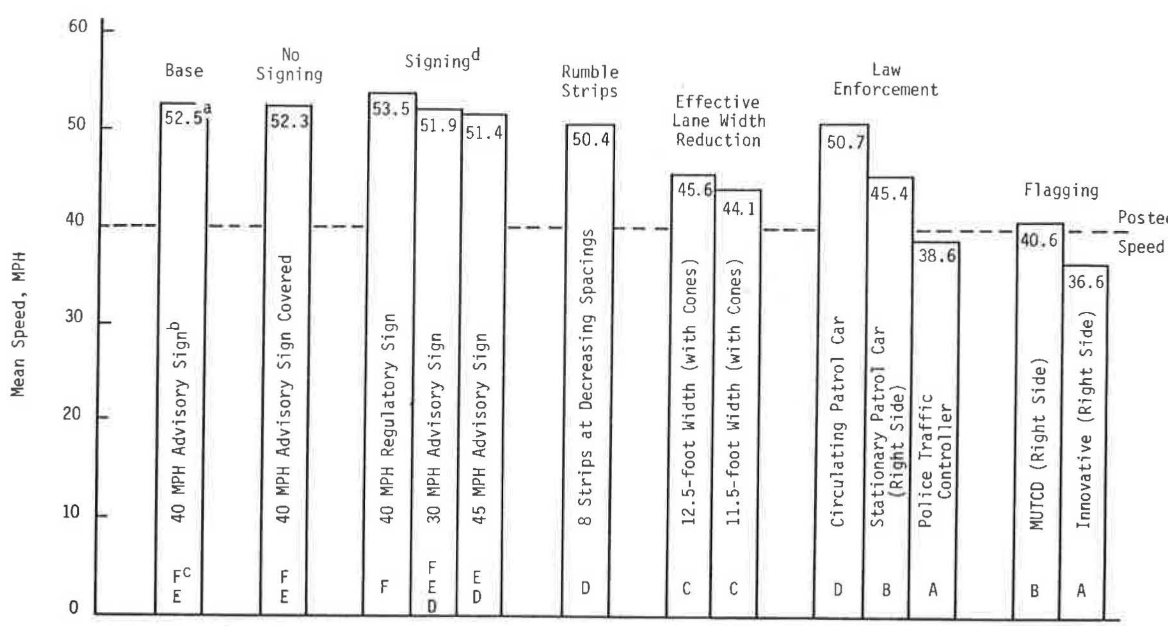


[a]Mean speed at Station 2 in miles per hour.

[b]Speed control Treatment.

[c]There is no statistically significant mean speed difference between treatments with the same letter, based on Duncan's Multiple Range Tests.

FIGURE 6 Comparison of speed control treatment means at Site 4 (urban freeway).



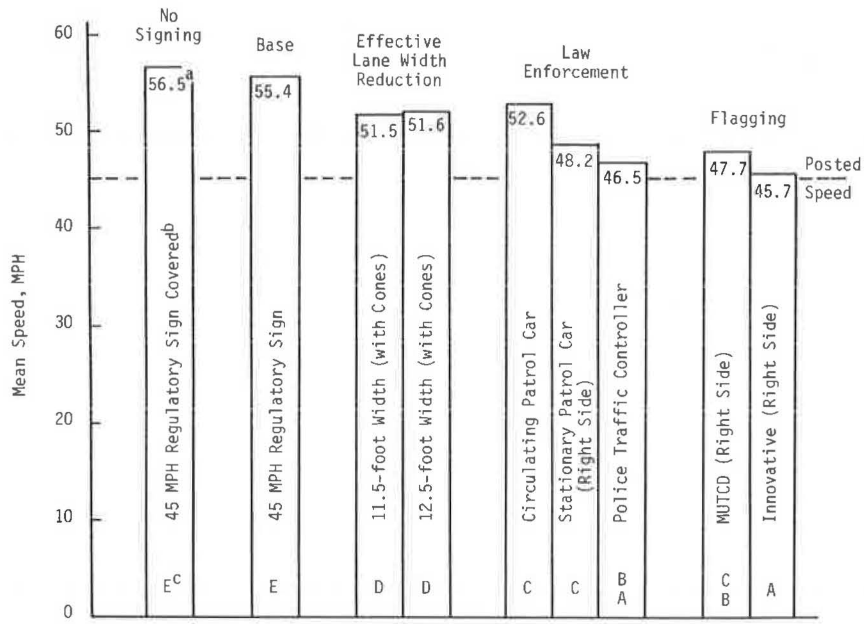
[a] Mean speed at 2 in miles per hour.

[b] Speed control treatment.

[c] There is no statistically significant mean speed difference between treatments with the same letter, based on Duncan's Multiple Range Tests.

[d] 40 MPH Advisory Sign (Base Condition Sign) was removed.

FIGURE 7 Comparison of speed control treatment means at Site 5 (rural two-lane, two-way highway).



[a] Mean speed at Station 2 in miles per hour.

[b] Speed control treatment.

[c] There is no statistically significant mean speed difference between treatments with the same letter, based on Duncan's Multiple Range Tests.

FIGURE 8 Comparison of speed control treatment means at Site 6 (rural two-lane, two-way highway).

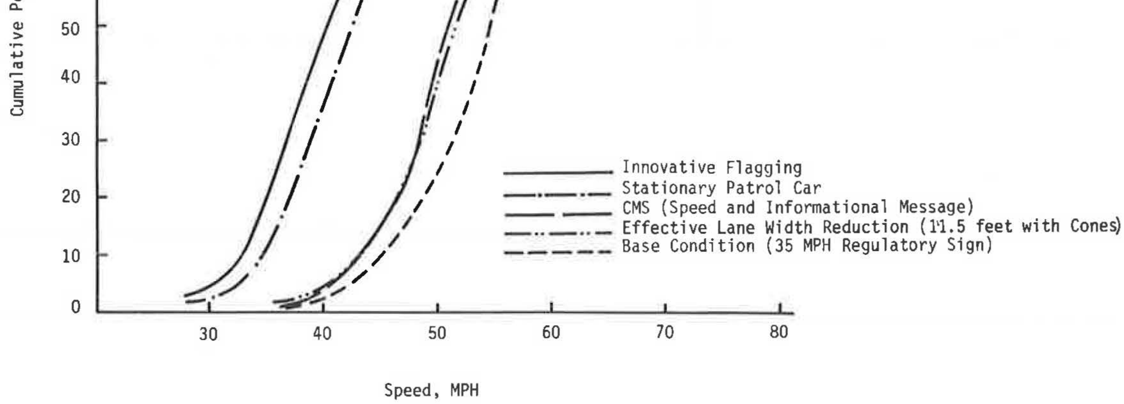


FIGURE 9 Cumulative speed distributions of selected speed control treatments at Site 1 (urban freeway).

surable effects on safety or traffic operations. Mean speeds would probably have to differ by 4 mph or more to support a contention that one treatment was truly better than another. However, this is merely speculation.

Speed Distributions

Figure 9 shows cumulative distribution plots of Station 2 speed data for selected treatments tested at the urban arterial site (Site 1). Included in the figure is a cumulative distribution plot for the base (i.e., signing-only) condition. From the figure it can be seen that certain of the speed control treatments significantly shifted the speed distribution to the left of the base curve. This indicates that these treatments lowered speeds in general (i.e., both fast and slow drivers responded to the speed control treatment). Most notably in Figure 9, the innovative flagging and stationary patrol car treatments shifted the speed distribution at Site 1.

It is also important to observe in Figure 9 that all of the distribution curves shown in the figure have approximately the same shape. This is further evidence that the treatments did not generally affect speed variance.

Safety Performance

As well as taking speed measurements, field personnel observed and recorded erratic maneuvers and other evidence of safety problems. None of the treatments resulted in any accidents or recurring safety problems at any site. Only a few minor incidences were witnessed during the studies:

1. At one of the two-lane, two-way highway sites (Site 5) the flagger was at times too zealous and aggressive in using the innovative flagging procedure. As a result, a few drivers (i.e., three or four in a 1-hr period) overreacted and slowed excessively. One driver even pulled onto the shoulder thinking that he was supposed to stop. These problems were avoided at the remaining sites simply by exercising proper flagging techniques.
2. At the two-lane, two-way highway sites (Sites 5 and 6) effective lane width reduction was accomplished by placing cones on the pavement edge and centerline. When the 11.5-ft treatment was implemented at these sites, cones were hit or blown out of place on several occasions. On one occasion several cones were hit by a truck and knocked into the travel lane. Rather than running over the displaced cones, a motorist stopped in a lane and got out of his vehicle to move the cones. Several other vehicles in turn were forced to stop and wait for the motorist to move his car. In another incident, a wide mobile home passed through the narrow lane section and took out several cones.
3. At the freeway sites large trucks tended to "straddle" the lane line within the narrow lane section if other traffic was not present.

CONCLUSIONS

The study results, because of the relatively small number of study sites and the incomplete factorial design, do not answer all the questions concerning the treatments tested, nor do they address all of the critical issues relating to work zone speed control. The results are significant, however, because they indicate that certain methods can be used at some work zones to effectively slow drivers, which has positive effects on safety. Furthermore, the study results provide insight into the factors that influence work zone speeds and motorist response to speed control techniques.

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Abridgment

Selection of Work Zone Channelizing Devices Using the Value Engineering Approach

STEPHEN H. RICHARDS and CONRAD L. DUDEK

ABSTRACT

The use of value engineering for selecting work zone channelizing devices is investigated. For illustration, the approach is used to select devices for a lane closure taper at a rural freeway work zone. The results of the investigation indicate that value engineering can be a useful and practical work zone traffic management tool. It provides an objective means of evaluating any number of alternative channelizing devices using whatever performance and cost data are available. Most important, it encourages the selection of low-cost devices that are safe and effective under the prevailing work zone conditions.

There is a wide variety of channelizing devices currently available for use in highway work zones. The Manual on Uniform Traffic Control Devices (MUTCD) (1) presents basic design standards for these devices and general guidelines for their use; however, it is left up to the highway agency to decide where and when to use particular devices or sets of devices.

Typically, work zone channelizing devices are chosen on the basis of one of the following practices:

1. Select the device with the lowest initial cost,
2. Select a device that is normally used by the agency,
3. Select a device already in stock, or
4. Select the "very best" device just in case.

Each of these approaches has drawbacks, and collectively they have resulted in inflated job costs, unnecessarily large inventories, lack of uniformity, and, in some cases, improper device use.

VALUE ENGINEERING APPROACH

The selection of the most appropriate channelizing device for a work zone situation is a critical task. It requires an objective consideration of several factors including cost, safety, maintainability, availability, uniformity, project life, and work zone conditions. Because there is currently no widely accepted, objective means for selecting work zone channelizing devices, the need for a proven approach like value engineering is well founded.

Value engineering is a formalized problem-solving approach directed at analyzing the function of an item with the purpose of achieving the required function at the lowest overall cost (2). Two features of value engineering set it apart from other formal problem-solving techniques. First, it is concerned with function (i.e., identifying the desired function of an item or service). Second, it attempts to establish the relative value of alternatives for accomplishing a function.

The relationship between value (or worth as it is often called) and function is expressed in the following equation (3):

Value = Functional performance/Cost

From this equation, it is seen that value may be increased by (a) reducing costs, if performance is maintained or (b) increasing performance, but only if increased performance is needed and wanted and the user is willing to pay for it, or both (a) and (b).

The intent of value engineering is to find solutions that achieve the required function at the lowest overall cost. Value engineering does not strive to save dollars; dollar savings are automatic and maximum (4). In emphasizing function, value engineering lessens the chance that existing hardware limitations or established practices will confine creative thinking. Thus value engineering promotes objective and innovative problem solutions (2).

APPLICATION OF VALUE ENGINEERING

The selection of work zone channelizing devices using the value engineering approach involves 7 steps:

1. Determine the intended purpose (function) of the devices.
2. Identify available alternative devices.
3. Select appropriate measures of device performance (i.e., a means of evaluating how well a device performs its intended function).
4. Determine the performance of the alternative devices on the basis of selected performance measures. (If it has not already been done, alternatives that do not meet minimum performance criteria should be excluded.)
5. Estimate the total cost of each acceptable alternative.
6. Calculate the relative value of each acceptable alternative, where value = performance/cost.
7. Select the alternative with the greatest value.

Instead of describing each of these steps, the following sections will demonstrate how they are performed in selecting taper devices for a lane closure work zone. For the purpose of the illustration, data from NCHRP Report 236 (5) are used.

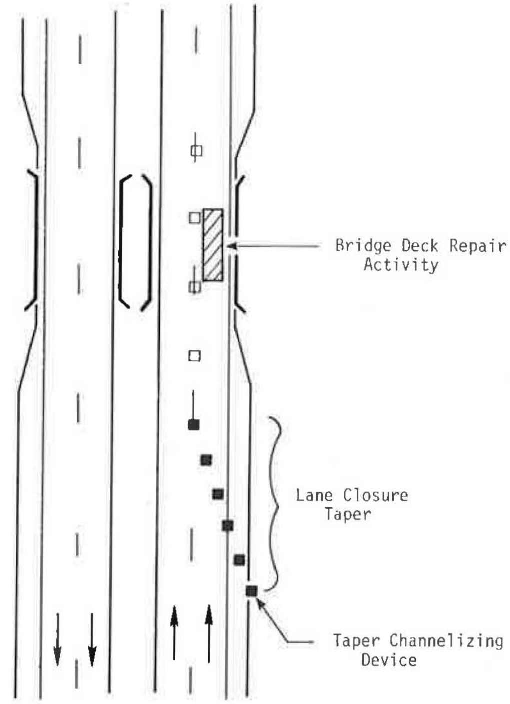

FIGURE 1 Freeway work zone site layout.

WORK ZONE SCENARIO

Bridge deck repair work is planned for the northbound, right lane of a rural freeway (Figure 1). The four-lane divided freeway carries low traffic volumes, and speeds are generally high (e.g., 55 mph). In the area of the work, sight distance is excellent and there are no ramps.

The value engineering approach will be used to select the channelizing devices for the lane closure taper. It is anticipated that the right lane will be closed, day and night, for approximately 2 weeks. It is also assumed that the minimum taper length and maximum device spacings recommended in the MUTCD will be used and that only one type of channelizing device will be used in the taper.

FUNCTION OF LANE CLOSURE TAPER

The first step in the value engineering analysis is to identify function. To accomplish this, a functional analysis of the channelizing devices used in a freeway lane closure taper was performed by a team consisting of five traffic engineers and one human factors engineer. The team first identified the various functions performed by channelizing devices in a lane closure taper and then categorized them as either basic or secondary functions.

A FAST diagram, based on the team input, for taper devices was developed. The FAST diagram, shown in Figure 2, indicates that the most basic function of channelizing devices in a lane closure taper is to display color or light, or both, to approaching motorists. The pattern of color or light identifies the closure, defines the workspace, and identifies the travel path.

AVAILABLE DEVICES

After device function has been assessed, the next step in the value engineering analysis is to identify alternative channelizing devices that perform the required functions. Data from NCHRP Report 236 (5) provide a basis for selecting appropriate candidate devices. From this report, the following six types of channelizing devices were selected as viable alternatives for a freeway lane closure taper:

1. 3-ft x 12-in. Type I barricades,
2. 3-ft x 12-in. Type II barricades,
3. 12-in. x 36-in. vertical panels,
4. 36-in. cones with reflective strips,
5. 42-in. tubes with reflective strips, and
6. 55-gal. drums with reflective strips.

It is recognized that there are many other possible alternatives, but these six serve the purpose of this illustration.

MEASURE OF PERFORMANCE

The third step is to identify appropriate measures of performance for the alternative devices. From NCHRP Report 236 (5), two measures of performance were selected:

1. Mean array (taper) detection distance and
2. Mean location of lane change relative to the beginning of the taper.

These performance measures were selected because

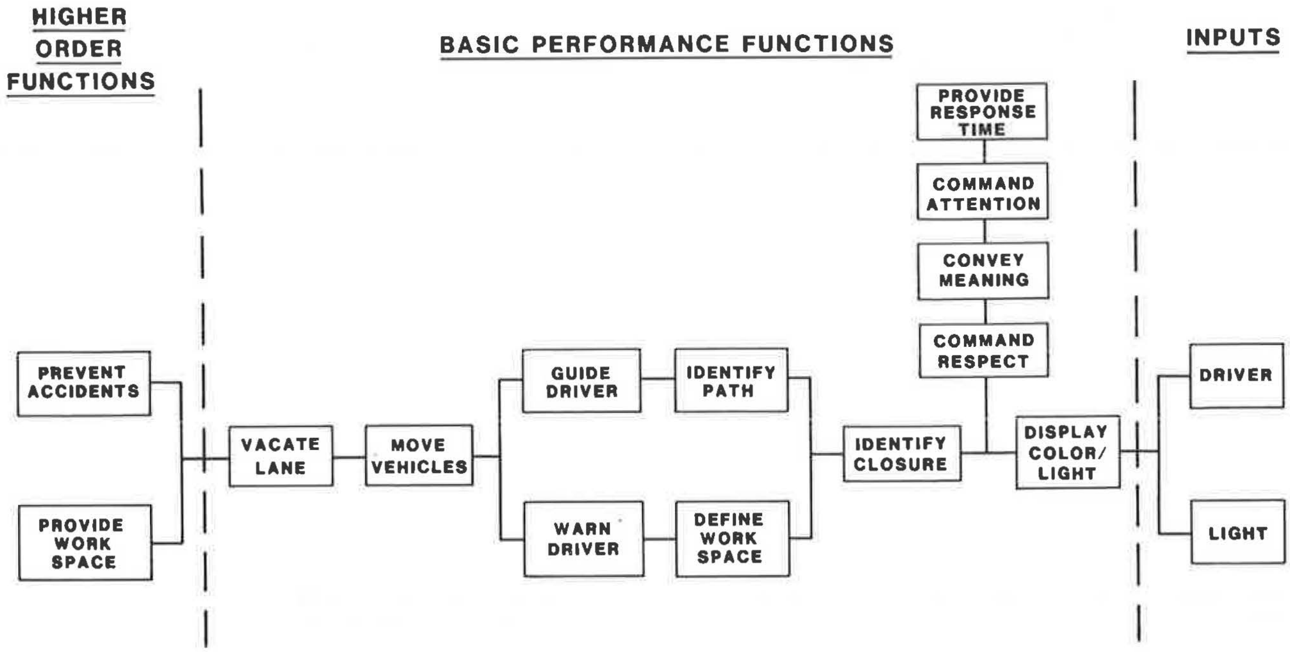

FIGURE 2 FAST diagram illustrating the basic functions of lane closure taper channelizing devices.

they correlate with the basic functions of taper channelizing devices (Figure 2) and because there are corresponding performance data available for each alternative device. With respect to the detection distance measure, there also are data available that provide a basis for establishing a minimum level of performance (6).

In addition to performing the basic channelization functions, it is essential that the alternatives perform required secondary functions. Secondary functions might include maintainability, durability, ease in placement and removal, sign support, and the like. It is assumed that the six alternative devices satisfy all the desired secondary functions at the freeway lane closure site.

DEVICE PERFORMANCE

After appropriate measures of performance have been selected, the next step is to determine device performance based on the established measures. Table 1 gives the mean array detection distance and the mean lane change location for each of the candidate device arrays under day and night conditions. These performance data were extracted from the NCHRP Report 236.

With respect to array detection distance, research

TABLE 1 Mean Detection Distances and Lane Closure Locations for Alternative Channelizing Devices *(5)*

	Mean Array Detection Distance (ft)		Mean Lane Change Location Relative to Taper (ft)	
Device Type	Day	Night	Day	Night
Type I barricades	4,250	3,150	640	660
Type II barricades	4,100	2,800	400	810
Vertical panels	4,400	3,300	370	500
Cones	4,400	1,450	460	250
Tubes	3,200	1,900	620	350
Drums	4,200	3,000[a]	540	560

[a]Estimate based on supplemental research by Texas Transportation Insitute.

by Richards and Dudek (6) suggests that the minimum detection distance for a freeway lane closure taper should be 1,000 ft. Types I and II barricades, vertical panels, and drums provide detection distances that greatly exceed this minimum value. However, based on the NCHRP studies, cones and tubes may not fully satisfy the minimum detection distance requirement at night. This limitation should be recognized and considered in selecting an appropriate channelizing device for the 2-week freeway work zone.

A basic assumption in this value engineering analysis is that channelizing device performance has no upper limiting values. In other words, all of the detection distance and lane change distance provided by a device is useful and therefore has value. At other work zones (e.g., on a minor city street or where sight distance is physically limited by geometric features), it might be desirable to establish upper performance limits. For example, devices on a city street may only need to be detected from a distance of 1,000 ft. Any detection distance greater than 1,000 ft provided by a device would not be used and should not be considered in computing value.

DEVICE COSTS

Cost data for the alternative devices were obtained from a traffic control device supplier in Texas. These data were used to generate the relative device costs given in Table 2. It should be noted that the costs in Table 2 were developed on the basis of some simple and general assumptions. They serve the purpose of illustrating the value engineering approach but should not be considered as truly accurate cost estimates.

DEVICE VALUE

Table 3 gives a value summary for the alternative devices. The table shows the relative value of each alternative device based on its ability to provide detection distance and encourage early lane changes under day and night conditions. The values in Table 3 were computed on the basis of the performance data

TABLE 2 Device Cost

Device Type	Cost Per Device ($)
Type I barricades	40
Type II barricades	45
Vertical panels[a]	22
Cones[b,c]	18
Tubes[d,c]	22
Drums[d]	25

[a]Portable vertical panel mounted on stand.
[b]4-in. reflective collar added.
[c]Cost based on 50 percent replacement during 2-week duration.
[d]Two 4-in. reflective collars added.

TABLE 3 Relative Value of Alternative Devices

	Device Cost per 100 ft of Array Detection Distance ($)		Device Cost per 100 ft of Lane Change Distance ($)	
Device Type	Day	Night	Day	Night
Type I barricades	0.94[a] (5)[b]	1.27 (5)	6.25 (5)	6.06 (4)
Type II barricades	1.10 (6)	1.61 (6)	11.25 (6)	5.56 (3)
Vertical panels	0.50 (2)	0.67 (1)	5.95 (4)	4.40 (1)
Cones	0.41 (1)	1.24 (4)	3.91 (2)	7.20 (6)
Tubes	0.69 (4)	1.16 (3)	3.55 (1)	6.29 (5)
Drums	0.60 (3)	0.83 (2)	4.63 (3)	4.46 (2)

[a]Example calculation: Device cost per foot of array detection distance = Device cost/Array detection distance = $40/4,250 ft = $0.0094/ft. Device cost per 100 ft of array detection distance = $0.0094/ft x 100 = $0.94.
[b]Ranking with respect to other devices (1 = best value).

in Table 1 and the cost data in Table 2 using the basic equation: Value = Performance/Cost. However, the values are expressed in inverse form in the table (i.e., device cost per unit of performance).

It should be noted again that the minimum taper length and maximum device spacing recommended in the MUTCD were assumed for all the alternatives. Thus each alternative array would contain the same number of devices. For this reason the values in Table 3 are expressed in device cost rather than array cost.

DEVICE SELECTION

From Table 3, vertical panels and drums are "good values" for combined day and night use at the freeway work zone, and vertical panels by a slight margin are the best value. From the table, a vertical panel costs only $0.67 for every 100 ft of nighttime detection distance it provides. This cost is slightly lower than that of drums that cost $0.83 per 100 ft of nighttime detection distance.

Vertical panels also are the best value for encouraging early lane changes at night. For each 100 ft of lane change distance, they cost $4.40. Drums also are a good value at $4.46 per 100 ft of lane change distance.

Both vertical panels and drums have relatively good value in the daytime. Only cones and tubes represent a better daytime value.

Thus, based on the value engineering analysis, vertical panels mounted on portable stands are recommended for the freeway work zone. Drums could be used as an alternate. Both of these devices are relatively low cost ($22 and $25, respectively), and they provide adequate performance, day and night.

CONCLUSIONS AND RECOMMENDATIONS

Value engineering appears to be a useful and practical tool for selecting work zone channelizing devices. It provides an objective means of evaluating any number of alternative devices using whatever performance and cost data are available. Most important, it encourages the selection of low-cost devices that are safe and effective under the assumed conditions.

To be most effective, a value engineering study should be based on comprehensive and accurate information. It is also important to use a team approach in which team members are well trained and diverse in experience and technical background. For these reasons, it is recognized that the value engineering approach is most appropriate for central office use. By pooling central office staff and data-gathering resources, value engineering can be used as an analytical tool for establishing work zone traffic control standards and for planning and allocating resources.

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Abridgment

Potential Impact of Speed Reduction at Freeway Lane Closures: A Simulation Study

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ABSTRACT

The objective of this study was to evaluate the potential impact of reduced speed limits at temporary freeway lane closures at work zones at arbitrarily assumed levels of compliance. Although some transportation engineers prefer to reduce speeds at work zones to protect the working crew, others are hesitant to introduce such a disturbance to the traffic flow. The study approach involved simulation experimentation, using FREESIM, a microscopic, stochastic model. A fractional factorial design was developed for the analysis of three independent variables: two-lane volumes (800, 1,200, 1,500 and 1,800 vehicles per hour); speed limits (55, 50, and 45 mph); and assumed compliance with speed limit (33, 66, and 100 percent). The number of uncomfortable decelerations and the variance of the speed distribution were selected as the dependent variables. These two variables were offered as a measure of the internal friction created by the merging of two-lane traffic into a single lane. It was hypothesized that this internal friction is increased by the introduction of lower speed limits. The results of this simulation study indicate that compliance with reduced speed limits will have no significant impact on the number of uncomfortable decelerations but will reduce variance in speed distribution. These results, therefore, do not support the assumption that effective speed reduction at work zones would create a potentially hazardous disturbance in the flow of traffic.

Freeway lane closures at work zones require properly developed traffic control plans to minimize the disturbance of the traffic flow and provide for the safety of both drivers and working crew. The introduction of reduced speed zones is a somewhat controversial aspect of traffic control. Although, at least intuitively, reduced speed implies greater safety especially for the working crew, it introduces a disturbance that may well have a negative impact on the safety of the traffic flow.

The objective of this computer simulation study was to evaluate the potential safety impacts of speed zones at freeway lane closures at different levels of assumed compliance. The specific configuration selected for analysis was the closure of the median (left or passing) lane on two-lane, rural freeway sections.

It is generally recognized that posted speed limits are not necessarily effective in reducing mean speeds, although there are means by which the effectiveness can be improved. This study, however, was concerned only with the impact of the reduced speed limits at specified levels of compliance on the stability of traffic flow.

SIMULATION MODEL

FREESIM is a microscopic, stochastic simulation model. The model logic is based on a rational description of the behavior of drivers in a lane closure situation. The vehicles are advanced in the system using the classical car-following approach. The model simulates lane changing as well as overtaking. The simulation program is written in SIMSCRIPT II.5 programming language (1).

Verification of the simulation model included operational testing of the simulation dynamics algorithms (i.e., car following and lane changing) and sensitivity analysis of measures of effectiveness to the exogenous (input) variables.

Validation of the simulation model was accomplished by the comparison of simulated time-headway, speed, and merging distributions with four sets of actual observations obtained from three different rural freeway lane closure sites (2). Also, the simulation model outputs on overall speed, flow, throughput, and lane-changing frequencies were compared with some well-known empirical data from the literature.

The input requirements for implementing reduced speed zoning are the reduced speed limit parameters (i.e., legibility and perception-reaction time) for the speed limit sign and the proportion of drivers that complies with it.

DESIGN OF SIMULATION EXPERIMENTS

A fractional factorial design was developed for the analysis of three independent variables: speed limit, specified compliance with the speed limit, and two-lane approach volume. Compliance levels of 0.33, 0.66, and 1.00 were used for each of the two reduced speed limits implemented: 50 and 45 mph. Four levels of two-lane approach volumes (vehicles per hour) were used: 800, 1,200, 1,500, and 1,800. A total of 140 simulation runs were made: five "replications" of each of the 28 combination of factor levels.

Because the objective of this study was to evaluate the safety impacts of speed zoning, two safety-related measures of performance were selected:

1. The number of uncomfortable decelerations per vehicle-hour (UNCOM.DECEL) and
2. The variance of speed distribution in the open lane at the beginning of the transition zone (i.e., taper).

UNCOM.DECEL is perhaps the best representation of driver response to unexpected, potentially unsafe conditions. An uncomfortable deceleration is defined as one that exceeds by more than 2 ft/sec² what is normally considered the comfortable deceleration rate for vehicular traffic at a given speed [see Table 2.7 of The Transportation and Traffic Engineering Handbook (3)].

FINDINGS

The average UNCOM.DECEL and the variance of speed at a transition zone in the open lane, for each combination of factor levels, are given in Tables 1 and 2, respectively. The impacts of reduced speed zoning on uncomfortable decelerations and on variance of speed are discussed separately.

Uncomfortable Decelerations

The base condition is represented by the 55 mph speed limit condition. In this case, each driver has a desired speed assigned from one of two normal distributions (means: 52.8 mph inner lane, 57.2 mph passing lane) intended to represent near free flow conditions.

It is quite clear that the UNCOM.DECEL increases rapidly as volumes approach single lane capacity. The question is: will compliance with a reduced speed limit compound the problem or will it offset it?

Compared to the base case, some reduction in the UNCOM.DECEL is observed in most cells in Table 1. Note, however, that changes due to compliance with reduced speed limits are insignificant in comparison to changes due to variation in volume.

Overall, reduced speed limits have a mostly positive but negligible, in magnitude, impact on uncomfortable decelerations.

TABLE 1 Average Number of Uncomfortable Decelerations per Vehicle-Hour

Speed Limit (mph)	Compliance (%)	Volume (vph) 800	1,200	1,500	1,800
55		0.0465	0.1130	0.2784	0.6989
50	33	0.0485	0.0947	0.2187	0.6767
	66	0.0525	0.0990	0.2808	0.6767
	100	0.0550	0.0937	0.3037	0.8416
45	33	0.0550	0.0847	0.2221	0.5453
	66	0.0465	0.1063	0.2597	0.6544
	100	0.0470	0.0837	0.2795	0.5844

TABLE 2 Variance of Speed (miles²/hr²) at the Beginning of the Transition Zone in the Open Lane

Speed Limit (mph)	Compliance (%)	Volume (vph) 800	1,200	1,500	1,800
55		33.69	44.41	80.19	102.11
50	33	21.35	38.43	62.00	97.07
	66	17.90	26.43	81.89	107.45
	100	9.80	22.26	84.49	112.35
45	33	26.37	32.47	53.90	83.12
	66	18.05	25.19	56.59	81.63
	100	10.14	20.07	64.83	84.70

Variance of Speed Distribution

The variance of speed distribution at the taper is suggested as another measure of disturbance created by the forced merge of two-lane traffic into a single lane. To facilitate the interpretation of the results of the simulation study, the mean speeds are also presented (Table 3). In the base case situation (at 55 mph speed limit), the mean speeds drop by about 10 mph as volumes increase from 800 to 1,800 vehicles per hour (vph). This is not unexpected as practically all the approaching vehicles will be traveling in the open lane at the beginning of the taper, having completed the necessary merging maneuvers. Because mean speeds are near or below 50 mph, compliance with a posted 50 mph speed limit would have little impact on mean speeds, especially at higher volumes. The data presented in Table 3 confirm this expectation. The reduction in speed variance is also limited to the lower volume ranges (Table 2).

Compliance with a 45 mph speed limit, however,

TABLE 3 Mean Speed (mph) at the Beginning of the Transition Zone in the Open Lane

Speed Limit (mph)	Compliance (%)	Volume (vph) 800	1,200	1,500	1,800
55		51.13	49.03	45.66	41.59
50	33	50.33	48.70	46.06	41.88
	66	49.68	48.67	45.26	41.63
	100	49.43	48.46	45.11	41.34
45	33	48.62	47.20	44.20	40.43
	66	45.60	44.68	42.25	39.37
	100	44.59	44.10	41.20	39.06

would result in both lower mean speeds and lower speed variances. The reduction in speed variance due to compliance with the 45 mph speed limit can offset or reduce considerably the impact of increased volumes. For example, at 1,800 vph, the speed variance at 45 mph is only slightly higher (83.12, 81.63, 84.70) than the 55 mph speed variance at 1,500 vph (80.19).

DISCUSSION OF RESULTS

There are two opposing interests in effect at work zones involving freeway lane closures. The protection of the working crew appears to require speed reduction to minimize the potential impact of cars accidentally crashing into the working zone. The obvious interest of the driving population is to pass by working zones without delay or disturbance.

This study was undertaken to test the hypothesis that the introduction of speed reduction will increase the disturbance created in the traffic flow by the lane closure. The simulation study, however, did not generate any evidence to support this assumption.

Compliance with either the 50 mph or the 45 mph speed limit had negligible impact on uncomfortable decelerations.

Compliance with the 50 mph speed limit resulted in a small reduction in the variance of speed reduction at lower volume ranges. At higher volume ranges, the changes were largely negligible. It was noted that at higher volumes the mean speeds were considerably lower than 50 mph even before the introduction of speed control. Both mean speed and variance of speed distribution were found to be strongly volume dependent.

Compliance with the 45 mph speed limit, however, offset at least partly the increase in speed variance created by increased volume levels.

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Guidelines for Positive Barrier Use in Construction Zones

DEAN L. SICKING

ABSTRACT

The need for positive barriers in construction zones is now based on individual judgment, and there is an expressed need for objective barrier placement criteria. The number, diversity, and variability of factors that affect barrier need within a work zone eliminate the possibility of development of a set of totally objective guidelines appropriate for any given circumstance. In an effort to develop procedures for determining barrier need at a given site, a computer program, which determines barrier warrants based on a benefit-cost algorithm, was developed. This program was adapted for use on microcomputers with a user friendly system that greatly simplifies its use. Further, a set of use guidelines for portable concrete barriers (PCBs) was developed for typical work zone sites through the application of the benefit-cost computer program. Barrier end treatment use guidelines were also developed with the computer program. End treatments studied included flaring the barrier away from the travelway and two crash cushions. The optimum flare rate for flared end treatments was found to be 10:1. Crash cushion end treatments were found to be cost beneficial relative to flared end treatments only at extremely high traffic volumes.

Considerable attention has been focused on hazards in construction zones that endanger both the traveling public and working personnel. Efforts are being made on the national and state level to improve safety in these zones. Many studies have been and are being made to evaluate traffic management and control measures, accident data, driver needs, and delineation devices, and to develop positive barriers for work zones.

A major problem highway engineers have faced is the absence of objective guidelines for selection and deployment of positive barriers in work zones. Engineers from 9 states were recently surveyed to determine the current practices regarding the use of positive barriers in construction zones. In general, the survey showed that judgment is the prevalent means by which most states establish barrier need and that the use of positive barriers in construction zones is determined on a project-by-project basis. Further, most of the engineers surveyed expressed a need for objective guidelines for positive barrier use in construction zones.

The benefit-cost method of evaluating safety alternatives has gained widespread acceptance in recent years. This method is normally based on either accident data analysis or encroachment probability predictions of accident frequency and severity that can be expected with each safety alternative.

Numerous accident data studies have attempted to quantify the frequency and severity of work zone accidents as a function of work zone characteristics (1-5). General findings were: accident rates typically show a moderate increase when good traffic control policies are implemented; lane closures, bridge work, and reconstruction of roadways create higher accident rates; there are no clear relationships between work zone characteristics and accident rates and severity; rear-end, head-on, and fixed-object accident rates increase greatly; and accident severities do not increase significantly. Ideally, accident data should provide estimates of work zone accident rates and severities. However, such estimates are impossible to obtain because of the limited quantity and lack of sufficient detail in existing work zone accident data records.

Encroachment probability models are more general in nature and can be adapted to fit almost any highway or work zone configuration. Therefore, this research was undertaken to use benefit-cost methodology with an encroachment probability model to develop simplified objective guidelines for the use of positive barriers in construction zones. The findings of a research project completed in 1983 (6) are described. The reader should refer to the cited report for more detailed information about this study.

REVIEW OF TEMPORARY BARRIER DESIGNS

A review of practices in selected states was made to determine the types of temporary positive barriers and barrier end treatments in use. With few exceptions, portable concrete barrier (PCB) designs are being used when a positive barrier is determined to be warranted. Some typical installations of the barrier are shown in Figure 1. A variety of end treatments has been used on portable concrete barriers including the construction zone GREATcz barrier (7), sand-filled plastic barrels, and flaring the barrier away from the travelway. Of these end treatments, only the GREATcz crash cushion has been crash tested and proven to meet existing performance standards. Another end treatment that has passed recommended performance testing is the steel drum and sand crash cushion (8). Although this cushion has only recently been developed and has little field experience, it offers a low-cost alternative to the GREATcz. Figure 2 shows the two PCB end treatments that have passed recommended crash tests.

IMPACT PERFORMANCE

In developing guidelines for positive barrier use in construction zones, the impact conditions for which a barrier will perform adequately, or performance level, must be quantified. It is generally known that the degree of impact loading or "impact severity" experienced by a barrier during an accident is related to mass, velocity, and impact angle of the

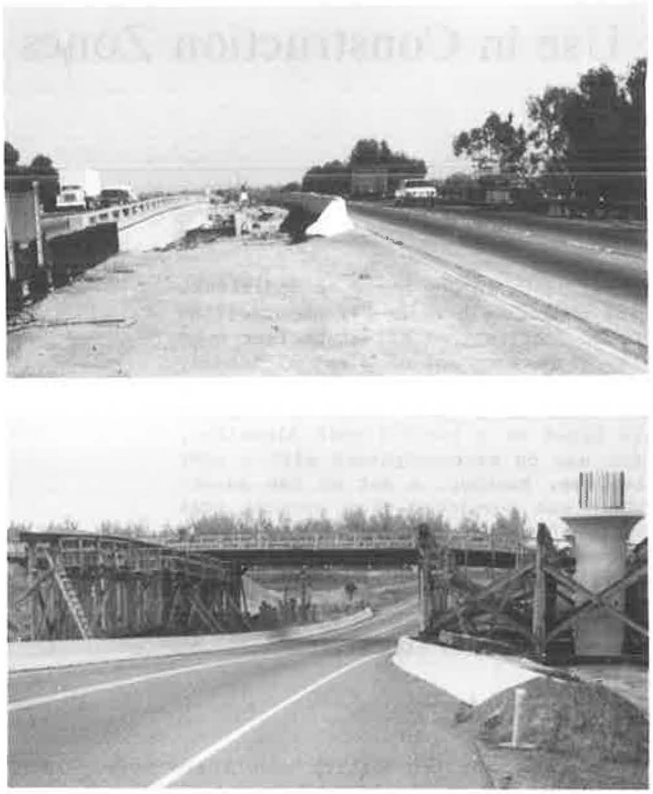

FIGURE 1 Typical use of precast concrete safety shape barriers in work zones: top, bridge widening project; bottom, construction overpass.

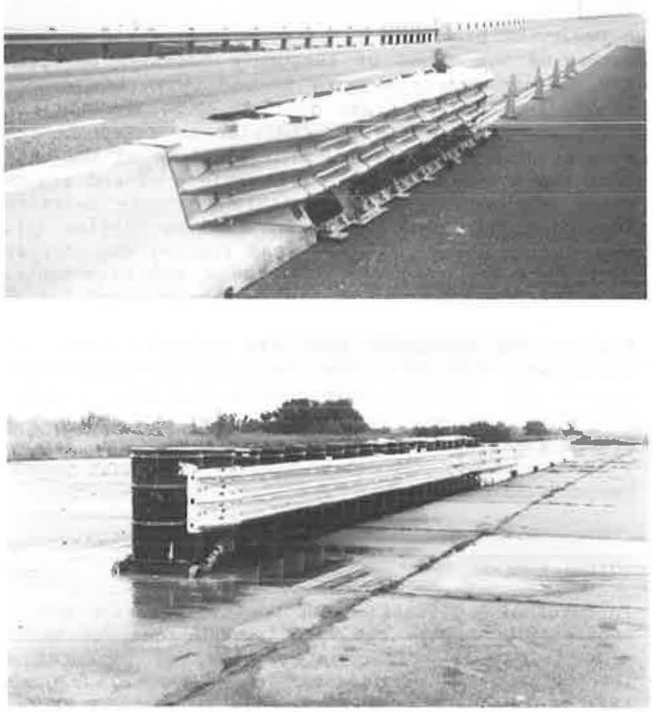

FIGURE 2 End treatments for portable concrete barriers that have been tested successfully: top, GREATcz; bottom, barrel and sand.

vehicle striking the barrier. Studies have shown the following relationship to be good indicator of impact severity and the demands imposed on longitudinal barriers (9-11):

$$IS = 1/2\ m(V \sin\theta)^2$$

where

- IS = impact severity (ft-lb),
- m = vehicle mass (lb-sec^2/ft),
- V = vehicle impact velocity (ft/sec), and
- θ = vehicle impact angle (angle between resultant velocity vector of vehicle and face of barrier).

For any barrier system there is a limiting value of IS beyond which the barrier will not contain or smoothly redirect the impacting vehicle, or both. The limiting value of IS is defined herein as the barrier's "performance level" (PL). When a vehicle strikes the barrier with an IS greater than its PL, the vehicle may penetrate the barrier. Table 1 gives a summary of performance levels established by Ivey (9) for many commonly used work zone barrier designs.

TABLE 1 Estimate of Performance Limit for Representative Work Zone Barriers *(9)*

Description of Joint	Performance Level, PL (ft-kips)	Estimated Displacement (ft)[a]
Tongue and groove	20.4	1.0
Steel dowel	20.4	1.0
Lapped joint	36.5	1.0
Pin and rebar	97.3	2.0
Vertical I-beam	97.3	1.0
Channel splice	97.3	2.0

[a]Values given are deflections expected to occur at given PL.

BARRIER COSTS

As part of the subject study, researchers at the Highway Safety Research Center (HSRC), University of North Carolina, gathered barrier cost and labor requirement information from six work zones in four different states. The work zones studied employed PCB barriers with performance levels (PL) ranging from 20,000 to 97,000 ft-lb. As part of another study, costs and labor associated with the use of PCBs were examined (9). Data from these sources were used to arrive at nominal cost figures for commonly used PCBs. A major finding was that high-performance precast concrete barriers are not significantly more costly than are low-performance barriers. The initial cost of PCBs with a PL of 96,000 ft-lb was found to be approximately $21.00 per foot and the salvage value was estimated at $10.50 per foot. Normal maintenance costs, exclusive of accident damage repair costs, were found to be quite low.

Another important cost is that required to restore a damaged barrier following impact by an errant vehicle. Barrier damage is generally believed to be proportional to the degree of impact loading on the barrier. As discussed earlier, barrier loading can be quantified by the impact severity (IS), and it follows that barrier repair costs should be roughly proportional to IS.

Average costs to repair PCBs have been collected from a limited number of highway work zones as mentioned previously. The researchers found that a large number of impacts caused no barrier damage and that the average cost to repair PCBs after accidents causing damage was approximately $100. Accident reports were also collected from the construction zones studied. Some of the accident reports show estimated impact conditions and barrier repair costs. PCB crash test reports also give impact conditions and barrier damage. Data from these two sources were used to develop a straight line least squares fit between IS and barrier repair costs. Figure 3 shows the relationship developed and the raw data points.

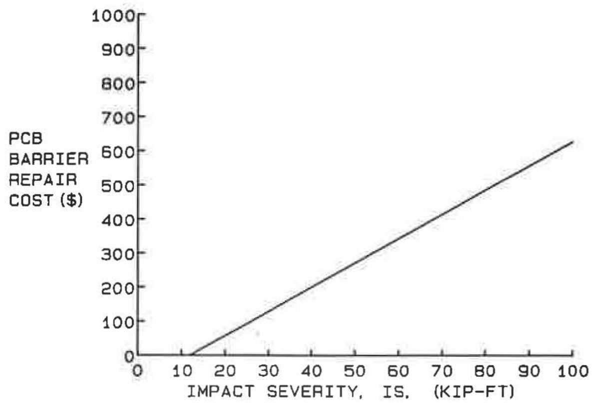

FIGURE 3 PCB repair costs.

Estimates of end treatment costs were limited to the two systems that have been successfully crash tested as end treatments for concrete barriers, the GREATcz and the steel drum and sand cushion. Initial cost estimates for the GREATcz system were obtained from the manufacturer. Similar estimates for the steel drum and sand cushion were obtained through basic cost estimating procedures. Table 2 gives estimated costs for the two systems. End treatment repair costs for the steel drum and sand cushion were estimated from crash test results. GREATcz system repair costs were assumed to be half those of the steel drum and sand system. Plots of repair cost versus IS for the two end treatments are shown in Figure 4.

TABLE 2 Crash Cushion Costs and Impact Severities

	Barrel and Sand Crash Cushion	GREATcz Crash Cushion
Initial cost ($)	4,000.00	11,000.00
Salvage value ($)	2,500.00	8,000.00
Maintenance cost ($/yr)	200.00	100.00
Severity index corresponding to 60 mph impact	5.0	5.0

BENEFIT-COST METHODOLOGY

The benefit-cost methodology compares the benefits derived from a safety improvement to the direct highway agency costs incurred as a result of the improvement. Benefits are measured in terms of reductions in societal costs due to decreases in the number or severity, or both, of accidents. Direct highway agency costs are comprised of initial, maintenance, and repair costs of a proposed improvement. A ratio between the benefits and costs of an im-

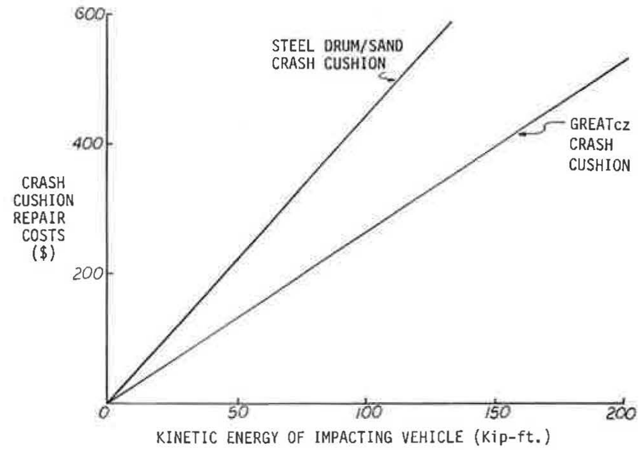

FIGURE 4 Repair costs for crash cushion end treatments.

provement is used to determine if the improvement is cost beneficial:

$$B/C_{2-1} = (SC_1 - SC_2)/(DC_2 - DC_1)$$

where

B/C_{2-1} = benefit-to-cost ratio of Alternative 1 compared to 2,
SC_1 = societal cost of Alternative 1, and
DC_1 = direct cost of Alternative 1.

When studying the cost effectiveness of work zone barriers, safety treatment Alternative 1 is normally an unprotected work zone and Alternative 2 is PCB protection of the work zone.

The benefit-to-cost ratio makes possible comparison of the benefits and costs of two alternatives, one of which is normally viewed as a safety improvement in comparison with the other. If the improvement has a benefit-to-cost ratio less than 1.0, the improvement normally should not be implemented. It does not follow, however, that all improvements with a benefit-to-cost ratio greater than 1.0 can be implemented. Budgeting limitations prevent the funding of all projects that have a benefit-to-cost ratio of 1.0 or more.

ESTIMATION OF BENEFITS AND COSTS

Benefits or reductions in societal accident costs and safety improvement repair costs can only be estimated through predictions of accident frequency and severity. As mentioned previously, an encroachment probability model is the only available method of predicting construction zone accident frequency and severity. Encroachment probability models have been used by numerous authors to predict roadside accident frequencies and severities (12-15). A similar model with certain improvements was used in this study. The most notable improvement was the development of a procedure to estimate the frequency of multiple vehicle accidents resulting from automobiles inadvertently crossing the centerline of a two-lane, two-way highway.

The societal cost or severity of an accident and the damage incurred by highway appurtenances due to an accident are related to many factors, including object struck and speed and angle of impact. Quantification of the costs associated with a predicted accident is therefore a formidable task.

Accident severity has frequently been measured in terms of severity indices as shown in the 1977 AASHTO Barrier Guide (13). To a limited extent the severity indices associated with accidents involving certain roadside features such as barriers, signs, ditches, and driveways have been determined via crash testing, computer simulation, and accident data. However, severity indices for many features had to be inferred or extrapolated from limited data, as shown in Figures 5 and 6. Techniques for determining severity indices used in the present effort are presented elsewhere (6).

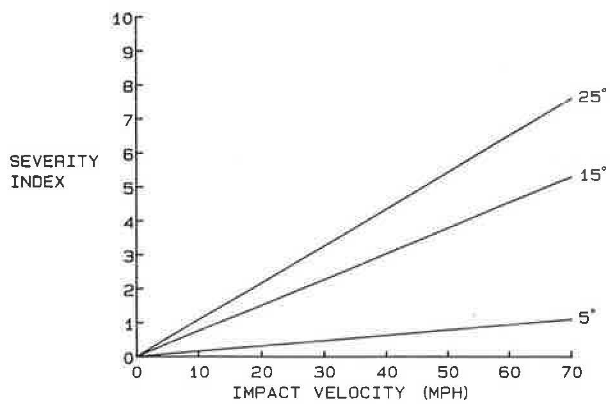

FIGURE 5 Accident severity versus impact speed for PCB.

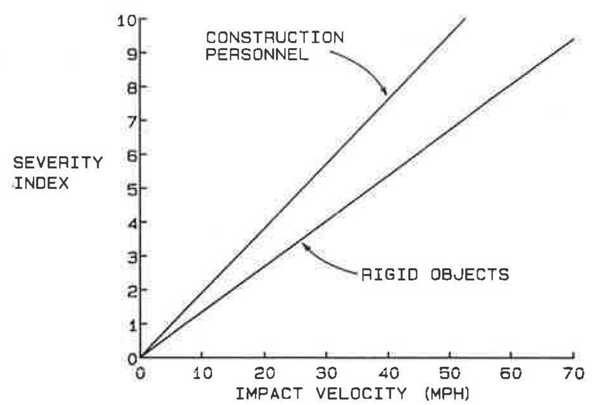

FIGURE 6 Impact severities of construction zone activities.

Direct costs arising from safety appurtenance repair can be estimated from crash test results and available accident data as described previously. Direct initial costs and normal maintenance costs can be determined through basic cost estimating procedures or previous contractor bids.

ASSUMPTIONS AND LIMITATIONS

A number of assumptions are necessary in an analysis of the type employed herein. These assumptions are as follows:

1. It was assumed that recommended traffic control and delineation devices (16) will be used regardless of the presence or absence of a positive barrier.
2. It was assumed that the rate of inadvertent encroachments from the travelway will not be altered by the presence of a positive barrier. The validity of this assumption depends to a large degree on the validity of Assumption 1.
3. It was assumed that the use of a positive barrier will not, on the average, appreciably alter access to the work zone. In other words, it was assumed that the total direct cost of a project, excluding barrier costs, is not altered by the presence of positive barriers. It is believed that worker productivity will be enhanced by the presence of a barrier, balancing any negative effects the restriction of access may have.

The analysis procedures and results presented herein, when combined with good engineering judgment, should enable an agency to develop a rational and uniform set of guidelines for the use of work zone barriers. However, the guidelines should not be viewed as absolute because of the diverse and complex nature of the problem and the inexactness of the analysis procedure. Those responsible for establishing policies for barrier use should become familiar with the procedure, its limitations, and the input parameters used in developing the criteria.

APPLICATION OF BENEFIT-COST METHODOLOGY

A computer program was written to facilitate implementation of the benefit-cost methodology. The original version of the program was written for use on a mainframe computer. The guidelines presented herein were determined by application of this program. The methodology has also been programmed for use on microcomputers with user friendly input features. This version of the program will allow engineers to apply the benefit-cost methodology to individual work zones.

BARRIER END TREATMENT

Barrier end treatments are a major factor in the cost-effectiveness of work zone barriers. Two basic treatments were examined, namely flaring the barrier away from the travelway and crash cushions.

For analysis of flared end treatments, it was assumed that the flare would begin at the upstream end of the work zone. If sufficient room is available to start this flare before the end of the zone, flaring the barrier becomes more cost beneficial relative to other end treatments. Three different barrier flare rates, 17:1, 10:1, and 6:1, were examined over a wide range of barrier end offsets. General findings were that the 6:1 and 10:1 flare rates were approximately equivalent, and that both end treatments are cost-effective over a wide range of conditions. The 10:1 flare rate is recommended because it provides a somewhat safer treatment at essentially the same cost. A 10:1 flared end treatment with a flare offset of 20 ft was found to be cost-effective over a wide range of conditions and was therefore used for all subsequent studies. Although the 17:1 flare rate was not found to be cost-effective compared to the other flare rates, this flare rate is the most cost beneficial when the flared region can be started within the required work zone because the flared region does not increase the required length of barrier.

The two crash cushions studied were the GREATcz (7) construction zone crash cushion and a steel drum and sand cushion (8). Estimated crash cushion costs are given in Table 2 and shown in Figure 4. Figure 7 shows the traffic volumes and barrier offsets at which each of the two cushions first becomes cost beneficial. Surprisingly, these end treatments do not appear to be cost beneficial until traffic volumes become relatively high. Note that the higher

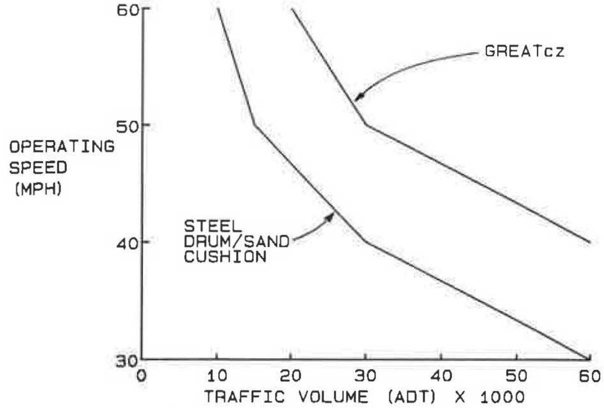

FIGURE 7 Guidelines for use of crash cushion end treatments (project duration = 1 year).

cost GREATcz crash cushion was not found to be more cost beneficial than the steel drum and sand cushion under any highway condition studied and that, for most reasonable traffic volumes, flaring the barrier was found to be more cost-effective than either crash cushion.

GUIDELINES FOR BARRIER USE IN SPECIFIC WORK ZONE ACTIVITIES

During the early phases of the study, numerous highway work zones across the country were visited and construction activities were photographically recorded. Films of these work zones were carefully reviewed in an effort to determine activities, distributions of obstacles and construction personnel, zone length, project duration, and so forth found in typical construction zones. This analysis revealed that, from the standpoint of number, type, and offset of roadside hazards, no two construction zones are alike. However, it was found that a large percentage of work zone activities can be placed in one of three basic categories. These are roadway widening, bridge widening, and construction of roadside structures such as bridge piers for an overpass structure.

It was impossible to define a single typical work zone for each of the construction zone categories identified. However, project-dependent parameters such as project duration, construction zone length, and number of rigid hazards were then varied over a limited range to determine a group of "typical" work zones for each construction zone classification. Guidelines were then developed for barrier deployment in each typical zone.

Another common construction zone activity of interest is the use of two-lane, two-way operations (TLTWO) on what is normally a divided highway. Fewer unknown parameters are of significance to this type of activity, and it was therefore not necessary to define a typical zone to develop barrier need. In addition to developing guidelines for barrier use in the TLTWO, an attempt was made to determine an optimum geometric configuration or layout for the barrier.

Barrier end treatments are a major factor in the cost-effectiveness of work zone barriers. As mentioned previously, flaring the barrier away from the travelway at a 10:1 flare rate over a distance of 200 ft was found to be cost beneficial over a wide range of conditions. This end treatment was used for all barrier applications studied.

BARRIER WIDENING

At the initiation of a bridge widening project, traffic is normally shielded from the construction zone by the existing bridge rail. After the bridge rail is removed, traffic exposure to the work zone can normally be divided into three distinct phases. During the initial phase, new bridge deck is under construction and a dropoff exists near the edge of the travelway. The new deck is poured during the second phase and the dropoff is moved to the edge of the new traffic lanes. The final phase is completed after the new bridge rail is in place and traffic is no longer exposed to a dropoff. Each of these phases is assumed to comprise one-third of the entire project duration.

Other hazards assumed to be present in the bridge widening construction zone included either four or eight workers and two large pieces of equipment. The equipment hazards were assumed to be in the zone only during the final phase of the project. Figure 8 shows the assumed locations of the workers and equipment in a bridge widening work zone. It should be noted that the lateral position of workers and equipment and equipment size affect the probability that a vehicle will strike these hazards. It was assumed that the longitudinal position of these objects had no influence on probability of impact.

Eight typical work zones were established for the bridge widening project. Variations between the zones involve the number of workers, the number of heavy equipment hazards, and the severity of the

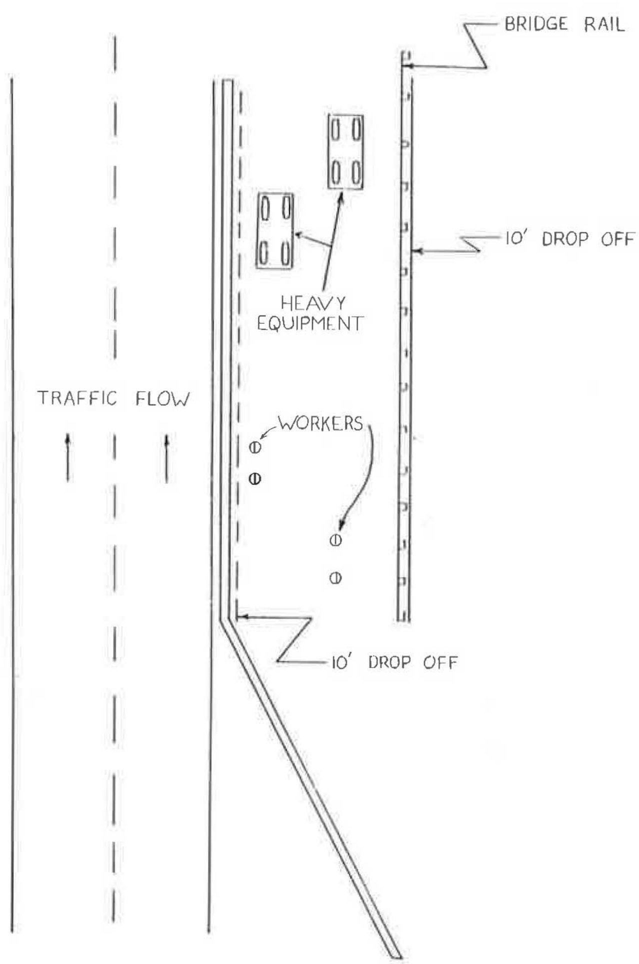

FIGURE 8 Typical bridge widening construction zone.

dropoff hazards. A bridge normally crosses a highway or a terrain discontinuity such as a river or a stream. The severity index associated with a vehicle running off a bridge at a grade separation was assumed to be approximately 15 percent greater than the severity index for leaving a bridge over a terrain discontinuity.

Figure 8 also shows the assumed geometric configurations of a construction zone barrier used to shield traffic from a bridge widening project. Although not explicitly shown in this figure, the roadway is assumed to be a four-lane divided highway and is typical of many roadside construction zones studied. For divided highways, traffic exposed to the work zone is only one-half the total traffic volume.

Guidelines for deployment of the PCB were developed for each of the bridge widening zones studied in the form shown in Figure 9. The line plotted in this figure corresponds to a benefit-to-cost ratio of 1.0 for a project duration of 1 year. As shown in Figure 9, barrier need decreases sharply as highway operating speed decreases.

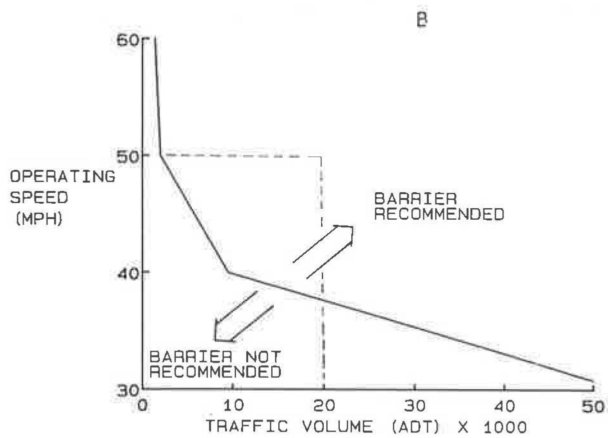

FIGURE 9 Typical guidelines for PCB protection of bridge widening zones (bridge over grade separation).

Guidelines such as those shown in Figure 9 can be used to determine when barrier implementation becomes cost-effective. Use of the guidelines is illustrated in the following example for a bridge widening project.

Example 1
Typical Bridge Widening Zone 1
(bridge over a grade separation)

Operating speed	50 mph
Project duration	1.0 yr
Average daily traffic	20,000 ADT

From Figure 9 the benefit-to-cost ratio of PCB use is determined to be greater than 1.0 and barrier use merits further consideration. As has been discussed, the decision should be made in conjunction with benefit-cost analyses of other safety projects in view of limited safety funds.

ROADWAY WIDENING

Hazards located in roadway widening projects are normally limited to four basic types: construction personnel, dropoffs, heavy equipment, and light equipment. Figure 10 shows the assumed hazard distribution and barrier configuration for a roadway

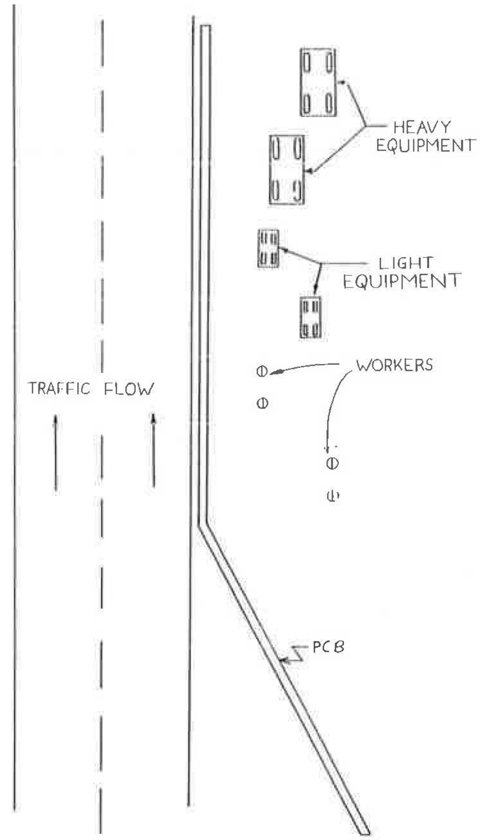

FIGURE 10 Typical roadway widening construction zone.

widening project. Sixteen typical work zones were established for roadway widening construction projects. Variations among the zones include number of construction personnel, number of heavy equipment hazards, work zone offset, and depth of dropoff. The pavement edge dropoff is assumed to be either 1 ft for simple pavement widening or 10 ft for vertical realignment projects.

Guidelines for PCB barrier implementation were again developed for each of the 16 typical roadway widening zones. Combinations of construction zone length and average daily traffic (ADT) which produce a benefit-to-cost ratio of 1.0 were plotted as shown in Figure 11. Use of these guidelines is demonstrated in the following example.

Example 2
Typical Roadway Widening Zone 4
(vertical realignment--dropoff depth = 10. ft)

Operating speed	40 mph
Average daily traffic	30,000 ADT

Figure 11 shows the benefit-to-cost ratio of PCB use in this zone to be less than 1.0 and therefore barrier use is not normally recommended.

ROADSIDE STRUCTURE CONSTRUCTION

Construction projects involving erection of roadside structures such as bridge supports at grade separations normally require construction of falsework structures. Large falsework structures are costly; pose a serious hazard to motorists; and, if damaged when struck by an errant vehicle, could cause injuries to workers and involve significant cost to

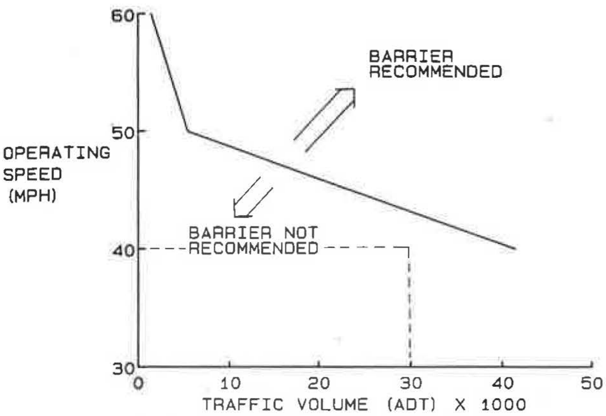

FIGURE 11 Typical guidelines for PCB protection of roadway widening construction zone (with vertical realignment activity).

restore. After removal of the falsework, a rigid support structure that can pose a serious hazard to motorists usually remains. For the purpose of generating barrier placement guidelines, it was assumed that the falsework would remain in place for one-half the total project duration and that a rigid support structure would remain during the balance of construction.

Other hazards assumed to be present during the construction of roadside structures include two heavy equipment hazards and either eight or twelve construction personnel. Figure 12 shows the assumed construction zone geometric configuration and hazard distribution. Note that heavy equipment and worker hazards are assumed to be present only while the falsework is present. A total of 12 zones were studied. Variations among the typical roadside construction zones included the number of workers, size of falsework, and the offset of the construction zone.

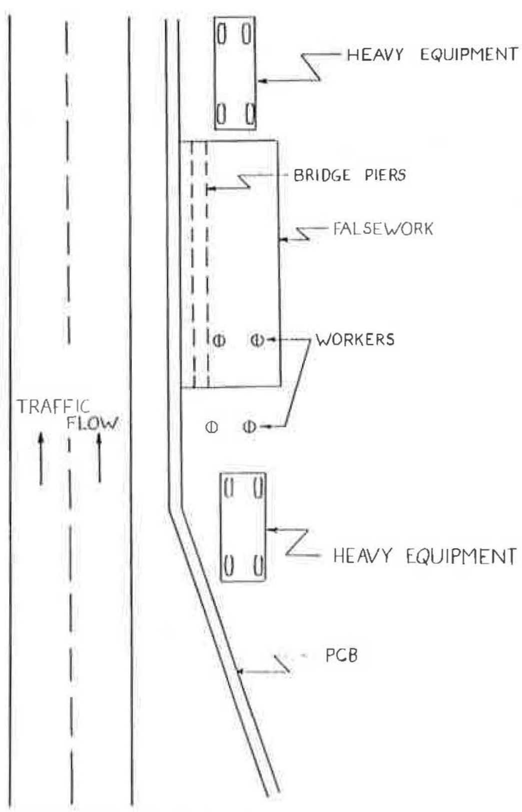

FIGURE 12 Typical roadside structure construction zone.

Guidelines for barrier implementation were developed on the basis of average daily traffic, highway operating speed, and damage to falsework resulting from an impact. Lines corresponding to a benefit-to-cost ratio of 1.0 were plotted for average daily traffic versus operating speed. Figure 13 shows guidelines developed for a typical roadside structure construction zone. Use of these guidelines is illustrated in the following example.

Example 3

Typical Roadside Structure Construction Zone 11

Potential structural damage	\$100,000
Operating speed	50 mph
Average daily traffic	5,000 ADT

Figure 13 shows that the benefit-to-cost ratio for barrier use in this zone is greater than 1.0 and therefore barrier use merits further consideration as discussed previously.

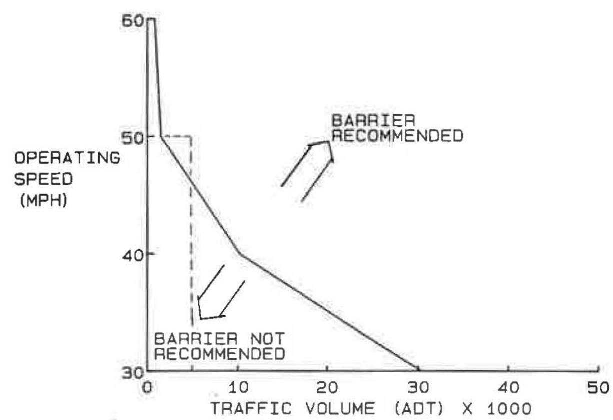

FIGURE 13 Typical guidelines for PCB protection for major structural elements under construction (falsewark cost = \$100,000).

TWO-LANE, TWO-WAY TRAFFIC OPERATIONS

Frequently, during long-term construction activities on a four-lane divided highway, all traffic is diverted to one side while the other side is closed for maintenance. This creates a two-lane, two-way operation (TLTWO) on what is normally a divided highway. For these work zones, multiple vehicle median crossover accidents are of primary concern.

In the development of barrier guidelines for TLTWO work zones, it was assumed that opposing vehicles comprise all hazards to encroaching vehicles. Further, no effort was made to evaluate the effect a barrier would have on rear-end accident rates. The reader should refer to Ross and Sicking (6) for a discussion of encroachment rates into opposing traffic lanes and the probability of affecting opposing traffic.

A typical TLTWO work zone can be described by determining the appropriate crossover configuration. Two basic types of barrier layouts were investigated. The barrier configuration shown in Figure 14 was found to be more cost-effective than the other configuration studied for all combinations of work zone length and traffic volume. Note that the treatment involves a barrier flared at a 10:1 rate to an offset of 20 ft from the edge of the travelway. No special end treatments were used on either end of

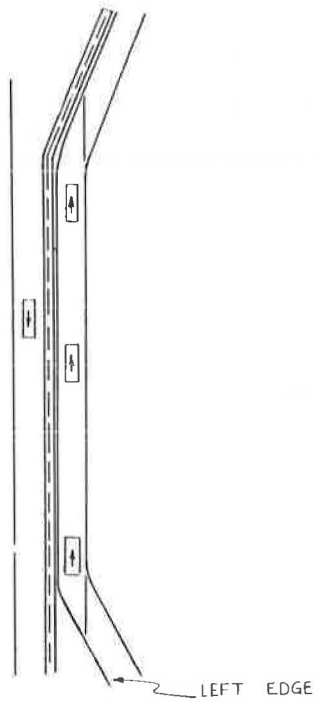

FIGURE 14 Two-lane, two-way traffic operations with positive barrier lane separation.

the barrier. Guidelines were developed by determining combinations of TLTWO operating speed, project duration, and traffic volume for which the benefit-to-cost ratio for barrier implementation equals 1.0 as shown in Figure 15. These guidelines are used in the same general manner as those developed for other barrier applications. It is important to note that the guidelines shown were developed on the basis of the assumption that the work zone contains no at-grade intersections in the TLTWO. If such intersections cannot be avoided, the benefit-cost computer program should be employed to determine barrier warrants on a case-by-case basis.

CONCLUSIONS

The problem of formulating a set of guidelines for barrier use in work zones is indeed formidable. The number, diversity, and variability of factors that affect barrier need within a given work zone greatly complicate the problem. Moreover, there are simply no two work zones that have the same characteristics. In other words, a set of totally objective guidelines appropriate for any given circumstance cannot be developed.

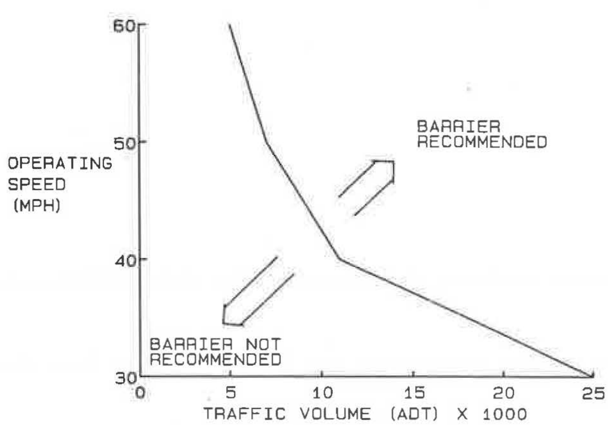

FIGURE 15 Guidelines for conventional PCB use in centerline of TLTW construction zones (project duration = 1 year).

To give engineers the option of determining barrier need for any specific site, a computer program, which automates the benefit-cost analysis, was developed. This program has been adapted for use with microcomputers and made user friendly to facilitate its use by design engineers.

A set of use guidelines for work zone barriers has been developed for typical work zone sites. The guidelines were developed through application of the previously mentioned computer program. The guidelines are applicable to a wide range of traffic volumes, operating speeds, and construction zone characteristics. These simplified guidelines provide the practicing engineer with an objective criterion against which to estimate positive barrier need.

General conclusions from the study are as follows:

1. Barrier end treatment: Flaring the barrier away from the travelway is quite cost beneficial over a wide range of traffic conditions. The optimum flare rate is approximately 10:1 when the flare cannot be started within the construction zone. High-cost crash cushion end treatments are not generally warranted when the barrier can be flared away from the traffic lanes 20 ft or more, even at relatively high traffic volumes.
2. Bridge widening zones: Positive barrier use in bridge widening construction zones on high-speed facilities is generally cost beneficial even at low traffic volumes.
3. Roadway widening zones: PCB use in roadway widening zones involving simple pavement widening is not generally cost beneficial for projects with durations of 1 year or less. However, barrier use in construction zones involving vertical realignment is warranted for moderate traffic volumes.
4. Roadside structure construction: Positive barrier use in these zones is quite cost beneficial, even at traffic volumes as low as 1,000 ADT.

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