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Contents

Foreword

This Record contains papers on maintenance management, traffic safety in work zones, and roadside maintenance. The papers should be of interest to maintenance, traffic safety, and roadside vegetation professionals.

The maintenance management section contains two papers. The first describes a decision-making framework to improve the efficiency and effectiveness of management practices in Indiana, and the second presents the results of a study to identify differences between accomplishing construction projects and maintenance activities by public and private means. The work zone safety section contains eight papers that describe traffic control guidelines, use of drone radar, speed change distribution, nonpermanent pavement markings, durable fluorescent materials, nighttime visibility of pavement marking tapes, paint durability tests, and the use of remote weather information systems in winter operations. The roadside vegetation section contains four papers that present information on training material on the application of herbicides and plant growth regulators, the selective control of Johnsongrass in tall fescue, the development of native wildflowers, and the repair of road edge scour on grassed shoulders.

PART 1

Maintenance Management

Framework for Systematic Decision Making in Highway Maintenance Management

KUMARES C. SINHA AND TIEN F. FWA

The results of a 4-year research study that was undertaken in Indiana are presented. The objective of the study was to develop a systematic decision-making framework to enhance the efficiency and effectiveness of the existing maintenance management practice at the subdistrict highway agency level, where detailed maintenance programs are planned and implemented. The forms of data required and the recommended basis and procedures of decision making are discussed for the following areas: (*a*) assessment of maintenance needs, (*b*) establishment of performance standards, (*c*) determination of the costs of maintenance treatments, (*d*) setting up of an integrated data base, (*e*) priority rating of maintenance activities, and (*f*) optimal programming and scheduling of maintenance activities. Examples of planning data and information obtained from the research study are presented. The proposed decision-making framework is intended to be useful as an aid to management in the planning and monitoring of highway maintenance programs to obtain improved results from better use of available resources.

The weaknesses of the existing maintenance management practice in most state highway agencies may be summarized as follows: (*a*) maintenance needs assessments are made based on subjective judgment and experience of individual unit foremen, (*b*) maintenance work load estimates are established primarily by individual foremen on the basis of historical averages and their own judgment, (*c*) cost estimates for routine maintenance work are based on historical data that may not reflect the actual needs in the field, (*d*) routine maintenance programs are planned without effective coordination with major rehabilitation activities, (*e*) a data base is not available to provide the management with timely access to inventory and capital program data, and (*f*) routine maintenance activities are manually selected and scheduled by unit foremen through experience and judgment.

An improved procedure was developed for the management of maintenance activities in the Indiana state highway system that would eliminate or minimize many of the existing shortcomings. Figure 1 shows the main elements of the proposed highway maintenance management framework. The aim is to enable managers at the subdistrict level to plan a maintenance work program efficiently to achieve an optimal utilization of available funds. The planning of the maintenance program requires input that includes quantitative assessment of maintenance needs in terms of work loads and the relative importance of these needs ranked according to their priority ratings. Also required is a routine maintenance data base that contains relevant budget, cost, and performance information. Figure 1 indicates that the data base needs to be constantly updated to provide the most up-to-date information for planning purposes.

The present study identified the following six areas crucial to achieving the ultimate objective of maintaining and preserving the entire road network effectively:

- Development of a procedure to assess routine maintenance needs to minimize inconsistencies in the needs assessments made by different foremen;
- Establishment of maintenance performance standards to enable better estimates of manpower, materials, and equipment requirements to be made;
- Determination of cost functions for individual routine maintenance activities for reliable cost assessments;
- Setting up of an integrated routine maintenance data base system to ensure timely availability of planning information;
- Priority setting of various maintenance activities to promote adherence of accepted maintenance strategies; and
- Optimal programming and scheduling of routine maintenance work to produce a maintenance program that best satisfies maintenance needs with the available funds and resources.

Descriptions of these six areas in terms of their significance and operational features are presented in the following sections. The procedure is illustrated with data from the Indiana Department of Transportation. Indiana has six districts, each of which is subdivided into six or seven subdistricts. Within each subdistrict, there are two to four maintenance units that are directly responsible for routine maintenance work in the field.

ASSESSMENT OF ROUTINE MAINTENANCE NEEDS

The development of the proposed procedure for assessing routine maintenance needs consists of two parts: devising a reliable and practical procedure of highway condition survey and establishing quantity standards by which work load requirements for each routine maintenance activity can be computed.

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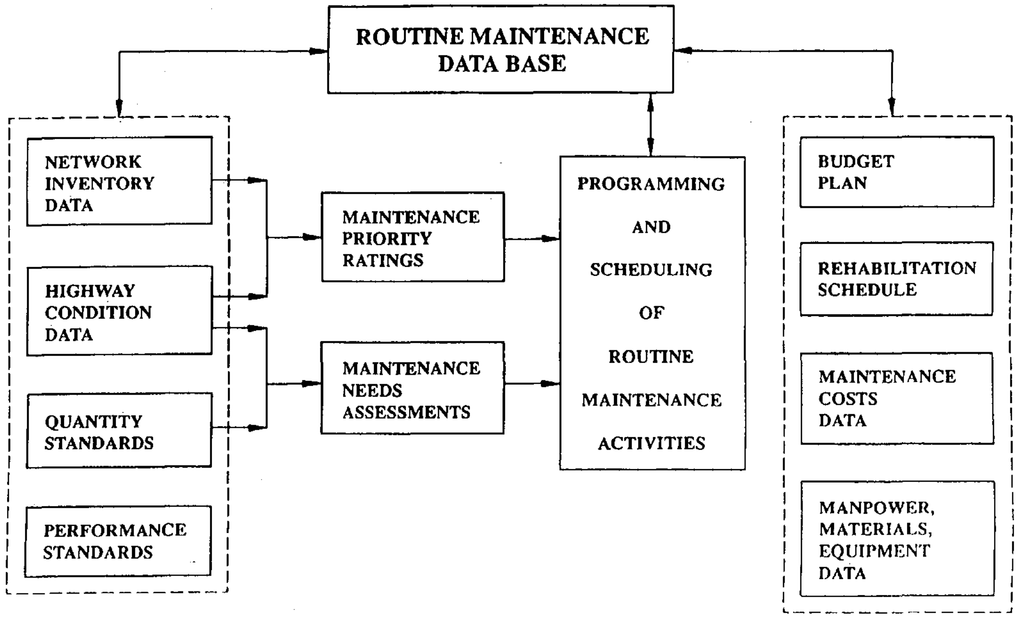

FIGURE 1 Elements of proposed highway maintenance management framework.

Highway Condition Survey

The current practice requires unit foremen to drive along the road to be inspected and report any deficiency using their own words. It was recognized in the study that a detailed condition survey completed with physical measurements of distress characteristics was not practical because it would be too time-consuming. It was, however, believed that more guidance could be provided to unit foremen in their field survey work and that standard descriptions and terms should be used in reporting. This led to the design of simple condition survey forms (*1*).

The distress types included in the survey forms were selected on the basis of past records of maintenance needs descriptions and finalized upon consultation with unit foremen and subdistrict maintenance management personnel. Each distress is identified by type, severity level, and extent or frequency of occurrence. Severity was considered in three categories: slight, moderate, and severe. The extent or frequency of occurrence was identified as many (m), some (s), or none (n). Exceptions are the descriptions for ditches and joints of concrete pavements, where the condition is classified as good, fair, or poor.

The use of the above procedure presented an improvement over the existing practice in that standard descriptions would be adopted in reporting of pavement distresses. This is especially important from the standpoint of quantifying maintenance needs, as will be described in the next section. It also provided a common reference for computing funding and resource requirements for different subdistricts on a statewide basis. In the long run, the availability of such systematically recorded distress data would be valuable for routine maintenance studies such as cost analysis of routine maintenance activities and effectiveness evaluation of routine maintenance treatments.

Choice of Maintenance Treatment

For any given distress type of a certain level of severity, several maintenance treatment alternatives are available to maintenance personnel. For example, Figure 2 shows relationships between common types of distress and maintenance treatments.

From the point of view of maintenance management, it is highly desirable that all subdistrict unit foremen be consistent in their choice of maintenance treatments. Whereas there is more than one maintenance treatment for a given distress, there is one treatment that will produce the best solution under the prevailing climatic and pavement conditions. The choice of maintenance treatments for correction of distresses can only be made more consistent through adoption of the most desirable treatment in every case.

A direct approach would be to conduct a survey to ask unit foremen to select the best maintenance treatment for each distress condition on the basis of their experience and judgment. A set of maintenance treatment selection guidelines may then be established on the basis of the collective opinion of unit foremen. Consultation with experienced maintenance personnel in Indiana indeed showed that there was also one preferred treatment for any particular distress.

A more rational approach, which is recommended in the present study, is to select the best maintenance treatment on the basis of cost-effectiveness considerations. This requires information on the cost and service life of different maintenance treatments. A survey was carried out in the study to determine effective service life for routine maintenance activities in the areas of pavement, shoulder, and drainage in Indiana (*2*). The effective service life of a maintenance treatment was defined as the time elapsed from application of the treatment to when, in the opinion of the foremen, it needed to be replaced.

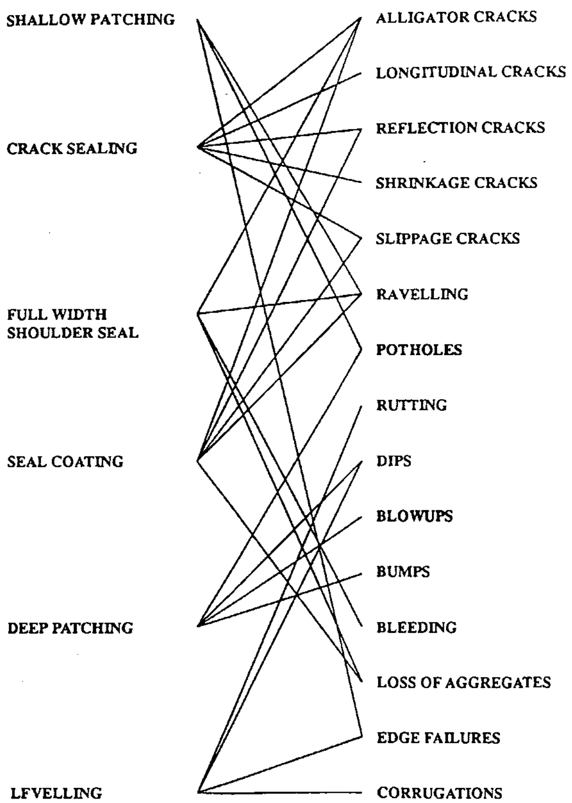

FIGURE 2 Relationship between maintenance treatments and distresses.

Because the effective service life of a treatment is highly dependent on the overall structural condition of the pavement concerned, estimates were obtained for the general condition levels of the entire roadway, namely poor, fair, and good. Estimates of minimum, average, and maximum effective service life were obtained from unit foremen. The average value can be used as a parameter for comparison purposes. It is an estimate by unit foremen of the average effective service life attainable when appropriate work practices are followed, with all necessary equipment, manpower, and time available to carry out the treatment work satisfactorily. These effective service life data can be combined with associated cost (to be discussed in a later section) to compute the desired cost-effectiveness of maintenance treatments.

Quantification of Maintenance Needs

Quantification of maintenance needs can be achieved by first identifying the appropriate unit of measure of work load for each maintenance treatment, followed by establishing quantity standards for work load estimation. Quantity standards express maintenance work load requirements in terms of appropriate units of measure for various distress types with different severity-extent combinations. The availability of such standards would help to reduce the uncertainties and variations involved in work load estimating in the existing practice that relies on the judgment of individual unit foremen. Two procedures were developed in this study for work load estimation: a statistical sampling approach and a computer-based expert system approach.

Statistical Sampling Approach

In this procedure the experience and collective know-how of subdistrict unit foremen were tapped to develop quantity standards for various distress type-severity-extent combinations. A statistical random sampling process was used to obtain representative values of foremen's estimates of expected work loads for various distress conditions. A statistical experiment was conducted to acquire the required information.

A total of 18 maintenance units were included in the study. The survey covered asphalt and concrete pavements in both the Interstate and the state highway systems. A stratified random sampling scheme (*3*) was used to select a total of 965 lane-mi of road for the experiment. The stratified random sampling scheme is a restricted randomization design in which experimental units are first sorted into homogeneous groups and then the required number of experiment units are randomly selected within each group. Unit foremen were asked to estimate the work load for every distress found on the test sections. A multivariable regression analysis, based on least squares fit, was used to develop equations that estimated maintenance work load on the basis of ratings of distress severity and frequency. Some examples of the derived quantity standards resulting from statistical analyses are given in Tables 1 through 4.

Computer-Based Expert System

An expert system using the LISP programming language (*4*) was developed to demonstrate its application in estimating maintenance work load. It can be used by subdistrict unit foremen to estimate maintenance needs on the basis of field observations of pavement distresses.

The program has three major components: input module, knowledge base, and output module. The input module is interactive in nature. It asks the user for information in the following two categories: (*a*) highway section geometric features, such as section length, number of lanes, lane widths, and shoulder widths; and (*b*) distress conditions such as distress type, severity, and frequency.

The knowledge base component stores all the rules. It has two distinct subdivisions. The first, known as the conversion module, converts qualitative assessment of distresses into numerical values. The second, the rules module, includes the rules to estimate maintenance work load requirements. The

TABLE 1 Example of Maintenance Quantity Standards for Work Load Computation—Clipping Unpaved Shoulder

	Frequency of Buildups	
Severity of Buildups	Some	Many
Slight	0.10	0.33
Moderate	0.25	0.50
Severe	0.45	0.90

NOTE: Quantity is in miles per shoulder mile (1 mi = 1.609 km).

TABLE 2 Example of Maintenance Quantity Standards for Work Load Computation—Shallow Patching

Severity of Potholes	Frequency of Potholes	
	Some	Many
Slight	0.50	1.20
Moderate	1.10	2.10
Severe	1.90	3.10

NOTE: Quantity is in tons per lane mile (1 ton/mi = 0.631 tonne/km).

lower and upper bounds of the 95 percent confidence interval for the estimated work load are computed.

The output module summarizes the estimated work load requirements for all the highway sections in standard units of measurements and displays all the values with proper titles and units. It also computes the estimated costs for various work load requirements when information on unit costs of maintenance activities is available. Figure 3 shows the input and output of an example problem.

PERFORMANCE STANDARDS FOR MAINTENANCE TREATMENTS

A maintenance treatment performance standard establishes the following for the maintenance treatment concerned: (*a*) the standard crew size needed, (*b*) the kinds and amount of equipment required, (*c*) the major types of materials that should be used, (*d*) recommended procedures for performing the work, and (*e*) an estimate of expected average daily accomplishment with standard crew size, equipment, and procedures. This information allows expected work load activity to be converted into manpower and work hour requirements.

Performance standards provide a basis for development of work programs and budgets at the subdistrict level. A set of performance standards is contained in the *Field Operations Handbook for Foremen* (*5*). This approach has functioned reasonably well in Indiana since it was implemented in 1975. However, a large variation in the average daily accomplishment of maintenance work still exists. Currently, a range of daily accomplishment quantity is specified for each maintenance treatment. However, it has been observed that daily accomplishments of maintenance work are dependent on roadway condition (*2,6*). An improvement in the estimation of daily accomplishment can therefore be made by identifying the accomplishment quantities for different roadway conditions.

A survey questionnaire was adopted for the maintenance work accomplishment investigation conducted in the present study. This survey was conducted together with the effective service life survey. Estimates of the number of accomplishment units attainable per day for each cell in the matrix for different maintenance treatments were obtained from the maintenance personnel interviewed. Table 5 gives examples of the survey results.

TABLE 3 Example of Maintenance Quantity Standards for Work Load Computation—Deep Patching, Potholes

Frequency of Potholes	Frequency of Bumps/Surface Failure		
	None	Some	Many
None	0.00	0.04	0.50
Some	0.10	0.50	1.30
Many	0.90	1.70	3.25

NOTE: Quantity is in tons per lane mile (1 ton/mile = 0.631 tonne/km).

TABLE 4 Example of Maintenance Quantity Standards for Work Load Computation—Deep Patching, Ditches

Condition of Ditch	Work Load (ft per ditch mile)
Poor	693.0
Fair	190.0
Good	2.0

NOTE: 1 ft/mi = 0.189 m/km.

COST OF MAINTENANCE TREATMENTS

Considerable research has been undertaken in Indiana in recent years into the cost of routine maintenance treatments (*7,8*). The general form adopted in the present study for estimating the costs of maintenance treatments is given by

$$T_k = \sum_i \sum_j F_{ijk} * R_{ijk} * C_{ijk} \tag{1}$$

where

T_k = total cost per production unit of the kth maintenance treatment (dollars),
F_{ijk} = usage factor of the jth element of the ith resource when required to produce one unit of the kth maintenance treatment,
R_{ijk} = rate of consumption of the jth element of the ith resource required to produce one unit of the kth maintenance treatment, and
C_{ijk} = unit cost of the jth element of the ith resource.

The usage factor, F_{ijk}, is calculated as

$$F_{ijk} = \frac{n_{ijk}}{N_k} \tag{2}$$

where n_{ijk} is the total number of jobs observed using the jth element of the ith resource in the kth maintenance treatment and N_k is the total number of jobs in the kth maintenance treatment.

The consumption rate, R_{ijk}, is obtained from

$$R_{ijk} = \frac{u_{ijk}}{U_k} \tag{3}$$

where u_{ijk} is the total number of units of the jth element in the ith resource used in the kth maintenance treatment and U_k is the total number of units of the kth maintenance treatment produced.

The cost components included were labor costs, materials costs, and fuel costs. Labor and materials unit costs were obtained from the Indiana Department of Transportation (*5*). Fuel consumption rates were obtained from the results of a field study conducted by Sharaf et al. (*7*).

INPUT			
ROAD SECTION LENGTH		:	10 MILES
NUMBER OF LANES - ONE WAY ONLY		:	1
LANE WIDTH		:	11.5 FT
INSIDE SHOULDER WIDTH		:	3 FT
OUTSIDE SHOULDER WIDTH		:	6 FT
CRACKS	SEVERITY	:	MODERATE
	FREQUENCY	:	MANY
POTHOLES	SEVERITY	:	SLIGHT
	FREQUENCY	:	FEW
RAVELING	SEVERITY	:	SLIGHT
	FREQUENCY	:	NONE
BLOWUPS / BUMPS / SURFACE FAILURES	FREQUENCY	:	MANY
RUTTING AND DIPS	SEVERITY	:	SEVERE
	FREQUENCY	:	MANY

OUTPUT	
AMOUNT OF WORK	
SHALLOW PATCHING	5.2 - 6.1 TONS
CRACK SEALING	1359 - 1584 GALLONS
FULL WIDTH SHOULDER SEALING	0 - 7 FT-MILES
SEAL COATING	0 - 1.7 LANE-MILES
DEEP PATCHING	8.6 - 9.8 TONS
LEVELING	161 - 162 TONS
ESTIMATED COSTS	
SHALLOW PATCHING	\$ 596 - 701
CRACK SEALING	\$ 2,827 - 3,294
FULL WIDTH SHOULDER SEALING	\$ 0 - 832
SEAL COATING	\$ 0 - 2,858
DEEP PATCHING	\$ 575 - 655
LEVELING	\$ 7,411 - 7,477

FIGURE 3 Example of maintenance work load estimation by expert system.

TABLE 5 Examples of Estimated Daily Accomplishment as a Function of Roadway Condition

Maintenance Treatment	Accomplishment Per Crew Day			Unit of Measurement
	Poor Roadway Condition	Fair Roadway Condition	Good Roadway Condition	
Shallow patching (Hot mix)	$\bar{x}$ = 7.2 σ = 1.5	$\bar{x}$ = 4.2 σ = 0.8	$\bar{x}$ = 2.8 σ = 0.5	Tons of Mix
Shallow patching (Cold mix)	$\bar{x}$ = 7.1 σ = 2.6	$\bar{x}$ = 3.9 σ = 1.2	$\bar{x}$ = 2.6 σ = 0.8	Tons of Mix
Premix leveling	$\bar{x}$ = 120.0 σ = 38.7	$\bar{x}$ = 88.6 σ = 36.6	$\bar{x}$ = 55.0 σ = 34.8	Tons of premix
Seal coating (Chip Seal)	$\bar{x}$ = 6.3 σ = 2.0	$\bar{x}$ = 6.9 σ = 1.7	$\bar{x}$ = 7.5 σ = 1.7	Lane miles
Seal coating (Sand Seal)	-*	$\bar{x}$ = 8.2 σ = 2.0	$\bar{x}$ = 8.2 σ = 2.0	Lane miles
Full width shoulder seal	-*	$\bar{x}$ = 73.5 σ = 16.8	$\bar{x}$ = 74.5 σ = 14.0	Foot miles
Sealing longitudinal cracks and joints	$\bar{x}$ = 6.3 σ = 2.1	$\bar{x}$ = 8.4 σ = 2.4	$\bar{x}$ = 10.2 σ = 3.8	Linear miles
Sealing cracks	$\bar{x}$ = 1.5 σ = 0.6	$\bar{x}$ = 3.0 σ = 0.9	$\bar{x}$ = 4.5 σ = 1.6	Lane miles
Spot repair of unpaved shoulders	$\bar{x}$ = 46.4 σ = 1.3	$\bar{x}$ = 30.5 σ = 8.2	-*	Tons of aggregate
Blading shoulders	$\bar{x}$ = 10.6 σ = 1.3	$\bar{x}$ = 13.2 σ = 2.2	-*	Shoulder miles
Recondition unpaved shoulders	$\bar{x}$ = 3.4 σ = 1.1	$\bar{x}$ = 4.5 σ = 1.1	-*	Shoulder miles
Clean and reshape ditches	$\bar{x}$ = 696 σ = 269.7	$\bar{x}$ = 1255 σ = 419.8	-*	Linear feet of ditch

Notes: (1) *indicates treatment is not applicable
(2) 1 mile = 1.609 km, 1 foot = 0.3048 m, 1 ton = 1.016 tons

ROUTINE MAINTENANCE DATA BASE

Extensive data are required for successful implementation of a highway maintenance management system. Figure 1 shows clearly the wide scope and diversity of the types of information needed for effective decision making in arriving at the final maintenance work program. The establishment of a routine maintenance data base with automated data handling and management capability is essential to efficient management of modern road networks. The data base facilitates collecting, storing, processing, and retrieving of information required in a maintenance management system. The following features are desirable:

- There must be a common referencing system to ensure compatibility and transferability of information derived from different data files.
- Linkages with information systems of other management levels (such as the central and the district levels) should be provided. Such linkages help to avoid duplicative data collection efforts. They also enhance coordination between work program planning at different management levels.
- The data base should be structured to facilitate constant updating to include the most up-to-date data. It must also allow for future improvement and expansion.

Types of Data

Three categories of data can be identified in Figure 1: subdistrict-specific data, policy and standards guidelines, and district and central office planning information. Subdistrict-specific data include network inventory data, highway condition data, and resources data. Network inventory data consist of highway functional classification, roadway geometry records, pavement structural characteristics, and roadside appurtenance records. Highway condition data are to be periodically updated through condition surveys performed by unit foremen, as described earlier. Resources data are information on the available manpower, materials, and equipment within the subdistrict concerned.

Policy and standards guidelines refer to quantity standards, performance standards, and cost information of maintenance treatments. The guidelines are essential for systematic decision making in maintenance management, and they reflect directly the maintenance policy of the central office. They are typically established on a statewide basis. The procedures for developing them have been presented earlier.

Budget plan and rehabilitation schedule come under the category of district and central office planning information. Unfortunately, as in many other states, the routine maintenance programs at the district and subdistrict levels in Indiana had not been effectively coordinated with major rehabilitation programs planned at the central office (*9,10*). Highway maintenance consists of corrective and preventive activities performed on a regular or continual basis, whereas rehabilitation includes major facility improvement such as replacement, reconstruction, overlays, resurfacing, and surface recycling. Although the criteria for the development of major highway facility replacement or rehabilitation programs may differ from those of routine maintenance programs, both programs have a goal of preserving the condition of the highway system. Effective coordination between the two programs can result in considerable savings. A major emphasis in the routine maintenance data base development undertaken in the current study was therefore to establish an efficient link between the two types of program.

Coordination of Maintenance and Rehabilitation Planning

After rehabilitation schedule information is made available to the subdistrict maintenance personnel, a natural question to ask is how this information could be used in maintenance planning. A highway agency would do certain adjustments to its routine maintenance program once the schedule of a rehabilitation project on a given highway section is known. Currently no specific information is available as to what maintenance treatments should be withheld or how long before the rehabilitation project such treatments should be discontinued. Because these are useful decision-making aids in maintenance planning, a project was initiated in this study to obtain the information (*11*).

The approach adopted was similar to the service life survey described earlier. A total of 36 representatives of maintenance staff were randomly selected from the subdistricts in the state. The factors included in the survey were maintenance treatment type, highway class, and pavement distress level of the highway section needing treatment. A suspension period was defined as the length of time before a rehabilitation work that a given maintenance treatment would not be carried out at all. Each maintenance staff member surveyed was asked to indicate the length of suspension period for different maintenance treatments by highway class and pavement distress severity level. On the average, for each highway class, more than two-thirds of the maintenance treatments surveyed had suspension periods longer than 3 months. This clearly indicates that the planning of most maintenance treatments can benefit from knowledge of the rehabilitation schedule.

PRIORITY RATING OF MAINTENANCE ACTIVITIES

Priority ranking of highway sections according to their maintenance needs is an integral part of a highway maintenance management system. It provides the required input for programming and scheduling maintenance activities. The relative priorities of different maintenance needs have a direct impact on the final outcome of a highway maintenance programming analysis. Because of the lack of priority information on routine maintenance activities in Indiana, a survey was conducted in the study to acquire the necessary data (*12*).

To reduce the rating items to a manageable size, a partitioned survey approach was used. In Partition I, priority scores were assigned by raters to individual maintenance activities in accordance with their relative importance in preserving the condition of a given highway section. In Partition II, priority scores were assigned to road sections of various highway classes by distress severity level according to the relative urgency of the need for maintenance treatments. Surveys for the two partitions were conducted separately.

The final priority ratings for all routine maintenance activities were computed as follows:

$$P_{ijk} = (f_{\mathrm{I}})_i \times (f_{\mathrm{II}})_{jk} \qquad i = 1, 2, \ldots, N_1; \quad j = 1, 2, \ldots, N_2; \; k = 1, 2, \ldots, N_3 \tag{4}$$

where

- P_{ijk} = priority rating for routine maintenance activity i on highway class j with distress severity level k, $1 \leq P_{ijk} \leq 100$;
- $(f_{\mathrm{I}})_i$ = priority score obtained from Partition I survey for routine maintenance activity i in relation to all other routine maintenance activities, $1 \leq (f_{\mathrm{I}})_i \leq 10$;
- $(f_{\mathrm{II}})_{jk}$ = priority score obtained from Partition II survey for combination of highway class j and distress severity level k in relation to all other combinations of the two factors, $1 \leq (f_{\mathrm{II}})_{jk} \leq 10$;
- N_1 = total number of routine maintenance activity types;
- N_2 = total number of highway classes; and
- N_3 = total number of distress severity levels.

Experience gained from the survey indicated that the rating procedure was well received by raters, and satisfactory results were obtained. The final form of priority ratings is given in Table 6.

OPTIMAL PROGRAMMING OF MAINTENANCE ACTIVITIES

At the subdistrict level, maintenance units have to perform diverse routine maintenance activities on a large number of highway routes over extended areas. Because of constraints of resources, not all maintenance needs identified can be attended to as and when required. Selection of highway sections to be included in a maintenance work program has so far been made on a subjective judgmental basis in Indiana. To ensure that consistent decisions are made by different subdistrict maintenance personnel that achieve the best return for the funds and resources committed, an analytical optimization tool for maintenance activities programming is needed. In the present study, an integer-programming optimization model was recommended for maintenance management at the subdistrict level in Indiana. The detailed mathematical formulation of the model is described elsewhere (*13,14*). The major components of the model are explained below.

The objective function of the model was to maximize total work units within the analysis period to accomplish the needed maintenance treatments as much as possible according to their relative priority ranking. In doing so, it was first necessary to convert work measurement units into a common basis of reference. Equivalent workday was chosen because routine maintenance tasks are assigned to field crews on a daily basis in Indiana. Such tasks are authorized daily at the subdistrict

TABLE 6 Examples of Priority Ratings of Routine Maintenance Activities by Highway Class and Distress Severity Level

Routine Maintenance Activity	Interstate			High Volume OSH*			Low volume OSH*		
	Distress Severity Level			Distress Severity Level			Distress Severity Level		
	Severe	Moderate	Slight	Severe	Moderate	Slight	Severe	Moderate	Slight
Shallow patching	99	86	62	93	77	43	73	49	10
Deep patching	96	84	60	90	75	41	71	47	10
Premix leveling	72	63	45	68	56	31	53	35	7
Full-width shoulder seal	49	43	31	46	38	21	36	24	5
Seal coating (chip seal)	64	56	40	60	50	28	47	31	6
Sealing longitudinal cracks and joints	67	58	42	63	52	29	50	33	7
Crack sealing	68	59	43	64	53	29	50	33	7
Sand seal	56	49	35	53	44	24	41	27	6
Spot repair of unpaved shoulders	78	68	49	73	61	34	58	38	8
Blading shoulders	70	61	44	67	55	30	52	34	7
Clipping unpaved shoulders	46	40	29	43	36	20	34	23	5
Reconditioning unpaved shoulder	42	37	26	39	33	18	31	21	4
Clean and reshape ditches	37	32	23	35	29	16	27	18	4
Motor patrol ditching	19	17	12	18	15	8	14	9	2

*High volume OSH are other state highways that carry 400 or more vehicles per day. Low volume OSH are other state highways that carry less than 400 vehicles per day.

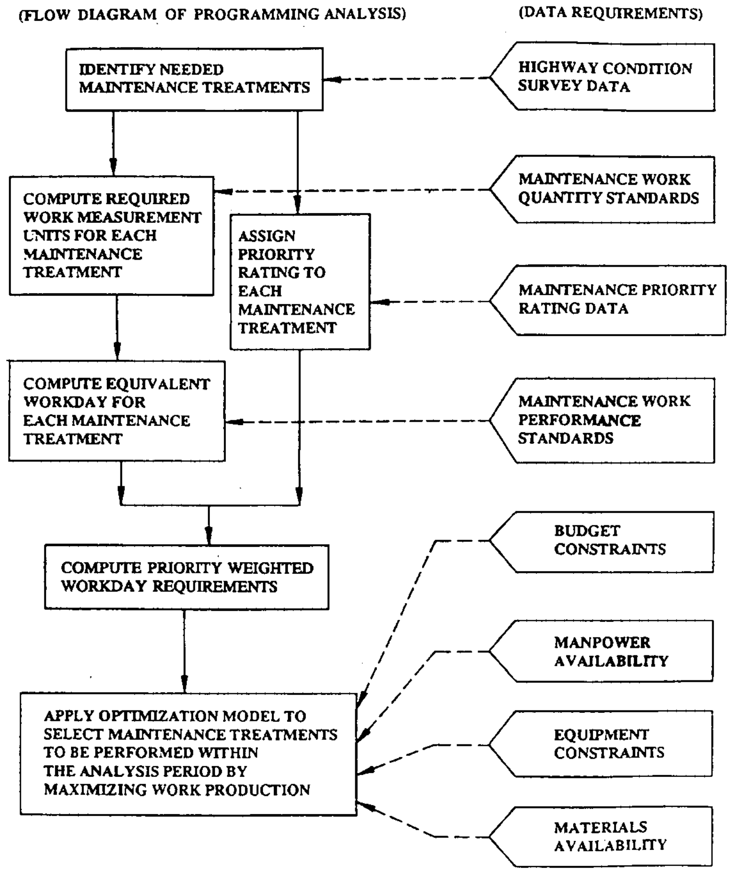

FIGURE 4 Programming maintenance activities using optimization model.

level by unit foremen to each crew by means of crew day cards (*5*). There is also a well-defined relationship between work quantity and workdays established by the performance standards (for example, see Table 2). Expressing work quantity of a routine maintenance treatment in terms of equivalent workdays therefore has a direct practical meaning easily understood by both field and planning personnel.

Six forms of constraints were considered in the model. They were production requirements, budget constraints, manpower availability, equipment availability, material availability, and rehabilitation schedule constraints. Production requirements simply state that the amount of maintenance work assigned for each treatment type should not exceed the need for it. Budget, manpower, equipment and material availabilities represent resources constraints, which ensure that the total amount of maintenance work selected will be within the means of the subdistrict concerned. Rehabilitation constraints are specified such that unnecessary maintenance treatments will be suspended on highway sections that have been scheduled for rehabilitation.

Figure 4 shows the steps involved in the programming analysis of routine maintenance activities and the data requirements for the analysis. Besides functioning as a tool for programming maintenance work, the model can also be used to analyze the impacts of shortfalls of resources. Possible benefits of reallocating resources can be investigated by performing parameter sensitivity analysis. The amount of certain resources to be made available may be adjusted to achieve better results. For example, the number of temporary laborers to be hired over a given period of the year could be determined by means of such analyses.

SUMMARY AND CONCLUSIONS

This paper addressed the important elements in a routine maintenance management system proposed for the Indiana Department of Transportation. Six key areas of concern were highlighted and discussed in detail: (*a*) maintenance needs assessments, (*b*) establishment of performance standards, (*c*) determination of the costs of maintenance treatments, (*d*) setting up of an integrated data base, (*e*) priority rating of maintenance activities, and (*f*) optimal programming and scheduling of maintenance activities. The types of data required and the procedures for acquiring them were explained in each case. The proposed framework can be followed by other highway agencies with appropriate modifications.

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Project Cost Evaluation Methodology Approach to Privatization in the Washington State Department of Transportation

JOHN F. CONRAD, PAUL NELSON, AND KELLY JONES

For many years there has been an ongoing debate between the private and public sectors as to which entity can accomplish highway construction or maintenance work of better quality at lower cost. In 1985 the Washington State legislature commissioned a study to better understand the differences and develop cost comparisons. The initial study objectives were to review existing roadway accounting and cost practices, develop a new methodology for comparing public and private contractor costs on a project-by-project basis, and recommend changes in laws and regulations. The Project Cost Evaluation Methodology (PCEM) was developed as a process that organizes cost-estimating data, puts projects out for private- and public-sector bids, documents award decisions, captures actual costs for comparison, and reports results so that comparisons can be made on a project-by-project basis. The Washington State Department of Transportation participated in a 3-year test of the methodology in which both construction and maintenance projects were put through the PCEM process. The results of the maintenance element of study have demonstrated that there is more potential for savings if more flexibility were available. The testing of the PCEM process has demonstrated that there is no clear-cut final answer to the question of whether the public or the private sector can accomplish work at lower cost. PCEM has demonstrated the need to evaluate projects individually when change from current practice is contemplated.

Throughout the years there has been an ongoing debate between the private and public sectors as to which entity can accomplish highway construction or maintenance work of better quality or at lower cost. Within Washington State, these decisions are currently guided by artificial bid limits or day labor requirements. The Washington State Department of Transportation (WSDOT) is constrained by a bid limit of $30,000. Identifiable maintenance or construction projects are subject to contract if they exceed $30,000. Virtually all of the major maintenance programs for resurfacing or other roadway surface treatments in excess of 500 ft are considered a part of the construction program and are contracted out.

On the other hand, the WSDOT maintenance program is also constrained by court rulings, labor agreements, and state law. Washington State statute prohibits contracting out any work done by state employees before April 23, 1979. Accordingly, elements of the maintenance program not identifiable as construction projects are done with agency forces. Less than 5 percent of the total maintenance budget is contracted out.

The bid limits were the subject of intense lobbying from both the public and private sectors. In 1985 the Legislative Transportation Committee (LTC) commissioned a study of roadway project costing. The study was directed by a steering committee appointed by the LTC. The committee included representatives of the LTC, Department of Transportation, labor, contractors, cities, and counties. A consultant team of Deloitte & Touche (Deloitte, Haskins & Sells) and Tudor Engineering was selected to conduct the study. The study objectives were to review existing roadway accounting and cost practices, develop a new methodology for comparing public and private contractor costs on a project-by-project basis, and recommend changes in laws and regulations.

Several issues were identified for evaluation during the study:

- Project cost accounting systems,
- Level playing field,
- Local tax impact of contracting out work,
- Overhead cost allocation,
- Accounting for materials,
- Accounting for equipment,
- Inspection and quality control requirements,
- Impact of bid limits and day labor requirements,
- Labor and union agreements,
- Interagency contracting,
- Self-insurance costs,
- Definitions of construction and maintenance, and
- Essential services provided by government agencies.

PROJECT COST EVALUATION METHODOLOGY

The steering committee agreed with the consultant that it was not practical to develop a cost comparison methodology that would take into account each of the adjustments that would be necessary to address all of the issues. A new methodology was developed to

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- Capture all indirect and direct costs;
- Be simple and practical to use;
- Give results that are accountable and open for review;
- Account for real world constraints such as emergency conditions, manpower constraints, project timing, and so forth; and
- Achieve cost savings.

The Project Cost Evaluation Methodology (PCEM) was developed by the consultant in response to the issues and criteria identified by the steering committee (*1*). The PCEM approach recognizes that it is impossible to develop a model that ignores the fundamental differences between the public and private sectors. However, PCEM organizes cost-estimating data, documents award decisions, captures actual costs for comparison, and reports results such that a "fair" comparison can be made. The PCEM approach consists of six main activities, which ultimately lead to a comparison of public- and private-sector bids on a project-by-project basis. The process does not lead to a "final" determination of whether public or private is "best." Rather, it is a continuing process to evaluate projects and make decisions on an individual basis. The PCEM process uses a series of forms (see Figures 1 through 6).

The project control summary documents the entire budget and serves as a master list to track the projects during the completion of the process.

PCEM Part A forms document the budget amount for each activity and the decision as to whether these activities are biddable. For example, an agency may decide that certain activities such as snow removal are too critical to be contracted out. Other activities may be kept in-house because of a need to maintain current practice due to work force requirements.

PCEM form Part B is completed for activities that may be available for bidding. PCEM Part B consists of a series of forms that require the agency to prepare an estimate of agency costs, including labor, materials, and equipment necessary to complete the project. The costs include estimates for project management, overhead, and inspection to be comparable with private-sector bids. The forms and estimates are completed by the agency in the context of a "bid" and are compared with bids submitted by contractors following a formal project advertisement for bids.

Whereas preparation of specifications and bid documents is a normal process for traditional contracting of construction projects, it is necessary to prepare similar documents for work traditionally done in-house that will now be put through the PCEM process.

Actual costs and quantities must be collected for PCEM projects that are completed with agency forces as they would be for contracted work. If the costs and quantities are different from the bid, this information is documented to track whether the project was actually completed for less than

PCEM PROJECT CONTROL SUMMARY

Agency: ______________________ Maintenance? _____ Date: __________
Construction? _____

Budget Item	Reference Number	Budget Dollar Amount	Part A		Part B		Bid			Are Spec's:		Date to Bid	Work Start
			Complete?	$	Complete?	$	Y/N	$	Avail?	Needed?	When?		
Total:		______		___		___		___					

FIGURE 1 Project control summary.

Project Cost Evaluation Methodology (PCEM)

Part A Preliminary Decision Alternatives

Agency: WSDOT Dist. 4	**Year:** 1990	**Will project cover more than one budget year?**	**Yes** ____	**No** ✓	

Budget Item Description:	Shoulder Residual Herbicide Spraying	**Reference Number:**	1875A
		Today's Date:	4/18/90
Maintenance Budget?	✓	**Construction Budget?**	____

Budget Item Estimate: $23,971

1. **Is the budget item work activity(ies) of such scope that it would automatically go out for bid?** ____ **Yes** ✓ **No.** Go to Question 2

   If yes:

   1a. __ Inadequate equipment available
   1b. __ Exceeds agency technical expertise
   1c. __ Other ____________

2. **Is the budget item work activity(ies) of such scope that it would be constrained to agency force labor?** ____ **Yes** ✓ **No.** Go to Question 3

   If yes:

   2a. __ Quick project response time needed
   2b. __ Budget too small to warrant cost effective bid
   2c. __ Emergency/liability issues
   2d. __ Other ____________

3. **If Questions 1 and 2 are "No," use Part B of PCEM Worksheets to develop the agency bid and document the lowest contractor bid.**

Prepared By: ____________	Title: ____________	Date ____________
Reviewed By: ____________	Title: ____________	Date ____________

FIGURE 2 Preliminary decision alternatives.

the "unsuccessful" bid amount adjusted for changes in quantities.

Just as in normal contracting, projects through PCEM must be inspected even if the work was completed with agency forces. Differing levels of inspection are necessary for different kinds of work, such as brush cutting or asphalt patching.

TESTING THE PCEM PROCESS

Legislative Action

Following the development of the PCEM, the steering committee elected to test the new methodology against real world maintenance and construction. The Washington State legislature enacted RCW 47.28.180, which, until June 30, 1991, required agencies participating in the project to "use the project cost evaluation methodology for evaluation of projects. The projects shall be performed based on the lowest estimated cost regardless of who had performed the work historically." Six cities and five counties volunteered to participate in the methodology test under these conditions.

Unfortunately, in response to labor concerns, the bill further specified that WSDOT participate in the project with a portion of one or more of its districts but that the PCEM process be used only to evaluate its projects and draw conclusions as to which projects would have been done in-house and which would had been contracted out had the quoted flexibility been available.

This so-called "shadow approach" only allowed WSDOT to identify projects, prepare in-house bids, call for bids from contractors, track actual agency costs for completing the projects, and then compare the results. No projects could actually be awarded to private contractors. Whereas this dampened the enthusiasm of the districts and private contractors, a good faith effort was made by all parties to give the process a valid test.

Project Cost Evaluation Methodology (PCEM)

Part B1 Comparison of Cost Estimates

Agency: Washington State Department of Transportation

Project Name: Shld. Res. Herb. Application **Project Number:** 1875 A

Related Budget Item (i.e., Part A): ______________________

Project Description: Vegetation control on state highway shoulders in Clark County Area 5 of District 4.

Historically Performed by Agency? **Yes** _____ **No** _____

Summary Cost Comparison

Agency	Estimated	Actual	Lowest Contractor Bid	Estimated	Actual
1. Agency Cost	21,027	30,070	1. Contractor Price	31,185	
2. Project Administration	2,944	750	2. Project Administration	6,237	
3. Direct Project Cost	23,971	30,070	3. Direct Project Cost	37,422	
4. Overhead (On Direct Labor) (@59.1%)	7,411	4,250	4. Overhead (On Agency Proj. Admin) (@_____%)	3,686	
5. **Total Project Cost**	31,382	35,070	5. **Total Project Cost**	41,108	

Note: Please submit one copy of each Part B.1 after evaluating bids (without actual costs) and another copy after work is completed (which includes actual cost).

Alternative Selected: __X__ Agency ____ Contractor

If the lower cost alternative was not selected, please explain:

____

____

____

____

Estimate Prepared By: ______________________ Date __________

Selection Approved By: ______________________ Date __________

Post Project Review By: ______________________ Date __________

FIGURE 3 Comparison of cost estimates.

Problems with Implementation

A test period of three construction seasons (1988–1990) was required to learn the process and adequately test it with viable projects. Typical problems for WSDOT and local agency participants included the following:

- A lack of bid specifications: Work that had been traditionally done in house did not previously require specifications. The development of these specifications as well as bid documents was a learning process for personnel not previously involved in these processes.
- Incompatibility of budgets with project identification: The WSDOT maintenance budget consists of 10 major work groups and subcategories of work functions and work operations (e.g. roadway maintenance—asphalt patching—roller operation). Maintenance superintendents were not used to planning the work according to predetermined projects. Rather, work was identified and completed in generally small increments as conditions and availability of work force and equipment allowed. This proved to be a continuing problem throughout the course of the study.
- Traditional construction projects: WSDOT has long completed its construction program through private-sector contracting. Whereas PCEM was intended to provide an opportunity to test the completion of construction work by in-house forces, maintenance personnel do not have sufficient labor, equipment, or expertise to complete major construction work, which is typically completed by private-sector contractors. As a result, it was difficult to develop any viable bids for typical construction work.
- A contractor distrust of the process and a lack of interest in participation: Some projects failed to generate any contractor bids. An extensive public information campaign was

Project Cost Evaluation Methodology (PCEM)

Part B2 Agency Cost Estimate
*For Fixed Price Contracts

Project Name: Shld. Res. Herb. Application **Project Number:** 1875 A
Agency: WSDOT, District 4 **Date:** 4/18/90

Direct Labor Hours	**Labor Estimate**	
Labor Type		
Maint. Tech. II Spray Operator	97 Hours @ $19.73 /Hour = $1,913.81	
Maint. Tech. II Driver	97 Hours @ $18.50 /Hour = $1,794.50	
Maint. Tech. II Traffic Control	41 Hours @ $18.50 /Hour = $ 758.50	
______	__ Hours @ $____ /Hour = $______	
______	__ Hours @ $____ /Hour = $______	
______	__ Hours @ $____ /Hour = $______	
(1) **Direct Labor Subtotal**	$ 4,466.81	
Fringes (@ incl. %)	$ -0-	
(2) **Labor Total**		$ 4, 466.81

Material Estimate				
Type	**Quantity**	**Cost/Unit**	**Cost**	
Krovar	480.81 kg.	7.29	$7,727.40	
Diuron	377.02 kg.	3.91	$2,903.24	
Roundup	201.45 L	60.35	$3,198.55	
R-11 Surfactant	201.45 L	9.02	$478.06	
Sta-Put Drift Retardant	201.45L	7.83	$414.99	
(3) **Material Total**				$14,722.24

Equipment Estimate				
Type	**Quantity**	**Cost/Unit**	**Cost**	
6D6 Truck	97 hrs.	8.80	$853.60	
21035 Sprayer	96 hrs.	8.75	$848.75	
5D3 Pickup	41 hrs	1.06	$43.46	
10D9 Arrow Board Trailer	41 hrs	2.25	$92.25	
(4) **Equipment Total**				$1,838.06

(5) **Total Agency Cost Estimate** (Total Lines 2,3, and 4)	$21,027.11
(6) **Project Administration Cost** (Must be developed prior to bid opening) (____ Hours @ $____/Hour & Fringes @____%)	$2,943.80
(7) **Direct Project Cost** (Line 5 plus 6)	$23,970.91
(8) **Agency Overhead on Direct Labor** (@ ______%)	$7,410.61
(9) **Total Project Cost** (Total Lines 7 and 8)	$31,381.52

FIGURE 4 Agency cost estimate for fixed price contracts.

directed at increasing the contractor's knowledge of and participation in the project. Some projects failed to generate bids because the contractors were too busy with work traditionally awarded to them.

Project Results for WSDOT

The dollar impact of PCEM on projects can be identified in two areas. First, direct savings are measurable by comparing the actual costs of the project with the bid submitted by the agency or the private contractor that normally would have done the work. Second, indirect savings are not readily measurable, but in general include efficiency gains that should occur over time through improved crew productivity, competitive bidding with better bids, and improved methods and procedures. The results of WSDOT participation are measured only in terms of direct savings.

During the 3-year test of PCEM, WSDOT applied the process to both construction and maintenance projects. Typical maintenance projects included brushing, mowing, herbicide spraying, striping, signpost installation, raised pavement marker replacement, sand hauling, guidepost replacement, and safety berm construction. A total of 21 small maintenance projects with a total budget estimate of $530,000 were tested. Had the shadow approach not been required, seven of these projects normally done with in-house labor would have been awarded to private contractors. Had awards been made on the basis of lowest bids, it was estimated that approximately $47,000 would have been saved.

It was more difficult to apply the process to construction projects because the majority of these projects are beyond the scope and capability of WSDOT maintenance staff capabilities. To attempt to test PCEM for construction, agency force bids were prepared for selected bid items in larger con-

Project Cost Evaluation Methodology (PCEM)

Part B2 Agency Cost Estimate
*For Multiple Bid Item Projects

Project Name:	Shld. Res. Herb. Application	**Project Number:**	1875 A
Agency:	WSDOT, District 4	**Date:**	4/18/90

Agency Cost Estimate

Bid Item	Estimated Quantity	Cost/Unit	Bid Cost
Shldr. Res. Herb. Spraying	85.793 H	99.18	21,027.11

(1) **Total Agency Cost Estimate**	$21,027.11
(2) **Project Administration Cost** (Must be developed prior to bid opening) (- Hours @ $ - /Hour & Fringes @ incl. %) 14%	$2,943.80
(3) **Direct Project Cost** (Line 1 plus 2)	$23,970.91
(4) **Agency Overhead** (@ 59.1 %) **on Direct Labor Estimate of** ($______)	$7,410.61
(5) **Total Project Cost** (Line 3 plus 4)	$31,381.52

FIGURE 5 Agency cost estimate for multiple bid item projects.

struction contracts. These were construction projects being completed as a part of the normal WSDOT private-sector construction program and were not readily suitable for PCEM. Nevertheless, 26 bids were prepared for the selected bid items contained in these projects. Typical work included guardrail installation, striping or temporary traffic signals, culvert repair, and bituminous surface treatments. An analysis of the results indicates that WSDOT bids for selected work items were less than contractor bids in 15 instances. However, most of these results are not considered a valid test since contractors bid projects in their entirety, and the individual bid items may be over-or underloaded based on the application of overhead or cashflow considerations.

Although not directly the subject of this paper, the PCEM process was more extensively tested with city and county projects. The primary reasons for the more extensive testing were the larger number of participants (11) and the ability to actually award projects to private contractors. For all agency participants, there were a total of 68 maintenance projects bid at a total of $3.5 million in the third year of the project alone. Direct measured savings totaled $326,000 (*2*). These are actual savings and validate the savings measured through the shadow approach used by WSDOT.

CONCLUSIONS AND RECOMMENDATIONS

WSDOT has traditionally contracted the major portion of its highway construction program. Whereas the participant districts made a good faith effort to participate in the process with construction projects, the results have confirmed that WSDOT does not intend to make fundamental changes in the way construction program contracts are awarded to the private sector. There is a limited potential for construction work to be completed with agency forces. The current bid limit of $30,000 represents an artificial barrier to WSDOT's ability to

Project Cost Evaluation Methodology (PCEM)

Part B3 Lowest Contractor Price Estimate

Project Name:	Shld. Res. Herb. Application	**Project Number:**	1875 A
Agency:	WSDOT, District 4	**Date:**	4/18/90
Contractor:	P. S. G. Chemical Applicators		

Contractor Price Estimate

Bid Item	**Estimated Quantity**	**Price/Unit**	**Bid Price**
Shldr. Res. Herb. Spraying	85.793 H	147.10	31,185.20

(1) **Total Contractor Price Estimate**	$31,185.20
(2) **Agency Project Administration Cost** (Must be developed prior to bid opening) (- Hours @ $ - /Hour Fringes @ incl. %) 14%	$6,237.04
(3) **Direct Project Cost** (Line 1 plus 2)	$37,422.24
(4) **Agency Overhead** (@ 59.1 %) **on Project Administration Direct Labor** (Line 2)	$3,686.09
(5) **Total Project Cost** (Line 3 plus 4)	$41,108.33

Are there expectations that operational factors would have a major impact on awarding the project? ____ Yes ____ No

If yes, explain: ____________________

FIGURE 6 Lowest contractor price estimate.

use its own forces for the limited portion of work that could be done with its own forces, and state law and union agreements prevent contracting of work that could be better done by the private sector.

The results of the maintenance element of study participation have demonstrated that there is more potential for change within WSDOT if more flexibility were available. The bid limit of $30,000 and state law and union agreements represent artificial constraints on the most efficient way of managing the maintenance program. The savings identified from the maintenance projects is consistent with the expected 8 to 10 percent identified by the consultant team and is a realistic expectation should artificial constraints be removed.

The consultant team's conclusions are that the PCEM process, when properly applied, provides an effective decision-making tool and provides potential cost savings and better utilization of resources. Further, PCEM is an efficient tool for many, but not all, decision situations. It is efficient for larger projects, with a single quantifiable objective over a discrete location or area (asphalt work). It is less efficient for smaller, less defined projects with specifications more subject to interpretation (street cleaning). PCEM can be implemented but should not be mandated. PCEM is recommended for use by agencies at the discretion of management or at the request of contractors after review of agency work plans. Bid limits or day labor requirements would not be in effect for agencies using PCEM. Savings of 8 to 10 percent are considered to be reasonable expectations of implementation (*2*). It is expected that these recommendations will be presented to the Washington State legislature during the 1993 legislative sessions.

Most of the work anticipated for potential PCEM application for WSDOT falls within the category of smaller, less defined projects with specifications more subject to interpretation or of such small dollar value that there is limited contractor interest. As such, full implementation of the PCEM concept within WSDOT is not recommended. Rather, a limited project-by-project approach should be undertaken. Projects proposed for possible change from current practice (beyond $30,000 with state forces or contracting work currently done with state forces) should be subject to a PCEM or similar economic analysis. In many instances a full detailed PCEM approach would not be justified to accomplish projects at the lowest cost for the taxpayer. Such items as detailed contract plans or project inspection may not be necessary on every project.

Whatever approach is used, the PCEM process has demonstrated that there is no clear-cut final answer to the question of whether the public or private sector can accomplish work at the lowest cost. PCEM has demonstrated the need to evaluate projects individually when change is contemplated.

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PART 2

Traffic Safety in Work Zones

Traffic Control Guidelines for Urban Arterial Work Zones

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Urban arterial work zones have several unique characteristics that have not been adequately addressed in previous research. These characteristics were identified, and traffic control guidelines for urban arterial work zones were developed. Significant research activities included a review of previous literature and current practice, analysis of traffic and accident data, surveys of motorists, measurement of the capacity of an arterial lane closure, and a signal operations analysis near a lane closure. The literature review indicated a lack of previous research, although limited portions of other work zone research were found to be useful. The review of current practice explored the opinions of traffic engineers and examined work zone traffic control manuals of local agencies. Accident, volume, and travel time data were collected and analyzed to determine trends specific to arterial work zones and to identify characteristics that needed to be addressed in the guidelines. Motorist surveys attempted to evaluate driver comprehension of work zone traffic control devices and to identify some of the more significant driver concerns about urban arterial work zones. Other study activities included investigations into arterial lane closure capacity and the relationship between traffic signals and lane closures. The research results were used to develop guidelines addressing traffic control for urban arterial work zones. Use of these guidelines should help to improve safety and traffic flow in arterial work zones.

Urban arterial work zones have several unique characteristics that distinguish them from rural highway or freeway work zones. The characteristics are primarily related to traffic conditions, traffic signals, geometrics, and limitations on work zone traffic control. Among the most important of the characteristics are higher speed variations, highly variable volumes, limited maneuvering space, frequent turning and crossing maneuvers, multiple access points, higher pedestrian volumes, frequent traffic obstructions, greater competition for driver attention, and more traffic signals. These characteristics require special consideration when preparing a traffic control plan for construction activities. Unfortunately, urban arterial work zones are not sufficiently addressed in current work zone guidelines, and the topic has not been adequately researched in the past. A recently completed research study sponsored by the Texas Department of Transportation (*1*) evaluated some of the unique characteristics of urban arterial work zones and developed guidelines for traffic control in this type of work zone.

STUDY ACTIVITIES

The guidelines were developed through the completion of several research activities, which included a review of pertinent literature and current practices, an analysis of traffic and accident data, two surveys of motorists, the measurement of the capacity of an urban arterial lane closure, and the analysis of signal operations near a lane closure.

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Literature Review

Previous work zone research has traditionally focused on freeway and rural highway work zones. Therefore, it was not surprising to find that there have been few research projects specifically addressing urban arterial work zones. However, some limited portions of other work zone research that could be applied to urban arterials were identified in the course of the review. Most useful information is located in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*2*) and the *Traffic Control Devices Handbook* (TCDH) (*3*), although these documents do not specifically address the needs of urban arterial work zones. Because of the limited amount of previous research on urban arterials, most of the guidelines developed in this study were the result of other research activities.

Current Practice

The lack of previous research created the need to identify the current traffic control practices being used in urban arterial work zones. These practices were identified through discussions with traffic engineers at local transportation agencies and examinations of work zone traffic control manuals produced by a number of local agencies.

The discussions with traffic engineers indicated that there is variation in the emphasis given to traffic control for arterial work zones. Several engineers indicated the MUTCD did not sufficiently address traffic control for arterial work zones and that the more significant problem areas involve intersections and intersection-related traffic control. Several individuals described the benefits of having one or more inspectors whose only responsibility was inspecting traffic control in the work zones.

Traffic control manuals produced by local agencies rely heavily on the MUTCD, although some agencies have modified the MUTCD guidelines. Table 1 indicates the variability in sign spacing existing between several agencies.

Accident and Traffic Data Analysis

Some of the more obvious characteristics of urban arterial work zones can be identified by simple comparisons with other

TABLE 1 Comparison of Arterial Work Zone Sign Spacings

Speed - kph (mph)	Distance Between Signs - meters (feet)				
	TCDH (3)	TxDOT (4)	Arlington (5)	Seattle (6)	Victoria (7)
48 (30)	76 (250)	24 (80)	38 (125)	46 (150)	not available
56 (35)	76 (250)	37 (120)	49 (160)	Note: Seattle specifies sign spacing according to roadway classification, not roadway speed	46-73 (150-240)
64 (40)	76-153 (250-500)	49 (160)	61 (200)		not available
72 (45)	153 (500)	73 (240)	76 (250)		73-110 (240-360)
81 (50)	153 (500)	98 (320)	92 (300)		not available

types of work zones. On the other hand, other characteristics can only be ascertained by analyzing traffic data from urban arterial work zones. In this study, accident, volume, and travel time data were collected at three urban arterial work zone sites. Each site was a four-lane arterial being widened to six lanes. Two sites were in Houston and the third was in Dallas.

The safety impacts of work zones on urban arterials were assessed by comparing preconstruction and during-construction accidents in several different categories, including accident frequency, accident rates, accident types, causes of accidents, locations of accidents, and accident periods. Statistical analysis of the data indicated that there are overall increases in accident frequency and rate in an urban arterial work zone. The increase in frequency ranged from 35 to 77 percent, and the increase in rate ranged from 59 to 106 percent. The arterial construction appears to have caused a statistically significant increase in the number of accidents occurring at or near intersections and driveways and in the number of nighttime accidents. Figure 1 shows the accident rate for arterial segments at one of the study sites. The figure shows the increase in accidents at intersection and driveway locations during the period of construction.

Average weekday traffic volumes and average travel times were also collected to identify the traffic flow characteristics of arterial work zones. However, the traffic volume and travel time data were highly variable and did not indicate any trends unique to urban arterials. It appears that the traffic volumes are lower when the construction area is located in the middle of the roadway between opposing traffic flows. The data indicated a wide variation in the traffic volumes, and therefore the traffic control plan should be prepared to accommodate traffic volumes that are comparable with preconstruction volumes. Increases in travel time through the work zones could not be attributed to changes in traffic control within the work zone.

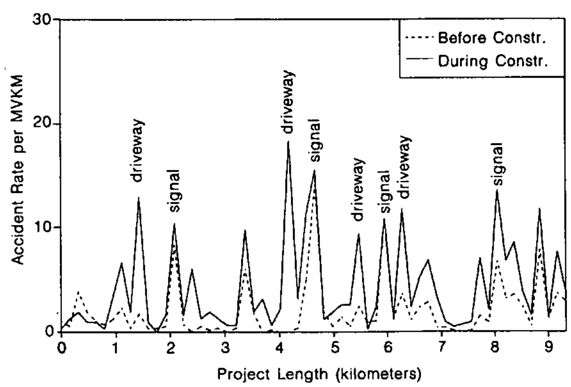

FIGURE 1 Accident rate by arterial segment.

Motorist Surveys

Two motorist surveys were conducted near the study sites (*8,9*). The surveys evaluated motorist understanding of work zone traffic control devices and identified motorist concerns related to construction activities. The surveys indicated that drivers are more concerned with issues such as the length of the project, duration of construction, and travel delay than they are with traffic control devices. Therefore, it is not surprising that the surveys found several devices with low comprehension levels. Some symbol signs were not understood very well, and about half of the drivers could not identify the difference between identical orange and yellow signs.

Analysis of Urban Arterial Lane Closure Capacity

An estimate of the capacity of a lane closure located within an urban arterial work zone was determined by measuring traffic flow at one site in Arlington, Texas. Two lanes were reduced to one. Volumes were measured for 25 periods, each of 5 min. A queue was present at all times. As indicated in Table 2, the 5-min flow rates ranged between 612 and 864 vehicles per hour (vph), and the 15-min flow rates for the same time period ranged between 652 and 808 vph. The average flow rate for both the 5- and 15-min periods was about 736 vph. These preliminary observations indicate that a realistic estimate of the capacity of a lane closure in an urban arterial work zone is in the range of 750 to 800 vph. This represents about 56 to 60 percent of the capacity of a freeway lane closure with similar geometrics as described in Chapter 6 of the *Highway Capacity Manual* (*10*).

Signal Operation near a Lane Closure

One of the major concerns associated with lane closures in urban arterial work zones is that the queue resulting from the lane closure may back up into an upstream intersection and prevent cross-street traffic from entering the intersection. This situation is particularly critical when the lane closure is located a short distance downstream of a signalized intersection. In situations where the lane closure queue has the potential to block a signalized intersection, it may be appropriate to locate the lane closure upstream of the intersection.

TABLE 2 Flow Rates for Arterial Work Zone Lane Closure

Time Start	5-Minute Volume	Equivalent Hourly Flow	15-Minute Volume	Equivalent Hourly Flow
7:43	57	684	---	---
7:48	54	648		
7:53	60	720	176	704
7:58	62	744		
8:08	52	624		
8:13	51	612	163	652
8:18	60	720		
11:09	51	612		
11:14	53	636	169	676
11:19	65	780		
11:24	68	816		
11:29	68	816	192	768
11:34	56	672		
11:39	63	756		
11:44	63	756	189	756
11:49	63	648		
11:57	68	816		
12:02	65	780	201	804
12:07	68	816		
12:12	61	732		
12:17	62	744	195	780
12:22	72	864		
12:27	68	816		
12:32	62	744	190	760
12:37	60	720		
Average	61	735	184	738
Minimum	51	612	163	652
Maximum	72	864	202	808

--- indicates no data

A simplified operational analysis of the relationship between a lane closure queue and an upstream signal indicated that the maximum queue typically forms as the result of saturated flow from the intersection during the initial portion of the green interval. In other words, a queue of vehicles forms at the intersection during the red portion of the signal cycle. When the signal changes to green, this queue moves downstream to the lane closure as a platoon and forms a queue at the lane closure. If the capacity of the lane closure is greater than the arrival rate of vehicles, the queue length will decrease following the arrival of the saturation platoon.

The purpose of the operational analysis was to determine the separation distance needed between the lane closure and the signal to prevent the lane closure queue from blocking the upstream traffic signal. The lane closure capacity described earlier was used as the basis of the analysis, and it was assumed that the arterial volume is less than the capacity of the lane closure and that cross-street turning volumes are low enough that the lane closure queues clear before the arterial platoon arrives at the lane closure. The Poisson distribution was used to determine the probability that the separation distance would be exceeded.

Figure 2 shows a simplified graphical method of determining the separation distance as a function of the arterial volume, length of arterial red, and the probability that the queue will not exceed the separation distance. A minimum separation of 50 ft is recommended. Figure 2 applies only to an arterial with two lanes in each direction. If the separation distance cannot be obtained, the beginning of the lane closure should be extended upstream of the signal.

URBAN ARTERIAL WORK ZONE GUIDELINES

The following guidelines were developed from the results of research activities and investigations into current arterial work zone practices. The guidelines have not undergone an extensive experimentation or evaluation period in the field.

Traffic Control Guidelines

Traffic control guidelines include those related directly to the movement of traffic through the work zone and the traffic control devices used to control the traffic. The traffic control guidelines address signalized intersections, intersections, lane closures, speed control, channelization, and pavement markings.

Signalized Intersections

The overall capacity of an arterial is typically limited to the capacity of the signals on that arterial. During construction,

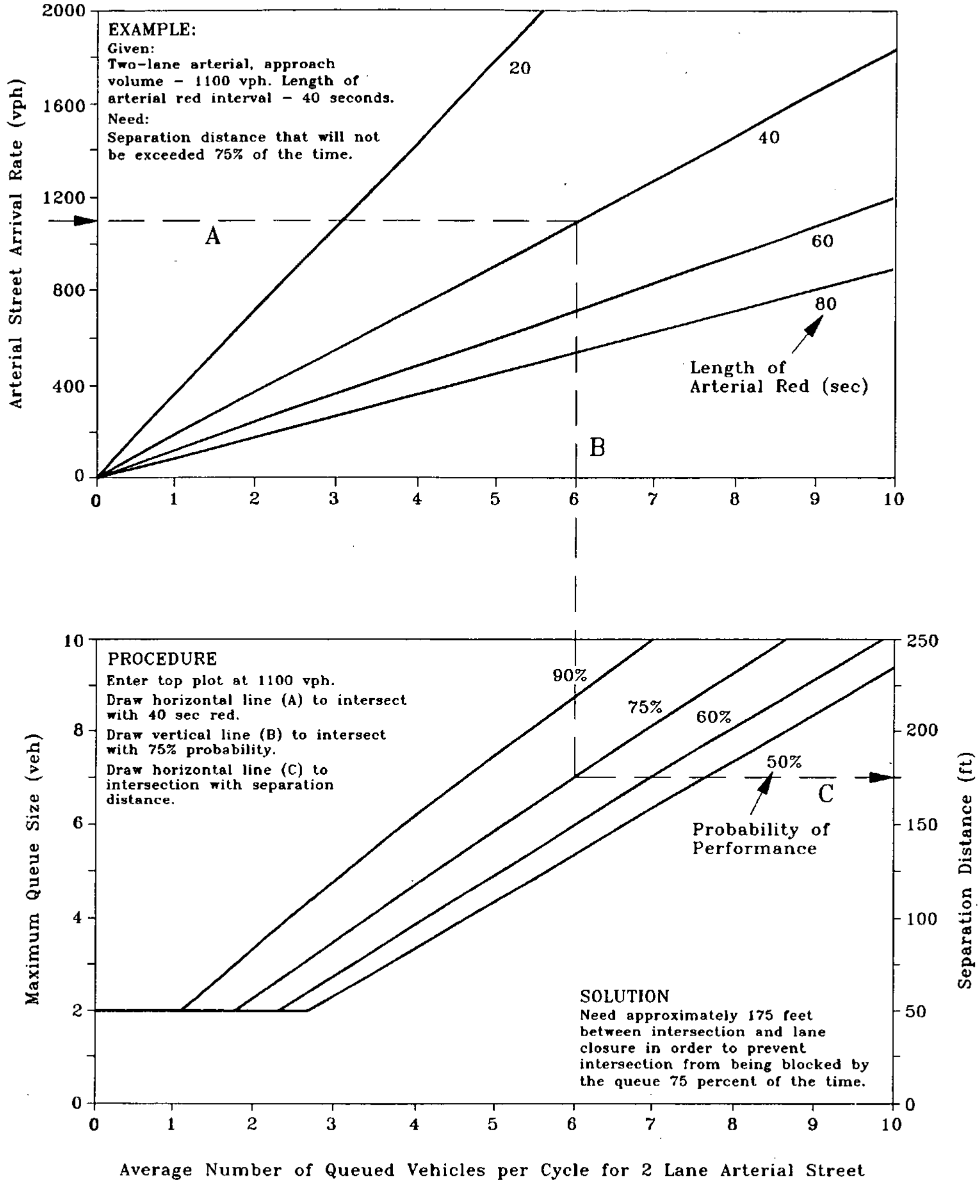

FIGURE 2 Lane closure-intersection separation.

the capacity of signalized intersections is often reduced. Therefore, it is important that steps be taken to ensure that the traffic signals within the work zone are operating in the most effective manner possible, given the restrictions of the work zone.

Signal phasing and timing should be adjusted with each change in construction phasing, and signal operation should be checked in the field after each adjustment. Construction activities cause a significant disruption of normal traffic patterns, and construction phasing may alter the lane arrangements at approaches to signalized intersections. All of these factors may negate preconstruction signal phasing and timing. Because changes in construction phasing may take place on a relatively frequent basis, changes in phasing or timing may be required more often than normal. As with normal signal operation, the effectiveness of new phasing or timings should be regularly checked in the field after implementation.

Short cycle lengths may be useful in reducing queue backup into the intersection. The effects of cycle length on queuing should be carefully observed at signalized intersections in the work zone. If queues due to construction activities or traffic generators are common, a shorter cycle length may be effective at minimizing queue lengths.

The positions of traffic signal heads should be shifted to line up with lane arrangements any time lane positions are modified. Signal heads should be located within the cone of visibility described in the MUTCD. The typical construction phasing plan for an urban arterial work zone uses narrow lanes and shifts the positions of the lanes within the intersection. If the signal head positions are not changed accordingly, the signals may not have enough target value for drivers to identify them in a complex urban work zone environment.

The operation of actuated signal detectors should be checked on a regular basis. If detection capability is lost, actuated controllers should be converted to pretimed operation. Any number of construction activities may affect or prevent the operation of traffic signal detectors. Without detection capability, an actuated signal becomes a pretimed signal by default, and the signal phasing and timings should be developed accordingly.

Time base coordination should be used to provide progression if the interconnection between signals is disrupted. Interconnection between signalized intersections may be lost in the same way that detection capabilities may be lost. If this occurs, progression cannot be provided for a series of signals. Maintaining progression is especially important if the traffic signals must operate in a pretimed mode. If progression is needed during construction to minimize motorists' delay, time-based coordinators can be used to provide progression without a physical connection between the controllers.

Pedestrian push buttons should be used with actuated controllers to maximize the efficiency of signals in a work zone. The congestion and delays associated with signals in a work zone are compounded by the need to accommodate pedestrians at signals. Although pedestrians are usually infrequent, sufficient crossing time must be provided for them. The most efficient method of accommodating pedestrians is to install pedestrian push buttons to reduce the amount of unused green. Even if vehicle detection capability is lost and the signals are operated in a pretimed manner, the pedestrian phase can still operate in an actuated mode.

New or temporary signals in arterial work zones should use 305-mm (12-in.) signal lenses. The large number of construction activities, traffic control devices, other vehicles, vehicle maneuvering, and development present in urban arterial work zones creates many demands for the driver's attention. Using 305-mm (12-in.) signal lenses will help the driver identify new or relocated traffic signals in the work zone.

Left-turn lanes should be provided at major signalized intersections. Left-turn movements can be a significant hindrance to traffic flow at signalized intersections. The lack of a left-turn bay can significantly increase delay because of left-turning vehicles blocking a through lane while waiting for an acceptable gap. Although the addition of left-turn lanes may create some difficulties for construction scheduling and activities, the benefits associated with these lanes make it desirable to provide them at major signalized intersections where left-turning vehicles are present. Figure 3 shows a potential layout for a left-turn lane that can be used when construction is taking place in the center of the road. The actual position of the lane can be shifted as needed to allow work to take place in the center area.

Intersections

The large number of intersections associated with urban arterial work zones introduce many difficulties related to work zone traffic control. Most of these difficulties are related to vehicle maneuvering and the intersection geometrics.

Large street name signs with block numbers should be provided at major signalized intersections, if possible. These street signs should be mounted overhead (on signal mast arms or span wire) to increase their visibility. When construction begins, many of the navigational aids, such as business signs and addresses, that drivers use are removed or become less visible. In addition, the preconstruction street signs may no longer be visible to drivers if the work space is located between the

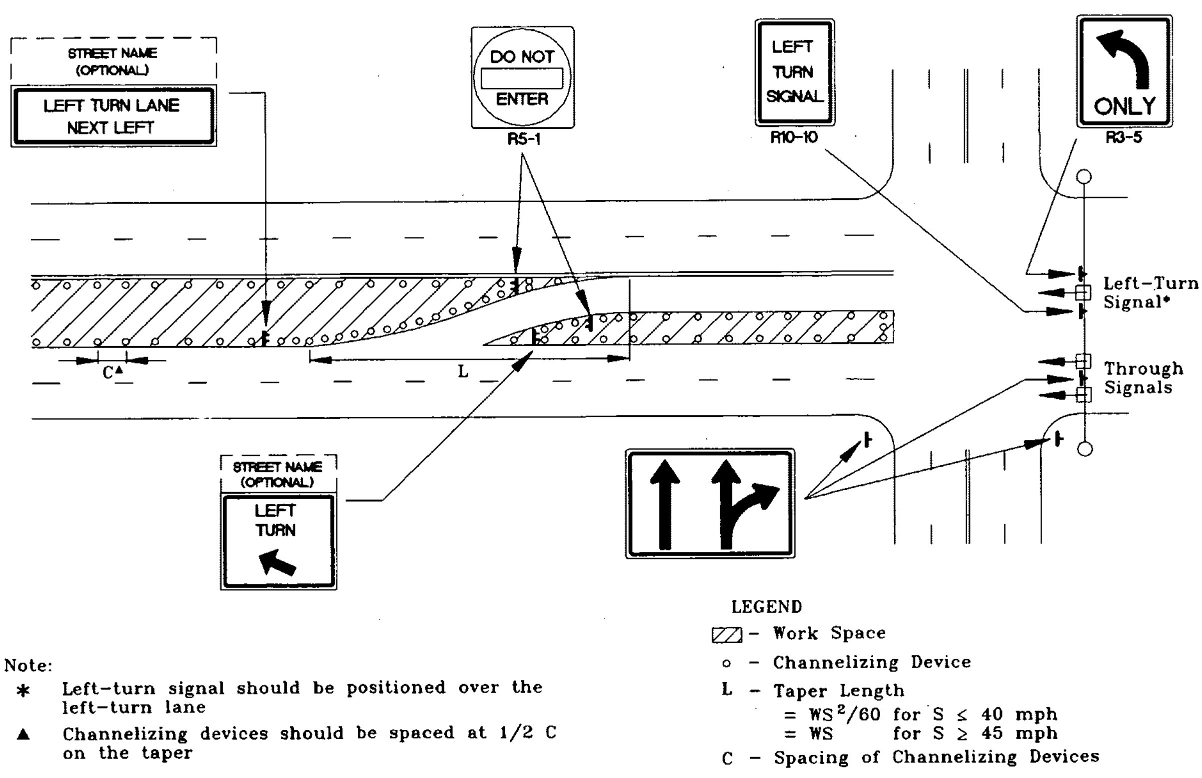

FIGURE 3 Layout of left-turn lane in work zone.

sign and traffic. Locating street signs overhead at signalized intersections will improve the visibility of street name signs.

As large a turning radius as possible should be maintained at driveways and intersections. The accident data from the study sites indicated an increase in the proportion of accidents occurring at intersections and driveways. One potential method of reducing accidents is to make it easier for vehicles to turn in and out of intersections and driveways by increasing the turn radius to reduce the potential for encroaching on adjacent lanes.

Driveways should be clearly marked and safe sight distances checked for each driveway. The presence of channelization devices may make it difficult for drivers on the roadway to identify the specific location of driveways and may create sight distance restrictions. Therefore, each driveway within the activity area should be checked to ensure that it is visible to drivers traveling down the roadway and that drivers in the driveway can adequately see traffic on the roadway.

Lane Closures

Although lane closures have a significant impact on traffic flow, they are a necessary part of any construction project. The detrimental effects of lane closures include the creation of queues that block intersections and driveways, the compounding of peak-period traffic congestion, and an increase in erratic lane changing.

An arrow panel should be used for lane closures on major arterial streets. Major arterials typically have high speeds and heavy volumes—conditions well suited to the use of an arrow panel for lane closures. On high-speed, high-volume arterials, an arrow panel should be used for lane closures in the same manner as for freeway lane closures. Arrow panels help motorists identify the location of the lane closure, and they may be more visible than some advance signing due to their greater mounting height.

Lane closures should be set up so that the queue will not block signalized intersections upstream of the lane closure. Queues often form upstream of a lane closure when volumes are high. If the lane closure is located too close to a signalized intersection, the queue may back up into the intersection and prevent cross-street traffic from entering the intersection. Sufficient distance should be provided between the lane closure and the intersection so that the queue will not block cross-street traffic. Figure 2 shows the minimum separation distance for various combinations of arterial volume, length of red, and probability of performance. If this distance cannot be obtained, the lane closure should be extended upstream of the intersection.

Lane closures should be located on a tangent section of roadway, if possible. Lane closures located on a curve present sight distance and maneuvering difficulties. A lane closure on a tangent section is more visible to approaching drivers, allowing them to change lanes further in advance of the merge point. Also, the lane change maneuver becomes less complicated because the driver is not negotiating a curve while changing lanes.

If possible, the lane closure should be located so that there are no intersections, driveways, or temporary median crossovers in the taper area or within 60 to 90 m (197 to 295 ft) of the beginning of the taper, as shown in Figure 4. Introducing turning and crossing maneuvers into the area where lane changing and merging are taking place brings turbulence into the traffic stream, creates more conflicts, and limits operational efficiency.

The lower capacity of an arterial lane closure should be considered when planning and implementing lane closures. Preliminary measurements of the capacity of a lane closure in an urban arterial work zone indicate that the capacity of two lanes being reduced to one is about 750 to 800 vph. This value is approximately 56 to 60 percent of the capacity of a freeway lane closure with similar geometrics (*10*).

Signing for a lane closure should be located upstream of a signalized intersection if the lane closure is less than 460 m (1,508 ft) downstream of a signalized intersection and arterial traffic volumes are high. Drivers may not be able to see a lane closure or signing for a lane closure when it is located close to a signalized intersection. However, the higher traffic density associated with saturation flow from a signalized in-

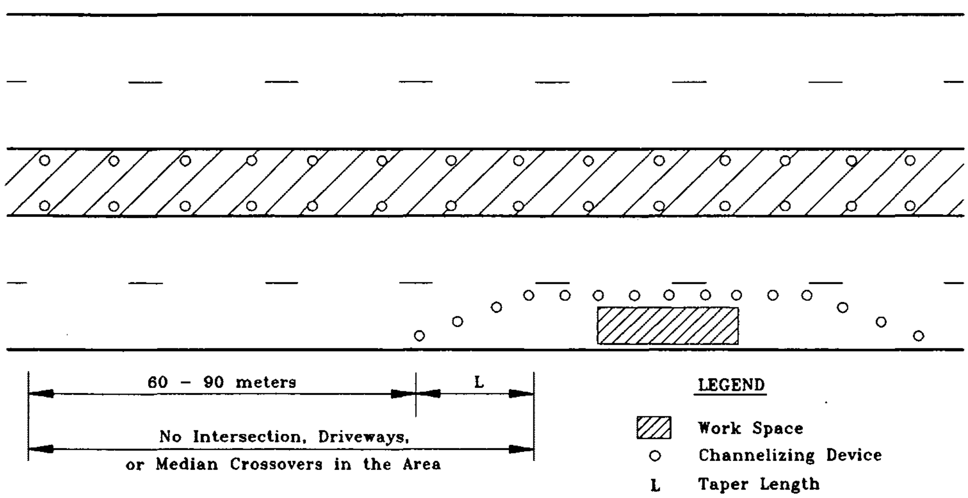

FIGURE 4 Lane closure taper location.

tersection eliminates many lane-changing opportunities. Placing the lane closure signing in advance of the signalized intersection gives drivers the opportunity to change lanes before reaching the queue at the intersection.

Speed Control

Speed reductions (both advisory and regulatory) are sometimes used in work zones for safety reasons. However, drivers do not always adhere to the speed reductions. Therefore, actions having an impact on vehicle speeds through the work zone should be evaluated carefully.

Speed restrictions should be avoided, if possible. If they are necessary, they should be carefully selected, recognizing that it may be necessary to supplement them with other more positive means of controlling driver behavior. Advisory speeds should be selected to be consistent with site conditions. Research has shown that drivers do not reduce their speed upon entering a work zone. Therefore, the normal arterial speed should be maintained in the urban arterial work zone, if at all possible. If speed restrictions are necessary, they should be carefully selected with the recognition that additional measures may be needed to slow traffic.

Speed information should be consistent. Advisory speed plates and speed limit signs with different speeds should not be placed within view of one another. The placement of speed limit and advisory speed information should be evaluated to ensure that conflicting speed information is not visible to the driver at one time. If a speed limit sign and advisory speed plate are visible to the driver at the same time, the driver will likely select the higher of the two speeds.

An enforcement area should be provided for police activities. The space restrictions associated with arterial construction may reduce the ability of police to enforce traffic laws. Police may not have an acceptable location to observe traffic and are hesitant to issue citations if a safe area to do so is not available. The lack of enforcement can breed disrespect for traffic laws. This may result in increased accidents and poor operations. However, even if an enforcement area is not provided and citations are not being issued, police presence in the work zone may help reduce vehicle speeds.

Channelization

Channelizing devices are often used in work zones for lane closures or for shifting the travel path of vehicles. The spacing of channelizing devices has some unique implications in urban arterial work zones.

Spacing between channelizing devices should be reduced in areas where vehicles may want to encroach on the construction area. The standard spacing for channelization devices on a tangent is a distance in meters (feet) equal to 0.379 (2.0) times the speed limit in kph (mph). At the speeds found on many arterials, vehicles can travel between the devices and drive on the wrong side. Drivers may cross the line of channelization devices to make an illegal turn, to pass an area of congestion, or because they are confused. Reducing the spacing of channelizing devices to a distance in meters (feet) equal to or less than 0.189 (1.0) times the speed limit in kph (mph) will discourage drivers from crossing into the work space.

Pavement Markings

The relocation of traffic lanes requires old markings to be removed and temporary markings to be placed. However, it is difficult to completely remove obsolete pavement markings.

Raised pavement markers, in conjunction with or in lieu of painted markings, should be used to enhance lane delineation in potentially hazardous areas. The removal and placement of pavement markings is one of the biggest challenges in work zones. Short of placing an overlay over old pavement markings, there is no method that will obliterate permanent pavement markings without leaving a scar. Raised pavement markers possess many advantages for use in urban arterial work zones. They can be easily placed and removed, and after removal, the remains of the markings do not provide as visible an indication of the lane lines as other types of markings. Raised pavement markers have greater visibility in periods of wet weather. They also provide a tactile indication to the driver when the vehicle begins to change lanes.

Construction Activity Guidelines

Issues associated with construction activities include difficulties related directly to performing construction activities: planning construction activities, scheduling the construction activities, and inspecting the traffic control.

Construction Planning

Several issues can be addressed in the initial stages of planning the construction activities that will help make the work zone safer and more efficient.

The construction phasing should be planned to minimize, as much as possible, the length of arterial under construction at any one time. The motorist surveys indicated that one of the most frequent complaints was the length of arterial under construction.

Unused construction equipment should not be left in public view for extended periods. Comments from traffic engineers indicated that they receive complaints about construction equipment being left along the arterial for extended periods. The complaints reflected a concern that construction progress was not occurring if equipment was not being used. Although the public does not understand the specifics of construction, it is important to avoid a lackadaisical appearance. Therefore, if construction equipment will not be used on a regular basis, it should be stored where it will not be seen.

High early strength concrete should be used to minimize the duration of construction as much as possible. The curing requirements of materials affect project scheduling and traffic flow. Numerous difficulties are related to the time spent waiting for concrete to cure before vehicles are allowed to travel on it. In addition, the public does not understand the need for the concrete to cure and perceives dry concrete that is not open to traffic as an inefficient construction practice. The use of high early strength concrete will allow newly paved areas to be opened to the public more quickly.

The outside travel lane should be wider in areas with large numbers of driveways and intersections. One of the unique characteristics of urban arterial work zones is the large number of vehicles turning onto and off the arterial. In some cases, these turns occur at locations with short turning radii on the curb return, causing some turning vehicles to encroach on the inside travel lane. Providing a wider outside lane will reduce the potential for encroachment on the inside lane.

Bus stops should be relocated to appropriate locations. Temporarily relocating bus stops to midblock or off-street parking areas may help to improve traffic flow through the arterial work zone because of the effects of construction on transit operations and pedestrian movements.

If construction is planned for a major arterial and the duration or impacts of the construction are expected to be significant, consideration should be given to improving alternate routes before construction begins on the arterial. At a minimum, consideration should be given to modifying the signal phasing and timings on the alternate route.

Project and Work Activity Scheduling

The impact of an urban arterial work zone on the nearby commercial, retail, and residential areas can be reduced through judicious scheduling of project and work activities.

To minimize traffic conflicts, lane and intersection closures during peak periods should be avoided. Traffic volumes on arterial streets are highest during the morning and evening peak periods. During these high-demand periods, all available capacity should be provided for traffic flow. Avoiding lane closures and intersection closures during these peak periods reduces congestion, delay, and vehicle conflicts. It may also be appropriate to avoid lane and intersection closures during the lunchtime peak period because of the high volumes that may be present at lunchtime in some areas.

If possible, projects should not be scheduled to begin construction between Thanksgiving and New Year's Day in heavy retail areas. The heaviest shopping period of the year is between Thanksgiving and the end of the year. Retail businesses generate more traffic than usual, and arterials adjacent to these businesses carry higher traffic volumes during this period. Therefore, it is desirable to avoid starting construction during the Christmas shopping season.

Inspection

Because of the large number of motorists who drive through an urban arterial work zone, qualified personnel should regularly examine the traffic control devices in the work zone.

Inspectors with specific training in work zone traffic control should inspect urban arterial work zones on a regular basis. The primary concern for many construction inspectors is the quality and progress of the construction activities. In some cases, the construction inspector may have little or no formal training in work zone traffic control. Therefore, it is important that an inspector whose primary responsibility is traffic control inspect the arterial work zone on a regular basis. This individual should have specific training in work zone traffic control and risk management.

Traffic control in the work zone should be checked during periods of darkness. Accident data from the study sites indicated that there was an increase in the number of accidents occurring during periods of darkness, despite the fact that construction activities were not taking place at night. Regular nighttime inspections by qualified traffic control inspectors can help identify locations where visibility of devices can be improved and where glare from other lighting sources interferes with visibility of the work zone. Such inspections can identify the needs of large nighttime traffic generators and provide indications of nighttime traffic characteristics.

Temporary Median Crossovers

The need for motorists to get from one side of the arterial to the other places many demands on a work zone. When the construction area is located between opposing traffic, the ability to provide temporary crossovers may be restricted by the proximity to traffic signals, the required geometrics of the crossover, and the relationship of a crossover to the arterial access locations.

In areas with heavy retail development and many access points on the arterial, it may be appropriate to locate one or more temporary median crossovers between each pair of traffic signals when the spacing between the signals exceeds 300 m (984 ft). However, temporary crossovers may not be necessary if through and left-turn movements at the intersection are light and the intersection can accommodate the increase in left-turn and U-turn volumes. Some areas create a heavy demand for left-turn movements. Typically, this type of area has a significant retail development and many access points on the arterial. When the work space is located between traffic flowing in opposite directions, left-turn movements are restricted to intersections and locations between intersections where temporary crossovers have been provided. If temporary crossovers are not provided, all left-turn demand is shifted to the intersections. If traffic volumes are heavy, the increased demand at the intersection may create operational problems and cause cycle failures. There should be enough distance between the signals so that the traffic turbulence created by the crossover does not affect operations at the signals. Temporary crossovers should be located a minimum of 90 to 125 m (295 to 410 ft) from any intersection. Signals spaced less than 300 m (984 ft) apart create some operational difficulties, which are compounded by the presence of a crossover between the signals.

The grade of a temporary crossover or temporary driveway should be as level as possible within 6 m (20 ft) of the higher elevation roadway to reduce sight distance restrictions. If a temporary crossover or temporary driveway is crossing an excavated area and has a pavement surface lower than the arterial, sight distance restrictions may be created by the channelizing devices along the activity area. By providing a nearly level approach to the arterial, these sight distance restrictions can be minimized. In some cases, the size of the activity area or the difference in elevation between the arterial pavement surfaces may make it difficult to provide a level crossover or driveway. If this is so, the sight distance should be checked. If sight distance is not adequate, the crossover or driveway should be eliminated.

U-turns should be permitted at traffic signals if a temporary crossover is not provided between the signal and the previous signal. If a temporary crossover is not provided between signals, vehicles will make left- and U-turns at the intersection to gain access to properties on the other side of the work space. Signal operation and intersection geometrics should be checked to ensure that U-turns are possible. If U-turns cannot be safely accommodated, alternative means of providing access to properties should be evaluated.

SUMMARY

This paper describes a research study intended to identify the unique characteristics of urban arterial work zones and develop traffic control guidelines for them. The completed research confirms the lack of existing guidance and has identified numerous guidelines that should help to improve both traffic flow and worker safety in urban arterial work zones.

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Temporal Speed Reduction Effects of Drone Radar in Work Zones

RAHIM F. BENEKOHAL, PAULO T. V. RESENDE, AND WEIXIONG ZHAO

Three experiments were conducted to evaluate the effectiveness of using drone (passive or unmanned) radar guns on vehicle speeds in work zones. Experiment 1 was an exploratory study to determine the immediate effects of using one drone radar gun on speed. Experiment 2 was conducted to evaluate the short-term effects of using one drone radar gun on speed. Experiment 3 measured the short-term effects of using two drone radar guns on speed. It was divided into three 1-hr time intervals to determine the lasting effects of using two radar guns on speed. The immediate effect of using one radar gun (Experiment 1) was a speed reduction of 13 to 16 km/hr (8 to 10 mph); however, such reduction should not be taken as a typical value. Experiment 2 showed that using one radar gun was not effective in reducing speed when drivers knew that it was drone radar. Experiment 3 indicated that the use of two radar guns increased the radar effectiveness, since drivers were not sure whether the signals would come from a police radar or drone radar. The effectiveness was consistent on trucks, but not on cars. The two-radar experiment reduced speeds of trucks by 5 to 10 km/hr (3 to 6 mph) in most cases, but speeds of cars were reduced by 5 km/hr (3 mph) only in two out of six cases. The speed reduction effects of the two-radar experiment on trucks were sustained over a time period of 3 hr.

Having a police officer in every work zone is a costly speed enforcement option. However, providing an indication of "threat" of police presence, such as using drone radar, is relatively inexpensive and may work to alleviate some of the speeding problems in work zones. This study was conducted to determine the short-term effects of using drone radar, also called passive or unmanned radar, and the lasting effects of continuous radar signal transmission on the speed of vehicles in a rural Interstate highway work zone in Illinois. At the time of this study, cars and trucks were still allowed to use radar detectors in Illinois.

The study consisted of three experiments. Experiment 1 was an exploratory study to evaluate the immediate (less than 1 hr) effects of transmitting radar signals on the speed of vehicles when motorists were traveling at excessive speeds inside and outside of the work zone. Experiment 2 was conducted to evaluate short-term (a few hours) effects of using one drone radar gun on speeds of vehicles. Experiment 3 was an attempt to determine the short-term effectiveness of using two drone radar guns as well as the lasting effects of radar signal transmission on vehicular speeds. In Experiment 3, two radar guns were used to increase the perceived "threat" of police and to make it difficult for the drivers to figure out the source of transmission. The assumption was that if drivers could not find out whether it was police or drone radar, they might consider the radar as a "threat" and keep lower speeds.

Traffic data were collected when one or two drone radar guns were added to standard Illinois Department of Transportation traffic control plans. These plans were prepared according to the procedures discussed in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*). Figure 1 shows the work zone signs used during the drone radar study. Illinois uses two arrow boards in such work zones.

The study sites were located on a rural section of I-57 in central Illinois. The highway has two lanes in each direction, with one lane per direction closed during the construction period. Average daily traffic was approximately 12,000 with nearly 22 percent heavy commercial vehicles. The speed limit outside the construction zone was 105 km/hr (65 mph) for cars and 89 km/hr (55 mph) for heavy trucks (over 4 tons); inside the zone it was 72 km/hr (45 mph) for all vehicles. The regulatory 72 km/hr (45 mph) work zone speed limit was in effect when two small yellow lights, mounted on top of the speed limit sign, were flashing.

BACKGROUND

Radar guns have been used by law enforcement officers to measure speeds of vehicles. Warren (*2*) synthesized the effects of law enforcement on regular highway sections (not in work zones) and reported that in most cases police enforcement decreased speed by less than 5 km/hr (3 mph), but reductions of up to 16 km/hr (10 mph) were also noted. Pigman et al. (*3*) used drone radar at two high-accident locations (not in work zones) on I-75 and reported that it was effective in reducing speeds of vehicles traveling at excessive speeds. They showed that speeds of vehicles with radar detectors decreased significantly compared with speeds of vehicles lacking radar detectors. Pigman et al. (*3*) also reported that 42 percent of trucks and 11 percent of cars had radar detectors.

There has been a very limited number of studies dealing directly with the effects of drone radar on vehicle speeds in work zones. Richards et al. (*4*) reported that, in a construction zone on an urban freeway with 64-km/hr (40-mph) regulatory speed limit, a stationary patrol car with radar on caused 5 km/hr (3 mph) more speed reduction than a stationary patrol car with radar off. Ullman (*5*) reported that radar transmission, without police presence in work zones, reduced the average speed by less than 2.7 km/hr (1.7 mph) in seven out of eight study sites. On the eighth site, a reduction of 7.2 km/hr (4.5 mph) was obtained, but this reduction was computed on the basis of a small sample of observations (less than 30 vehicles) and therefore may not be very reliable.

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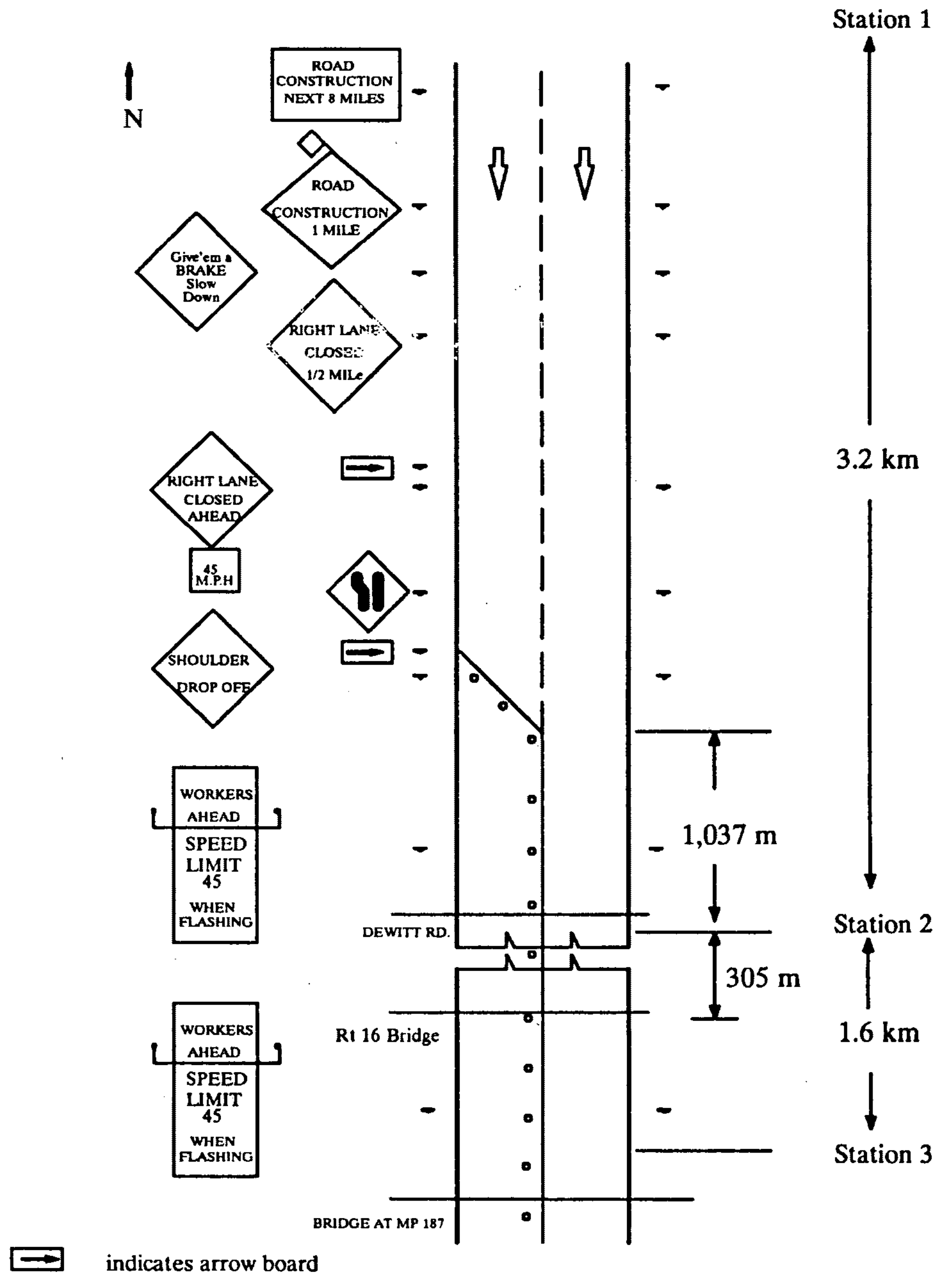


FIGURE 1 Work zone signs on southbound I-57 during drone radar study.

STUDY APPROACH

The approach used in this study is commonly known as before and after study with control group. Data collection and data analysis are performed according to this method.

Data Collection for Experiment 1

The study site was located in the northbound approach of a rural section of I-57 south of Champaign, Illinois (Site 1). Data were collected on September 22, 1989, for two time periods at two stations. During the first period (control) no drone radar was used. During the second period (one-radar treatment) one radar gun was used at Station 2. Control data were collected from 1:00 to 2:00 p.m. and treatment data from 2:15 to 2:50 p.m. Station 2 was located 260 m after the end of the lane closure taper where only one lane was open to traffic. Station 1 was located outside the work zone about 2.4 km before Station 2.

Data Collection for Experiments 2 and 3

Experiments 2 and 3 were carried out in a work zone on the southbound approach of a rural section of Interstate 57 near Mattoon, Illinois (Site 2). Data were collected at three locations. Station 1 was outside the work zone, and two others inside it (Figure 1). At Site 2, data were gathered for the following three conditions:

1. Control or base condition—no radar was used;
2. One-radar treatment (Experiment 2)—one radar gun was activated near Station 2; and
3. Two-radar treatment (Experiment 3)—two radar guns were activated simultaneously, one close to Station 2 and the other near Station 3.

Control data were collected from 10:00 a.m. to noon, June 12, 1990. For the one-radar treatment, they were gathered from 1:30 to 3:10 p.m., June 12, 1990. Data for the two-radar treatment were collected from 1:40 to 4:25 p.m., June 11, 1990. The two-radar treatment was divided into three 55-min periods to examine the lasting effects of drone radar. These time periods are denoted as Intervals I, II, and III, designating the first, second, and third periods, respectively.

Data Reduction

Vehicle speeds at each station were collected with mechanical traffic counters programmed to keep a record of individual vehicles. A Fortran program was written to perform sorting, classification, and error checking (*6*).

Data Analysis Approach

The minimum, mean, and maximum speeds, as well as standard deviation, frequency distribution, and percentage of vehicles exceeding a given speed level were determined. *F*-tests and *t*-tests were performed to compare speed variances and mean speeds, respectively. A 95 percent confidence level was used unless stated otherwise. Results of the *F*-test determined the type of *t*-test to be used for comparing the average speeds of the two data sets (*7*). Since free-flow speeds are used to compute speed variances, a change in variances should not be correlated with traffic safety in work zones.

An assumption in using the *t*-test is that the speed data ought to have a normal distribution. The data used in this study came from free-flow vehicles and, therefore, did not necessarily have a normal distribution. However, the *t*-test was still viable because of its relative insensitivity to normal distributions (*8*). All statistical analyses were performed using PC-SAS (*7*). A separate statistical analysis was performed for cars and trucks because of the differences in posted speed limits as well as in speed distributions.

Net Speed Reduction Analysis

Net speed reductions were computed to determine whether there were additional speed reductions due to the use of drone radar in the work zone. The net speed changes were computed from the following:

$$\text{Net speed change at Station } n = (\overline{U}_{nt} - \overline{U}_{nc}) - (\overline{U}_{1t} - \overline{U}_{1c})$$

where

$\overline{U}_{1t}$ = the treatment mean speed at Station 1;
$\overline{U}_{nt}$ = the treatment mean speed at Station n, n = 2 or 3;
$\overline{U}_{1c}$ = the mean speed for control data at Station 1; and
$\overline{U}_{nc}$ = the mean speed for control data at Station n, n = 2 or 3.

A *t*-test with 95 percent confidence level was used to determine whether the net reduction was statistically significant.

ONE-RADAR EXPERIMENT AT SITE 1 (EXPERIMENT 1)

Description

At Site 1, one radar was activated near Station 2 for a short period. A citizens' band (CB) radio was used to monitor the conversation among drivers. During the data collection, the flashing lights on the speed limit signs were turned on to indicate that a regulatory 72-km/hr (45-mph) speed limit was in effect. The speed limit signs were located at the end of the lane closure taper. A small construction crew (four to five people) with light equipment and a pickup truck was working north of our Station 2. The crew moved from one location to another as workers finished minor pavement repair jobs. The crew was far enough from Station 2 and its presence did not cause a noticeable speed reduction at Station 2. There were no police in the work zone.

Summary of Findings

The speed characteristics for the control and treatment data are given in Table 1. During the control period, cars and trucks at Station 1 were traveling at about 16 km/hr (10 mph) over their respective speed limits. At Station 2, the average speeds of cars and trucks were nearly 30.2 and 21.7 km/hr (18.8 and 13.5 mph), respectively, over the speed limit. During treatment, cars and trucks showed average speeds of approximately 118.8 and 103.8 km/hr (73.8 and 64.5 mph) at Station 1. At Station 2, average speeds were nearly 87.4 and 77.7 km/hr (54.3 and 48.3 mph). The drone radar experiment resulted in net speed reductions of more than 12.88 km/hr (8.00 mph) on cars and trucks. These reductions were found to be statistically significant.

The percentages of vehicles exceeding the speed limits at Station 1 were practically the same for treatment and control data. However, at Station 2, during the treatment period, there was a considerable decrease in these percentages for both cars and trucks.

Although this drone radar experiment resulted in net speed reductions of such magnitudes, these results are not typical and they have to be interpreted in the light of the following factors:

1. The net speed reductions were high because vehicles were traveling faster outside and inside the work zone. As a result, the speeding drivers may have been more concerned

TABLE 1 Speed Statistics at Site 1 (km/hr)

	CARS				TRUCKS			
STATION	1		2		1		2	
CONDITION*	Cont	Treat	Cont	Treat	Cont	Treat	Cont	Treat
MEAN SPEED	120.9	118.8	102.7	87.4	104.4	103.8	94.1	77.7
MIN. SPEED	88.5	88.5	75.6	67.6	85.3	94.9	77.2	57.9
MAX. SPEED	146.5	144.9	140.0	138.4	122.3	114.3	112.7	94.9
STANDARD DEVIATION	17.08	15.99	19.98	19.90	12.49	9.82	13.89	16.30
NO OF OBS.	173	116	178	96	36	21	26	13
% EXCEEDING SPEED LIMIT	92.5	91.4	100	90.6	97.2	100	100	76.9

* Cont = Control, Treat = Treatment

about the police threat than if they were traveling at the speed limit.

2. The net reductions may not reflect the long-term effects of drone radar since the results were based on data collected in a very short time period.

3. The location of Station 2 was 259.2 m (850 ft) from the end of the lane closure taper. This point might not reflect the speed further inside the work zone.

4. The net reduction for trucks was computed on the basis of a small sample of truck drivers who traveled in the work zone during the short time that the radar was activated. The long-term reductions were less likely to be so high.

From Experiment 1 at Site 1 it was concluded that, in a very short period of time, the drone radar was effective in reducing speeds at the beginning of a one-lane section when the average speed of traffic was high outside of the work zone. This radar experiment simulated a short-term maintenance work where the crew spent less than 1 hr in one location. Because of the limitations of Experiment 1, two other experiments were conducted.

ANALYSIS OF ONE-RADAR EXPERIMENT AT SITE 2 (EXPERIMENT 2)

Description

The first experiment at Site 2 was conducted when one radar gun was activated near Station 2. Speeds of vehicles were measured at three stations, and two CB radios were used to monitor drivers' conversations. Stations 2 and 3 were also monitored to record any unusual behavior that might disturb the normal flow of traffic close to the stations. The flashing lights on the speed limit signs were on during data collection and no police were present in the work zone. The construction crew was working on the bridge over Route 16.

Speed Characteristics

The average speeds were lower at Station 2 than at Station 1 but higher at Station 3 than at Station 2 (Table 2). This speed trend was observed for both cars and trucks. The control data showed that car drivers traveled as high as 135.2 km/hr (84.0 mph) outside and 123.9 km/hr (77.0 mph) inside the work zone. Their average speeds exceeded the speed limit by approximately 5.2, 14.1, and 25.7 km/hr (3.2, 8.7, and 15.9 mph) at Stations 1, 2, and 3, respectively. The percentages of cars exceeding the speed limits were about 73, 96, and 99 percent at Stations 1, 2, and 3, respectively (Figure 2). The percentages exceeding a given speed were higher at Station 3 than at Station 2 and close to the percentages for Station 1, although Station 3 was still inside the work zone.

Truck drivers traveled as high as 132.0 km/hr (82.0 mph) outside and 115.9 km/hr (72.0 mph) inside the work zone during the control period. The average speeds were nearly 11.7, 7.2, and 20.5 km/hr (7.3, 4.5, and 12.7 mph) higher than speed limits at Stations 1, 2, and 3, respectively. The percentages exceeding speed limits at Stations 1, 2, and 3 were about 89, 75, and 97 percent, respectively (Figure 2). Like cars, trucks traveled at higher speeds at Station 3 than at Station 2. The percentages of trucks exceeding a given speed at Station 3 were comparable with those for Station 1 (Figure 3).

Data for the one-radar experiment indicated that the average speeds of cars were nearly 3.3, 14.1, and 24.9 km/hr (2.0, 8.7, and 15.5 mph) over the speed limits. Speeding cars made up approximately 66, 92, and 99 percent of free-flow car traffic at Stations 1, 2, and 3, respectively (Figure 3). In the one-radar experiment, the trends for percentages of cars exceeding given speed levels were similar to those of the control data (Figure 2).

During the one-radar treatment, trucks were traveling 10.6, 5.2, and 20.2 km/hr (6.6, 3.2, and 12.5 mph) faster than the speed limits. About 88, 70, and 98 percent were speeding at Station 1, 2, and 3, respectively (Figure 2). The distribution of trucks with excessive speeds showed that speeds at Station 3 were higher than at Station 2 and closer to speeds at Station 1.

Net Speed Reductions

The net speed reduction for cars at Station 2 was −1.77 km/hr (−1.09 mph)—a speed increase—which, according to the *t*-test, was not significant. This means that activating one radar

TABLE 2 Speed Statistics at Site 2 (km/hr)

VEHICLE TYPE	CARS			TRUCKS		
STATION	1	2	3	1	2	3
CONTROL DATA						
MEAN SPEED	109.9	86.1	97.7	100.7	79.2	92.2
MIN. SPEED	77.2	67.6	69.2	72.4	64.4	56.3
MAX. SPEED	135.2	115.9	123.9	132.0	94.9	115.9
STANDARD DEVIATION	12.62	14.83	16.46	14.54	12.18	15.63
NO OF OBS.	274	193	187	112	114	97
% EXCEEDING SPEED LIMIT	73.4	95.9	98.9	89.3	75.4	96.9
ONE-RADAR TREATMENT						
MEAN SPEED	108.0	86.1	96.9	99.6	77.2	92.2
MIN. SPEED	78.8	56.3	67.6	74.0	62.7	66.0
MAX. SPEED	144.9	111.0	119.1	122.3	94.9	111.0
STANDARD DEVIATION	15.63	15.11	15.42	13.69	11.33	13.69
NO OF OBS.	149	170	87	88	98	55
% EXCEEDING SPEED LIMIT	65.8	92.4	98.9	87.5	70.4	98.2

did not significantly affect the average speed of cars at Station 2. The net speed reduction for cars at Station 3 was -0.80 km/hr (-0.49 mph), which was also considered not statistically significant. As a result, using one radar was not effective in reducing speeds of cars at this location.

The net speed reductions for trucks were 0.80 and -1.12 km/hr (0.49 and -0.69 mph) at Stations 2 and 3, respectively, with no statistical significance. Thus, the radar was not effective in lowering speeds of trucks at this location. The lack of effectiveness may be explained by the fact that, in less than 0.5 hr, truck drivers with CBs figured out the presence of a drone radar and, consequently, the absence of active speed limit enforcement in the work zone.

Results indicated that using one radar gun at Site 2 did not produce additional reductions on the average speeds of cars and trucks. The effectiveness of one drone radar at Site 2 was not as significant as that at Site 1. The main reasons for these findings may be as follows:

1. Cars and trucks were traveling at lower speeds at Site 2 than at Site 1. For control data at Station 1, cars and trucks traveled 11.0 and 3.7 km/hr (6.8 and 2.3 mph), respectively, faster at Site 1 than at Site 2. At Station 2, cars and trucks traveled 17.0 and 15.0 km/hr (10.3 and 9.3 mph), respectively, faster at Site 1 than at Site 2. Besides, at Site 1, during the control period the average speeds of cars and trucks were 30.2 and 21.7 km/hr (18.8 and 13.5 mph) greater, respectively, than the speed limit at Station 2. However, at Site 2, the average speeds of cars and trucks were only 13.7 and 6.8 km/hr (8.5 and 4.2 mph) greater, respectively, than the speed limit at Station 2. Thus, drivers at Site 2 may not have felt the need for slowing down as strongly as at Site 1.

2. The time period for Site 2 was about three times longer than that for Site 1, and drivers had enough time to figure out who was activating the radar and where it was being activated. In fact, within 0.5 hr some drivers with CB radios were advised of the absence of active speed limit enforcement, and they may not have felt threatened by the radar transmission.

ANALYSIS OF TWO-RADAR EXPERIMENT AT SITE 2 (EXPERIMENT 3)

Description

In Experiment 3, two radar guns were simultaneously activated in the work zone. One radar gun was located near Station 2 and another was close to Station 3. Two radar guns were used to increase the perceived "threat" of police presence and make it difficult for drivers to determine who was transmitting the radar signals. The assumption was that if drivers could not realize whether it was police or drone radar, they might consider the radar as a threat and keep lower speeds.

As in Experiment 2, speeds were measured at three stations, and two CB radios were used to monitor drivers' conversations. Stations 2 and 3 were also monitored to record any unusual behavior that might disturb the normal traffic flow near the stations. The speed limit sign flashing lights were on during data collection, and police were not present in the work zone.

The construction crew was working on the bridge over Route 16 until 3:30 p.m. After 3:30 p.m. (beginning of Interval III), the construction crew left the bridge, resulting in a few work-

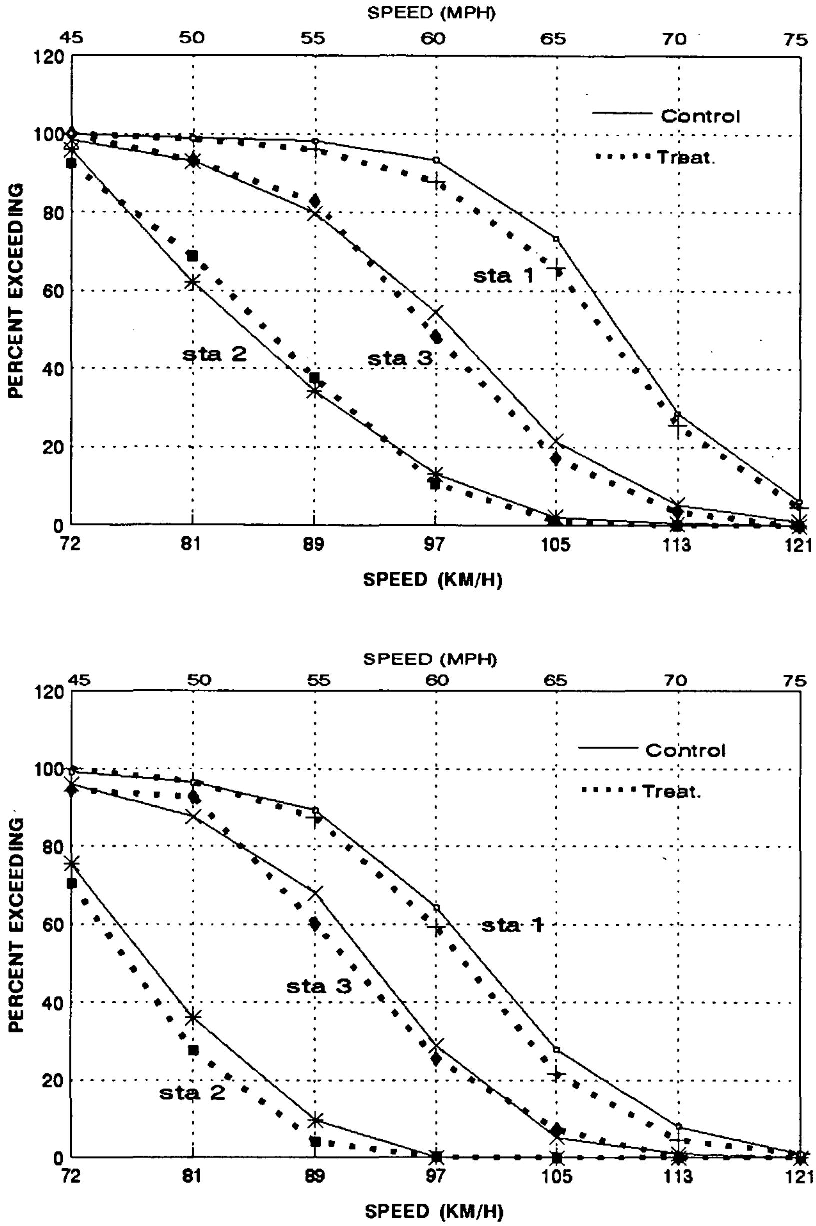


FIGURE 2 Percentage of cars (*top*) and trucks (*bottom*) exceeding a given speed at Site 2, control and treatment data, one-radar treatment.

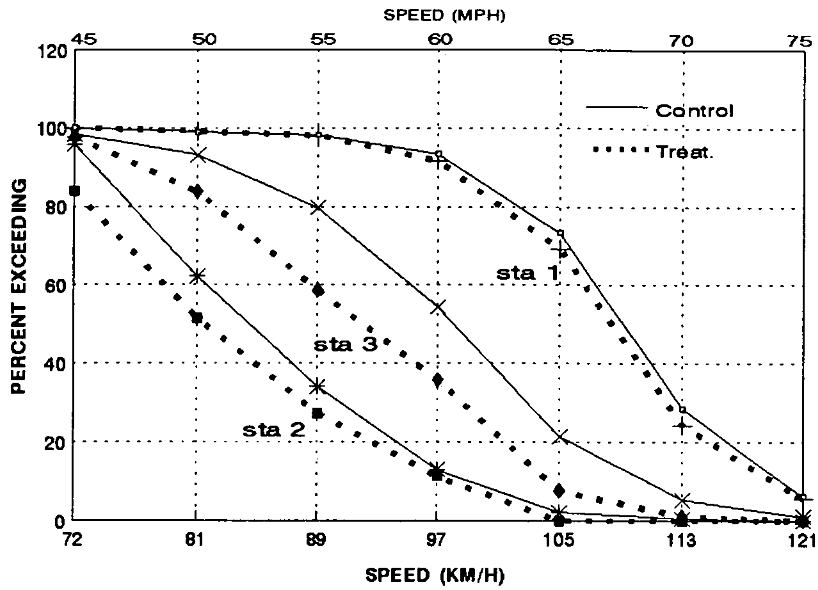
ers sporadically operating at the site. The flashing lights on the speed limit signs were turned off at the end of Interval III.

The radar near Station 2 was inside a car parked on the northbound direction and aimed at the drivers in the southbound direction. The second radar was placed in a tree located near Station 3 and aimed at the southbound traffic. The tree was selected close to an overpass to give the impression that police might be at the overpass.

Highlights from CB Monitoring

The study team used K- and X-band radar guns and activated them at 1:40 p.m., with the K-band radar placed in the parked car and the X band radar in the tree. From the beginning of this experiment an extensive conversation was going on among drivers trying to determine whether it was a false radar as well as its location. The study team was able to hear only part of their conversations, when drivers were close to Station 2 or 3. An example of the actual conversation, during a 45-min period, heard on the CB near Station 2 demonstrates the extent of the communication and the awareness of radar transmissions:

- 2:17 p.m.: "Smokey Bear doing a loop over here at 189, sitting there in northbound."
- 2:19 p.m.: "Southbound fuzz up here, county mounty."
- 2:30 p.m.: "SB plain gray wrapper."



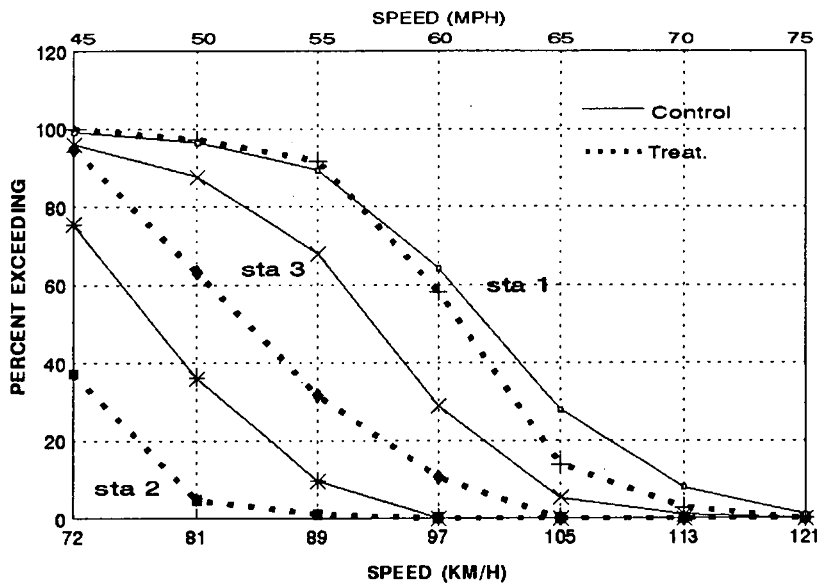


FIGURE 3 Percentage of cars (*top*) and trucks (*bottom*) exceeding a given speed at Site 2, control and treatment data, two-radar treatment, Interval I.

- 2:32 p.m.: "On your side 184 plain brown wrapper they got one of those false radars."
- 2:33 p.m.: "At 190 there is a cop down there but got off."
  "Ah . . . there is a bird dog down here."
  "They have got to have one of them damn radar set up here."
  "Must have been motor home."
  "Negative, wasn't motor home."
  "Something was back there."
- 2:37 p.m.: "Nada 2:44 . . ."
- 2:50 p.m.: "Probably one of those vehicles with damn radar in it."
  "Yah, you probably right, just trying to get a handle on it."
  "It is different radar."
  "Well I think I'll pay attention now."
  "Sure'n I don't pay attention it's a cop."
- 2:54 p.m.: "There may be a couple of them, but I am not sure what the guy is taking about."
- 2:56 p.m.: "Hey my hot dog's crying to tell me something in it."
  "Well one of those dumb guns in the construction truck."
  "A what?"
  "One of those hand-held radar units."
  "Okay 10-4."
- 3:00 p.m.: "My radar detector was going crazy but I slowed down, there is no one out here I missed."
- 3:12 p.m.: "Those two . . . on the side of the road with their walkie-talkies in that construction zone,

just expect every body to slow down cause they got a policeman."

- 3:25 p.m.: "Bird dog barking."
  "Yeah they've got one in their construction truck."
  "10-4."
  "Well I don't know where they got damn things set up but my lights are going red for sure."
  "Yeah but quits right up at that bridge."
  "Ya got a speed picture taken down the road down here."

A similar conversation went on indicating that drivers were still trying to determine the location of the radar and whether the threat was real. From the CB monitoring, it became clear that drivers could not conclude whether it was false radar, the location, or how many had been used.

Data Analysis

Data for the two-radar treatment were collected for 2 hr 45 min and divided into three 55-min periods. These time periods are referred to as Intervals I, II, and III, designating the approximate first, second, and third hours, respectively. This method was used to examine the immediate lasting effects of drone radar. The control data were the same as those of the one-radar treatment at Site 2.

Summary of Speed Characteristics

For all three time intervals the average speeds of both cars and trucks were lower at Station 2 than at Station 1 but higher at Station 3 than at Station 2 (Table 3). The average speeds of cars and trucks were approximately 4.3 to 6.7 km/hr (2.7 to 4.2 mph) and 10.0 to 13.5 km/hr (6.2 to 8.4 mph), respectively, higher than their speed limits at Station 1. At Station 2, the average speeds were about 11.2 to 18.3 km/hr (6.9 to 11.4 mph) and 0.0 to 5.4 km/hr (0.0 to 3.1 mph), respectively, faster than 72 km/hr (45 mph). At Station 3, cars and trucks had average speeds nearly 20.2 to 25.4 km/hr (12.5 to 15.8 mph) and 13.8 to 14.3 km/hr (8.6 to 8.9 mph), respectively, higher than the speed limit of 72 km/hr (45 mph).

Cars and trucks exceeded the speed limits inside and outside the work zone (Figure 3). Between 69 and 81 percent of cars and 90 and 95 percent of trucks traveled faster than their respective speed limits at Station 1. At Station 2, 84 to 94

TABLE 3 Speed Statistics for Two-Radar Treatment at Site 2 (km/hr)

VEHICLE TYPE	CARS			TRUCKS		
INTERVAL	I	II	III	I	II	III
STATION 1						
MEAN SPEED	109.0	110.1	111.4	99.0	102.5	101.1
MIN. SPEED	80.5	80.5	88.5	80.5	83.7	85.3
MAX. SPEED	140.0	135.2	133.6	119.1	120.7	127.1
STANDARD DEVIATION	13.69	14.15	12.49	12.62	11.33	13.63
NO OF OBS.	107	154	122	36	41	50
% EXCEEDING SPEED LIMIT	69.2	77.9	81.1	91.7	95.1	90.0
STATION 2						
MEAN SPEED	83.2	86.3	90.3	72.2	76.1	77.4
MIN. SPEED	66.0	54.7	69.2	56.3	56.3	64.4
MAX. SPEED	104.6	114.3	112.7	91.7	98.2	94.9
STANDARD DEVIATION	15.47	19.70	15.73	10.68	15.99	10.65
NO OF OBS.	124	115	104	43	44	57
% EXCEEDING SPEED LIMIT	83.9	88.7	94.2	37.2	56.8	75.4
STATION 3						
MEAN SPEED	92.2	93.5	97.4	85.8	84.6	86.3
MIN. SPEED	70.8	61.1	70.8	70.8	74.0	70.8
MAX. SPEED	115.9	117.5	119.1	101.4	104.6	101.4
STANDARD DEVIATION	15.94	18.66	16.41	12.70	12.99	11.56
NO OF OBS.	92	94	91	38	36	46
% EXCEEDING SPEED LIMIT	97.8	96.8	98.9	94.7	100.0	97.8

percent of car drivers and 37 to 75 percent of truck drivers were speeding. At Station 3, 97 to 99 percent of cars and 95 to 100 percent of trucks traveled faster than the speed limit of 72 km/hr (45 mph). Cars and trucks increased their speeds after passing the work space as indicated by their higher average speeds and the percentages exceeding the speed limit.

In all three intervals, the percentages of vehicles with excessive speeds were higher at Stations 1 and 3 and lower at Station 2 (Figure 3). The differences between the percentages at Stations 2 and 3 were higher during Interval I than during intervals II and III. Trucks showed similar trends (Figure 3), with percentage differences for Stations 2 and 3 decreasing throughout the three intervals. In Interval I, the percentages were higher at Station 3 than at Station 2. During the two other intervals, percentages at Stations 2 and 3 were closer to each other.

Net Speed Reductions

Net Speed Reduction for Cars

For cars at Station 2, there was a 1.93-km/hr (1.22-mph) net reduction during the first hour (Interval I), no additional speed reduction in the second hour (Interval II), and a 2.73-km/hr (1.69-mph) reduction during the third hour (Interval III). The reductions for Intervals I and II were not statistically significant, but the net increase in the third hour was significant. This increase cannot be attributed to activating radar, because any effect would be pointed out by a decrease (positive net speed) and not an increase (negative net speed).

The main reason for such speed increase might be the absence of crew over the Route 16 bridge during Interval III. The workers left the work site over bridge at the beginning of Interval III, but the speed limit remained 72 km/hr (45 mph) until the end of the interval. The Route 16 bridge was nearly 305 m (1,000 ft) from Station 2, and drivers may have increased their speeds after noticing that there were no workers on the bridge.

For cars at Station 3, the two-radar treatment caused net speed reductions of 4.66, 4.34, and 1.93 km/hr (2.89, 2.69, and 1.19 mph) for Intervals I, II, and III, respectively. Reductions for Intervals I and II were statistically significant, but that for Interval III was not. Drivers may have traveled at higher speeds at Station 3 because they did not see the crew working on the Route 16 bridge. Thus, the drone radar was less effective in Interval III than at Intervals I and II.

Net Speed Reduction for Trucks

For trucks at Station 2, there was a net speed reduction of 5.15, 4.99, and 1.93 km/hr (3.19, 3.09, and 1.19 mph) at Intervals I, II, and III, respectively. Reductions at Intervals I and II were statistically significant, but that at Interval III was not. The results indicated that trucks reduced their speeds at Station 2 when the radar was activated. Drone radar was less effective in Interval III, perhaps because truck drivers did not see any workers on the bridge.

For trucks at Station 3, net speed reductions of 4.66, 9.33, and 6.27 km/hr (2.89, 5.79, and 3.89 mph) were achieved during Intervals I, II, and III, respectively. These reductions were statistically significant, indicating that activating two radar guns caused extra reductions of approximately 5 to 10 km/hr (3 to 6 mph) for trucks. Station 3 was located after the work space where drivers tended to increase their speeds. Besides, during Interval III truck drivers may have increased their speeds at Station 3 because they did not see any crew working on the bridge.

CONCLUSIONS

Speed reduction effects of continuously transmitting drone radar signals at two construction sites for five time periods are summarized in Figure 4. Experiment 1 indicated that when one drone radar was used for a short period (less than 1 hr) where vehicles were going very fast inside and outside the work zone, speed reductions of nearly 13 to 16 km/hr (8 to 10 mph) were obtained. These reductions were for short time periods and may not represent typical speed reductions due to drone radars.

The results of Experiment 2 indicated that using one radar when drivers knew that it was drone radar did not reduce the average speed of cars or trucks. Also, the decrease in the percentage of vehicles with excessive speeds was relatively small.

The results of Experiment 3 indicated that there were additional speed reductions when drivers could not determine whether it was drone radar. The additional speed reductions were consistent for trucks but not for passenger cars. In five out of six cases, trucks showed statistically significant net speed reductions of 5 to 10 km/hr (3 to 6 mph). Cars showed statistically significant net reductions of 5 km/hr (3 mph) only in two out of six cases. One reason the drone radar was not effective on cars and was less effective on trucks during Interval III might be the absence of crew over the Route 16 bridge during Interval III. Another reason may be that car drivers do not use CB radios and radar detectors as much as truck drivers do. Pigman et al. (*3*) found that only 11 percent of cars (compared with 42 percent of trucks) used radar detectors. The speed reduction effects of drone radar did not diminish on trucks over a period of approximately 3 hr.

The differences in the percentage of vehicles exceeding the speed limits during the two-radar treatment and the control period indicated that at Stations 1 and 3 there were no significant reductions due to the use of radar. However, at Station 2, cars and trucks had speed reductions that decreased over time. The reductions were 12, 7, and 2 percent for cars and 38, 18, and 0 percent for trucks in Intervals I, II, and III, respectively.

Drivers with a radar detector or a CB talked about possible police presence in the work zone. The level of communication indicated that they paid more attention to their speeds in the work zone when threat of police presence existed. Paying more attention to traveling in work zones would, in turn, increase traffic safety in work zones. Thus, another benefit of radar use was an increase in drivers' concern about their speeds, which led to an increase in their awareness and attention in traveling through work zones.

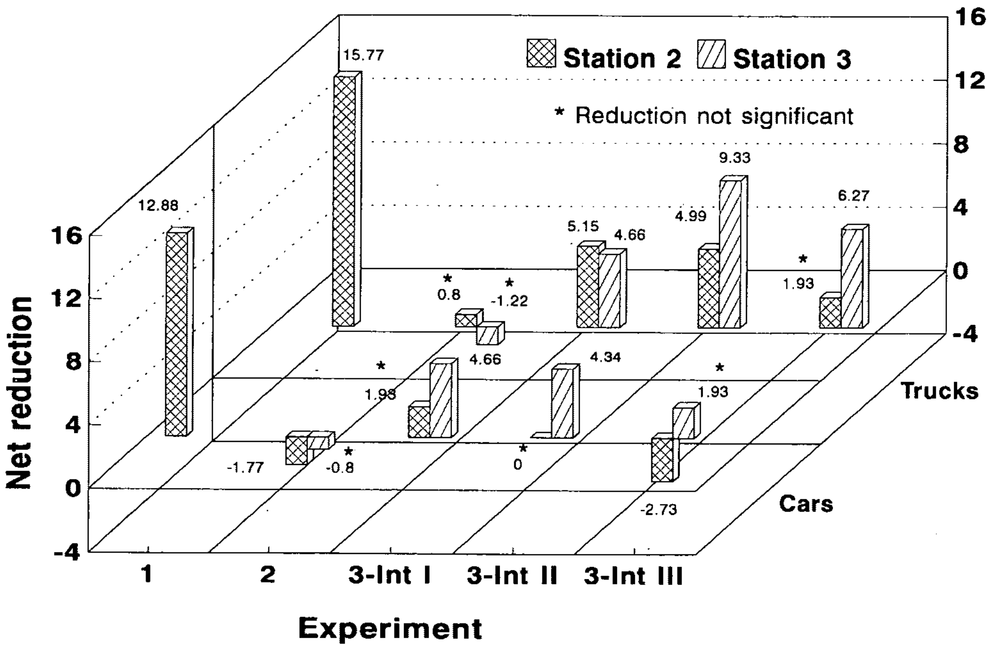


FIGURE 4 Net speed reductions (km/hr), Experiments 1, 2, and 3.

RECOMMENDATIONS

Drone radar may be used effectively to slow down a speeding driver who has a radar detector or uses a CB radio. However, the use of drone radar over a longer period of time diminishes its effects because drivers may detect the absence of active speed enforcement. Therefore, drone radar can be most effective during short periods when drivers have not identified the source of radar transmissions. The number of radars used directly affected the drivers' responses. The location of radar-transmitting stations should be selected to provide maximum threat of police presence and should not be easily identifiable by drivers. Drone radar should be used in conjunction with police enforcement so that drivers are kept "off balance" as to when the radar is real and when it is drone.

This study used conventional radar guns, which are commonly used by law enforcement officers. New radars use laser light pulses instead of radio waves and are called lidar (light detection and ranging). They are also called laser radars because they use laser light pulses. Conventional radars transmit radio waves and cover a wider detection area. However, lidars transmit a very narrow light beam that can be aimed at a specific vehicle in a traffic stream. The speed reduction effects of lidar guns need to be studied.

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Speed Change Distribution of Vehicles in a Highway Work Zone

RAHIM F. BENEKOHAL AND LI WANG

In response to roadway geometry and traffic control devices, motorists may change their speeds within a work zone. Speed profile data for 102 automobiles (cars and vans) and 49 trucks, which traveled on a section of a traffic control zone 2.4 km (1.5 mi) long, were obtained. The average speeds of automobiles and trucks were 8 to 29 km/hr (5 to 18 mph) and 2 to 19 km/hr (1 to 12 mph), respectively, over the work zone speed limit. Vehicles decreased their speeds to the lowest level near the work space (Route 16 bridge). Even at the work space, about 65 percent of automobiles and 47 percent of trucks traveled faster than the speed limit. Automobiles and trucks reduced their speeds by 2 to 21 km/hr (3 to 13 mph) and 5 to 19 km/hr (3 to 12 mph), respectively, compared with their speeds at the beginning of the merging taper. As drivers traveled further into the traffic control zone their speeds first decreased, then slightly increased, and finally reached their minimum value at the work space. After passing it, the speeds continuously increased until vehicles left the study section. Comparisons of speed reductions at similar distances before and after the work space indicated that vehicles attempted to reach the speeds they had before the bridge. The speed reduction distributions for each vehicle group indicated that a small percentage of drivers reduced their speeds by large amounts. Thus, the speed reduction distribution plots were not bell shaped but had long tails (similar to lognormal or Pearson Type III distributions). Statistical analyses based on properties of a normal distribution would not be appropriate for interpretation of speed reduction data for most of the locations within a work zone.

Most drivers slow down when they perceive a potential hazard on the road, such as the presence of crew or large equipment near the traveled lane (*1*). However, the extent of speed reduction for an individual vehicle and the distribution of the reductions at different locations within a work zone are not known. This study was conducted to determine speed reduction distributions of vehicles at different locations within a temporary traffic control zone (work zone). The speed reduction study provides information that is not available from the previous studies (*2–7*), which measured speed at one or two points within the work zones.

The field experiment consisted of obtaining speeds of vehicles as they traveled through the construction zone. The vehicles were videotaped from the time they entered a study section 2.4 km (1.5 mi) long until they left it. A speed reduction profile for each vehicle was computed from the data. Speed reduction effects of various traffic control devices and roadway features may be examined using these data. The terminology suggested by Lewis (*8*) is used whenever possible to identify different locations in a traffic control zone (work zone). According to the terminology, a traffic control zone is divided into four areas—advance warning, transition, activity, and termination. The activity area is further divided into two spaces—buffer space and work space. The work space is only one small part of a work zone.

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STUDY APPROACH

The study approach is based on finding speed reduction profiles of vehicles in a construction zone and performing statistical analyses on the speed reduction effects of work zone roadway features and traffic control devices. The speed of a vehicle was monitored from the time it entered the study section until it exited from it. Two video cameras were used to collect data as vehicles traveled in the traffic control zone.

A vehicle was labeled as influenced if it was slowed down by another vehicle in front of it or exited from the ramp; otherwise it was labeled as uninfluenced. The uninfluenced vehicles were in free flow traffic traveling at their desired speeds in the traffic control zone. The findings of this study are based on the speed characteristics of the uninfluenced vehicles. The uninfluenced vehicles were divided into two vehicle groups—the automobile group and the truck group. The automobile group included passenger cars, vans, and pickup trucks. The vehicles in the truck group are of the tractor-semitrailer type.

Study Site Description

The construction zone was located on Interstate 57 near Mattoon, Illinois. The highway has two lanes per direction, but one lane in each direction was closed because of the construction. The traffic control zone was about 5.6 km (3.5 mi) long. The construction work was mainly repair of bridge decks over State Route 16 and another bridge about 4.0 km (2.5 mi) south of Route 16.

The speed limit inside the construction zone was 72 km/hr (45 mph) for all vehicles. Outside the work zone it was 105 km/hr (65 mph) for cars and 89 km/hr (55 mph) for heavy trucks. The traffic control plan (TCP) used in the work zone was one of the Illinois Department of Transportation's standard TCPs, which is prepared according to the guidelines given in the *Manual on Uniform Traffic Control Devices* (*9*). Figure 1 shows the signs used in this work zone.

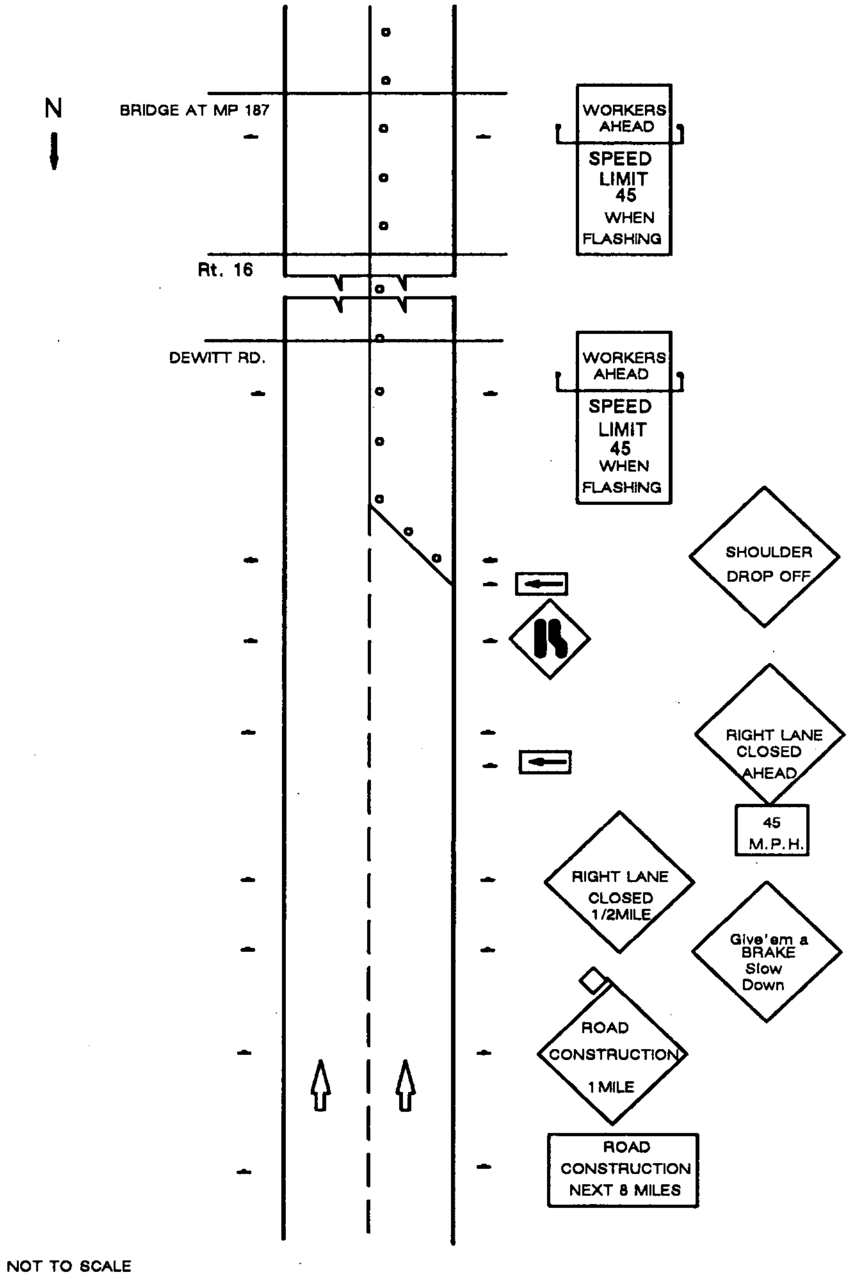


FIGURE 1 Work zone signs on SB I-57 during speed profile study.

Plan and Profile of Study Section

The plan and profile of the highway in the study section and the locations of speed measuring stations as well as the influence points are shown in Figure 2. The crest vertical curve, located in the middle of the study section, was approximately 854 m (2,800 ft) long. It started 122 m (400 ft) before the DeWitt Road overpass and ended 61 m (200 ft) before Route 16. There is a very short section with a 3 percent upgrade slope. The speed reduction due to the uphill section, if any, would be noticeable on the trucks but not on the cars (*10*).

Data Collection

Data were collected during weekdays and under normal weather conditions. Vehicles that were in free flow traffic in the beginning of the study section were videotaped to eliminate the effects of platooning. The average daily traffic on this section of the freeway was around 12,000 vehicles, with approximately 22 percent heavy commercial vehicles (*11*). A total of 208 vehicles were videotaped during the 3 days of data collection. Speed of a vehicle at a given point was computed on the basis of distance and time information. More details on speed calculation are given by Benekohal et al. (*12,13*).

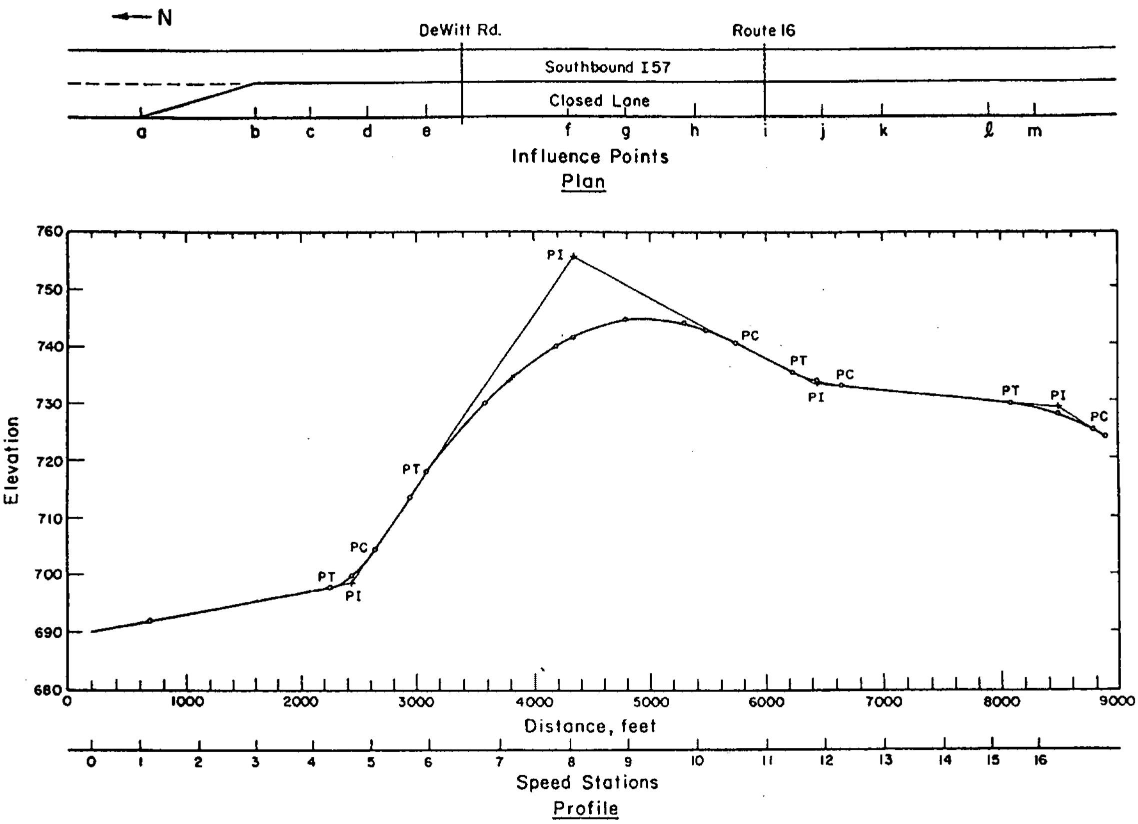


FIGURE 2 Plan, profile, and location of influence points and speed stations.

Data Reduction

Out of 208 vehicles, 57 were labeled influenced. The remaining 151 were labeled uninfluenced. The uninfluenced vehicles were divided into three vehicle types: passenger cars, tractor-semitrailer trucks, and vans and others (such as jeeps and pickup trucks). There were 74 cars, 49 trucks, and 28 vans and other vehicles in the uninfluenced group.

The speed characteristics of the car group were compared with those of the van group to determine whether there were significant differences for the two vehicle types. The results indicated that cars and vans had very similar speed characteristics. Thus, cars and vans were combined into one group, which is called automobiles. Therefore, the findings in this report are for two vehicle groups—the automobile group, which has 102 vehicles, and the truck group, which has 49 vehicles. For each vehicle, several sources of errors were identified, and their effects on speed were calculated. In general, the computed speed could be influenced by 1.6 km/hr (1 mph) or less because of these errors. Further details on data collection and data reduction are given by Benekohal et al. (*12,13*).

Influence Points

Throughout the construction zone, there are traffic control signs and roadway features that may influence the speed of a vehicle. An influence point (IP) is defined as a location within the construction zone that may have such a sign or roadway feature. Thirteen IPs, labeled *a* through *m*, were identified in this study. The IPs and their distances from the beginning of the study section are given in Table 1. The speed of a vehicle at these IPs was determined by using the speed profiles.

TABLE 1 Influence Points and Their Distances from the Beginning of the Study Section

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

[a] 1 foot = 0.305 meter (m)

Effects of Upgrade Slope on Speed

After the construction work was completed, adjustment data were collected to determine the speed reduction effects of the upgrade section. The mean speed reduction was 1.6 km/hr (1 mph) for cars and 8 km/hr (5 mph) for trucks. The speed change for most of the cars was concentrated between −1.6 and 3.2 km/hr (−1 and 2 mph), and for most of the trucks the concentration was between −4.8 and 9.7 km/hr (−3 and −6 mph) (*12*). It was not possible to separate the speed reduction effects of the traffic control devices (i.e., speed limit signs) from that of the upgrade on trucks.

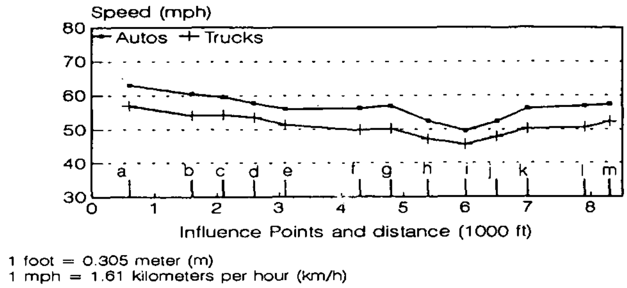


FIGURE 3 Average speed of vehicles at influence points in work zone.

OVERVIEW OF SPEED AND SPEEDING IN WORK ZONE

Speed Characteristics

At each IP, the maximum, minimum, average speed, and standard deviation of speed of automobiles and trucks were computed. These statistics are summarized in Table 2. Automobiles and trucks showed very similar speed characteristics in the study section. The mean speeds of trucks were about 6 to 11 km/hr (4 to 7 mph) lower than that of automobiles at all IPs. The mean speed profile for trucks is parallel to that of automobiles, as shown in Figure 3.

The construction zone over the bridge (work space) was delineated by portable concrete barriers (Jersey barriers). There were Jersey barriers over a length of about 76 m (250 ft). However, the open lane was wide enough [around 4.6 m (15 ft)] and did not give the feeling of going through a narrow lane. Although a previous study indicates that the concrete safety shape (Jersey) barriers do not affect highway capacity even when they are closer than 1.8 m (6 ft) to the traveled lane (*14*, p. 3-11), vehicles decreased their speed when they went through the work space. The main reason for the speed reduction seems to be the construction activities in the work space and the presence of the concrete shape barriers at this location.

Percentage of Automobiles Exceeding a Speed Level

The percentage of vehicles exceeding a given speed decreased over the bridge but increased to the same levels as before when drivers passed the bridge (see Figure 4). The percentage of automobiles exceeding a given speed at the second con-

TABLE 2 Speed Characteristics Statistics for Automobiles and Trucks (mph)

Influence Point	Minimum		Maximum		Mean		Std. Dev [a]	
	Auto	Truck	Auto	Truck	Auto	Truck	Auto	Truck
a	46.1	45.5	79.0	68.8	63.0	57.0	6.83	5.12
b	43.7	38.3	77.9	66.4	60.5	54.2	6.78	5.86
c	44.8	38.9	78.7	66.3	59.6	54.3	7.10	5.85
d	41.7	40.2	77.7	67.0	57.8	53.6	7.24	5.65
e	39.3	39.8	75.2	65.4	56.1	51.4	7.04	5.38
f	42.8	38.0	73.1	61.7	56.3	49.8	7.14	5.20
g	43.0	36.2	73.2	61.2	57.0	50.2	7.14	5.47
h	34.4	33.3	67.3	60.6	52.5	47.2	7.58	5.21
i	24.8	35.3	67.2	59.7	49.6	45.5	9.48	5.13
j	29.8	36.8	68.1	62.5	52.4	47.9	8.35	5.31
k	41.2	42.1	68.7	62.4	56.2	50.2	6.11	4.75
l	44.7	41.9	69.3	62.1	57.0	50.7	5.94	4.76
m	41.8	40.5	72.8	64.8	57.4	52.3	6.73	5.19

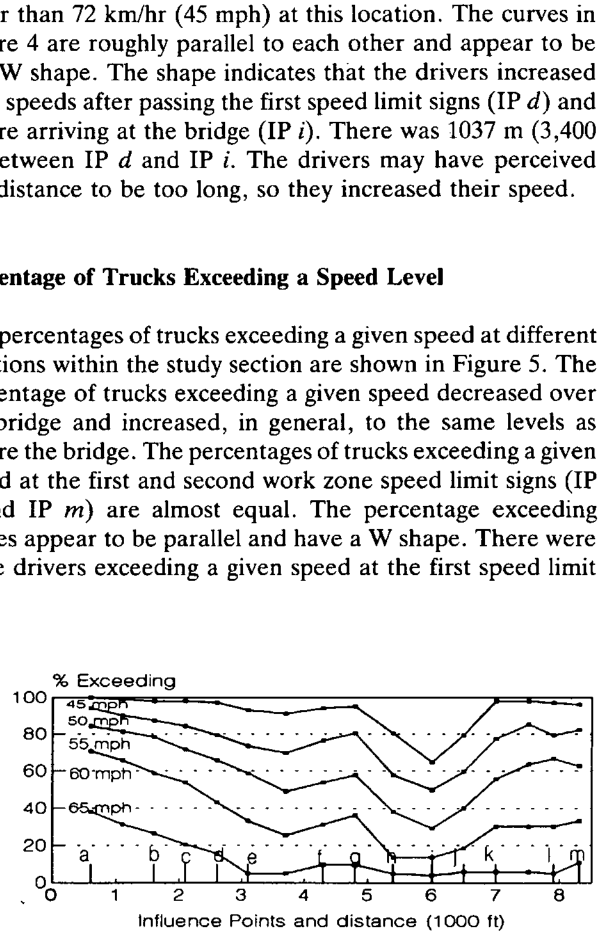
[a]Std. Dev = Standard Deviation
1 mph = 1.61 km/h

struction zone speed limit signs (IP *m*) almost reached the level of the first speed limit signs (IP *d*). This indicates that, on the average, the drivers decreased their speeds to the lowest level near the work space, but after passing it they accelerated to the same speeds they had at the first speed limit signs.

The percentage of automobiles exceeding the speed limit over the bridge (IP *i*) was the lowest compared with other locations; however, nearly 65 percent of automobiles traveled faster than 72 km/hr (45 mph) at this location. The curves in Figure 4 are roughly parallel to each other and appear to be in a W shape. The shape indicates that the drivers increased their speeds after passing the first speed limit signs (IP *d*) and before arriving at the bridge (IP *i*). There was 1037 m (3,400 ft) between IP *d* and IP *i*. The drivers may have perceived this distance to be too long, so they increased their speed.

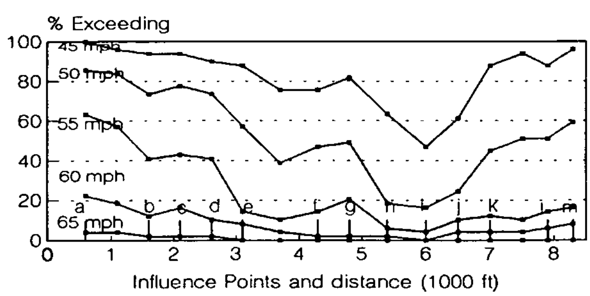
Percentage of Trucks Exceeding a Speed Level

The percentages of trucks exceeding a given speed at different locations within the study section are shown in Figure 5. The percentage of trucks exceeding a given speed decreased over the bridge and increased, in general, to the same levels as before the bridge. The percentages of trucks exceeding a given speed at the first and second work zone speed limit signs (IP *d* and IP *m*) are almost equal. The percentage exceeding curves appear to be parallel and have a W shape. There were more drivers exceeding a given speed at the first speed limit sign than at the bridge. At the bridge, 47 percent of trucks traveled faster than 72 km/hr (45 mph).



1 foot = 0.305 meter (m)
1 mph = 1.61 kilometers per hour (km/h)

FIGURE 4 Percentage of automobiles exceeding given speeds at influence points in work zone.



1 foot = 0.305 meter (m)
1 mph = 1.61 kilometers per hour (km/h)

FIGURE 5 Percentage of trucks exceeding given speeds at influence points in work zone.

SPEED REDUCTIONS AT INFLUENCE POINTS

In this section the distribution of speed differences rather than the difference of the average speeds is used to examine relative effects of roadway features and traffic control devices on speeds of vehicles. The speed reduction distribution provides more insight into drivers' responses than the difference of average speeds. To find the distribution, the speed difference at a given IP compared with a reference IP was computed for each vehicle. The speeds at all IPs were compared with the speeds at the first IP (IP *a*). In addition, speeds at selected pairs of IPs were compared with each other.

Paired *t*-tests were used to compare the speed differences between pairs of IPs. In a paired *t*-test, for a given pair of IPs, the mean of speed differences rather than the difference of the mean speeds is used to make statistical inferences. The results from the paired *t*-test analysis would help to examine how roadway features and traffic control devices affected the speed of vehicles and whether the effects were statistically significant.

Reductions Compared with Speed at the Beginning

Speed Change Statistics

Summaries of speed changes are given in Tables 3 and 4. The average speed reductions varied from 4.0 km/hr (2.5 mph) to 21.6 km/hr (13.4 mph) for automobiles and from 4.2 km/hr (2.6 mph) to 18.5 km/hr (11.5 mph) for trucks. As drivers traveled further into the traffic control zone, the reduction first increased, then slightly decreased, and finally reached its maximum value at the bridge. Beyond the bridge, the speed reductions continuously decreased until vehicles exited the study section. The standard deviations given in Tables 3 and 4 reflect the degree of the concentration of the speed reductions.

Instead of speed change confidence intervals, observed ranges for 90 percent of the speed changes at each IP are presented in Tables 3 and 4. Since most of the speed reductions were not normally distributed, the method of finding the confidence interval for the normal distribution should not be used here. For example, if the speed reduction had been normally distributed for automobiles at IP *f*, 90 percent of speed reductions would have been within 26.9 and −5.3 km/hr (16.7 and −3.3 mph). However, 90 percent of the observed speed reductions were within −29.6 and −1.1 km/hr (−18.4 and −0.7 mph). This example illustrates the importance of knowing the distribution of the differences to avoid an error in interpretation of the speed reduction data.

Speed Reduction Distributions

The speed reduction distributions for automobiles and trucks are given in Figure 6. Almost all of the frequency plots show a small percentage of drivers who reduced their speeds by a

TABLE 3 Average of Individual Vehicle Speed Changes Between Pairs of Influence Points for Automobiles (mph)

Influence Points Compared	Average Speed Differences	Standard Deviation	Standard Error	Confidence Level Speeds Are Different	Observed Range for 90% of Speed Changes	
					Lower Limit	Upper Limit
(IP b)-(IP a)	-2.5	2.80	0.28	99.99	-8.5	0.7
(IP c)-(IP a)	-3.4	3.76	0.37	99.99	-10.3	0.4
(IP d)-(IP a)	-5.2	4.67	0.46	99.99	-15.4	0.4
(IP e)-(IP a)	-6.9	5.21	0.52	99.99	-15.8	-1.1
(IP f)-(IP a)	-6.7	6.08	0.60	99.99	-18.4	0.7
(IP g)-(IP a)	-6.0	6.42	0.64	99.99	-18.4	2.5
(IP h)-(IP a)	-10.5	7.32	0.72	99.99	-23.8	-0.2
(IP i)-(IP a)	-13.4	9.02	0.89	99.99	-29.1	-0.5
(IP j)-(IP a)	-10.6	7.82	0.77	99.99	-25.1	-0.7
(IP k)-(IP a)	-6.8	6.02	0.60	99.99	-17.2	1.2
(IP l)-(IP a)	-6.0	5.49	0.54	99.99	-15.8	1.3
(IP m)-(IP a)	-5.6	6.14	0.61	99.99	-16.3	3.0
(IP h)-(IP j)	0.04	4.47	0.44	6.76	-5.7	7.9
(IP c)-(IP e)	3.5	3.56	0.35	99.99	-1.0	10.8
(IP d)-(IP m)	0.4	5.42	0.54	50.21	-7.8	10.2
(IP g)-(IP k)	0.9	4.37	0.43	94.83	-5.5	9.8

1 mph = 1.61 km/h

large amount. These drivers are represented by the left tail of the frequency curves.

The frequency distributions are not bell shaped. Most of the speed reduction distributions have a long tail, and they are similar to lognormal or Pearson Type III distributions. These shapes must be considered in interpreting speed reduction data. For example, the average speed reduction of automobiles at the end of the taper was 4.0 km/hr (2.5 mph). This indicates that automobile drivers reduced their speeds, on the average, by 4.0 km/hr (2.5 mph) when they reached the end of the taper. However, the speed reduction frequency distribution for IP *b* shows that 33 percent of automobiles reduced their speeds less than 1.6 km/hr (1 mph), and 53 percent of them reduced less than 3.2 km/hr (2 mph) between the beginning and end of the taper. Thus, the average speed reduction is greatly influenced by the 47 percent of automobiles that had reductions larger than 3.2 km/hr (2 mph). Similar arguments can be made for IP *c* and IP *d*.

Similarly, for trucks the average speed reduction at the end of the taper was 4.3 km/hr (2.7 mph). Nevertheless, nearly 22 percent of trucks reduced their speeds less than 1.6 km/hr (1 mph), and 47 percent of them reduced speed by less than 3.2 km/hr (2 mph) between the beginning and end of the taper. Thus, the remaining 53 percent significantly influenced the average speed reduction value. A similar analysis can be made for IP *c* and IP *d*.

TABLE 4 Average of Individual Vehicle Speed Changes Between Pairs of Influence Points for Trucks (mph)

Influence Points Compared	Average Speed Difference	Standard Deviation	Standard Error	Confidence Level Speeds Are Different	Observed Range for 90% of Speed Changes	
					lower Limit	Upper Limit
(IP b)-(IP a)	-2.7	2.14	0.31	99.99	-5.8	0.8
(IP c)-(IP a)	-2.6	2.47	0.35	99.99	-6.5	0.6
(IP d)-(IP a)	-3.4	2.76	0.39	99.99	-7.3	0.6
(IP e)-(IP a)	-5.6	3.20	0.46	99.99	-10.7	-1.4
(IP f)-(IP a)	-7.2	4.22	0.60	99.99	-15.5	-1.2
(IP g)-(IP a)	-6.7	4.78	0.68	99.99	-15.2	-0.3
(IP h)-(IP a)	-9.8	5.24	0.75	99.99	-18.8	-2.2
(IP i)-(IP a)	-11.5	6.03	0.86	99.99	-22.0	-2.2
(IP j)-(IP a)	-9.1	6.09	0.87	99.99	-19.2	-0.3
(IP k)-(IP a)	-6.7	5.21	0.74	99.99	-14.7	0.4
(IP l)-(IP a)	-6.3	4.69	0.67	99.99	-12.3	1.1
(IP m)-(IP a)	-4.7	5.00	0.71	99.99	-11.9	3.1
(IP h)-(IP j)	-0.7	3.44	0.49	82.55	-4.9	5.4
(IP c)-(IP e)	3.0	2.48	0.35	99.99	-0.1	6.5
(IP d)-(IP m)	1.3	4.24	0.61	96.61	-5.3	7.9
(IP g)-(IP k)	0.01	3.73	0.53	1.58	-6.2	6.5

1 mph = 1.61 km/h

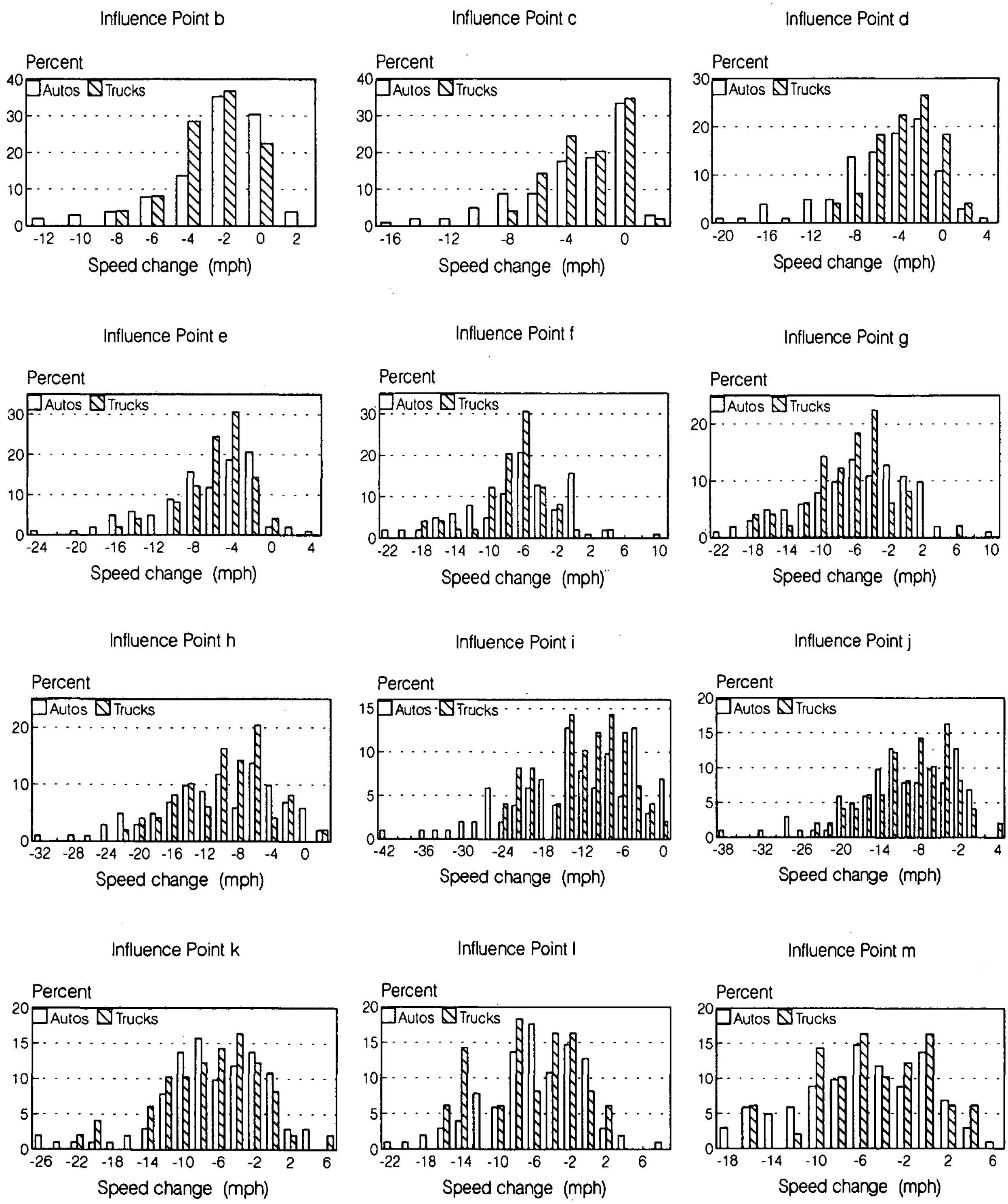
Normality Test for Automobiles and Trucks

As mentioned earlier, it is noticeable that not all of the speed reduction distributions were bell shaped. The distributions around the bridge are closer to a normal distribution than those of the other points. Statistical tests are needed to determine which distributions are normal. The method used here is the Shapiro-Wilk's statistic for normality testing (*15*). In this method, the Shapiro-Wilk statistic, W, which is the ratio of the best estimator of the variance (based on the square of a linear combination of the order statistics) to the usual corrected sum of squares estimator of the variance, is computed, where $W > 0$ and $W \leq 1$. Smaller values of W lead to rejection of the null hypothesis that the distributions are normal (*15*). Table 5 gives the normality test results for the distributions shown in Figure 6.

Speed reduction distributions for automobiles at all IPs are not normally distributed with a 90 percent confidence level (Table 5). Similarly, the speed reduction distributions for trucks are not normally distributed except at four locations near the bridge. These four IPs are located at 366 m (1,200 ft) and 183 m (600 ft) before the bridge and 153 m (500 ft) and 305 m (1,000 ft) after the bridge.

Speed Change Profile for Automobiles

The speed change profile and observed ranges for 90 percent of the speed changes for automobiles are given in Table 3.



1 mph = 1.61 kilometers per hour (km/h)

FIGURE 6 Distribution of speed changes at Influence Points *b* through *m* compared with Influence Point *a*.

TABLE 5 Shapiro-Wilk's Normality Test Results and Interpretation of Them with 90 percent Confidence Level

Influence Point	Auto		Truck	
	Prob<W	Normal	Prob<W	Normal
IP b	0.0001	No	0.0001	No
IP c	0.0001	No	0.0001	No
IP d	0.0001	No	0.0179	No
IP e	0.0001	No	0.0050	No
IP f	0.0184	No	0.0068	No
IP g	0.0393	No	0.1875	Yes
IP h	0.0098	No	0.2799	Yes
IP i	0.0006	No	0.0857	No
IP j	0.0001	No	0.3254	Yes
IP k	0.0001	No	0.7761	Yes
IP l	0.0905	No	0.0313	No
IP m	0.0735	No	0.0549	No

Automobile drivers displayed, on the average, a 4.0-km/hr (2.5-mph) speed reduction at the end of the taper compared with the beginning of it. They continued reducing their speeds after passing the taper. At the first speed limit signs (IP *d*), the mean speed reduction was about 8 km/hr (5 mph). The reduction fluctuated between 10 and 11 km/hr (6 and 7 mph) until the vehicles reached IP *h*. At IP *h*, which is about 183 m (600 ft) before the bridge, the reduction increased to 16.9 km/hr (10.5 mph). The maximum average speed reduction was 21.6 km/hr (13.4 mph), which occurred over the bridge (IP *i*). The maximum speed reduction for automobiles was 67.5 km/hr (41.9 mph), which also happened at the bridge. The standard deviation of speed differences increased as automobiles approached the work space, and the largest one [14.52 km/hr (9.02 mph)] occurred at IP *i*.

After passing the bridge, the speed reductions became smaller as drivers traveled further away from the bridge. The reductions on either side of the bridge are very similar, indicating that the drivers, after passing the bridge, almost reached the speeds they had before the bridge.

Speed Change Profile for Trucks

Trucks showed speed reduction patterns (Table 4) similar to those of automobiles. The largest average speed reduction was 18.5 km/hr (11.5 mph), which occurred over the bridge (IP *i*). The maximum speed reduction for trucks was 39.1 km/hr (24.3 mph), which also happened at the bridge. The standard deviation of speed differences increased as trucks approached the work space, and the largest ones, 9.71 and 9.80 km/hr (6.03 and 6.09 mph), occurred at IP *i* and IP *j*. After passing the bridge, truck drivers also increased their speeds even though there was another pair of speed limit signs ahead. And the reductions on either side of the bridge are very similar. This means that after passing the work space, the drivers almost reached the speed they had before it.

Reductions Compared with Other Points

Automobiles

Further comparisons between pairs of IPs were made (Table 3). The first pair to be considered were the points 183 m (600 ft) before and 153 m (500 ft) after the bridge (IP *h* versus IP *j*). For this pair, the speed difference for automobiles was 0.06 km/hr (0.04 mph), indicating that the speeds at these points were not significantly different. This means that the drivers reduced their speeds over the bridge but increased them to the same level as before the work space.

The second pair of IPs compared were the points 153 m (500 ft) before and 153 m (500 ft) after the first speed limit signs (IP *c* versus IP *e*). The mean speed reduction was 5.6 km/hr (3.5 mph), which is a significant reduction. This means that on the average the speed was reduced 5.6 km/hr (3.5 mph) around the first speed limit signs. Assuming that at 153 m (500 ft) before the first speed limit signs the drivers had reached their desired speed, the mean speed difference of 5.6 km/hr (3.5 mph) is mainly caused by the speed limit signs.

Thus, one may conclude that the speed limit signs were effective in reducing the average speed of automobiles by 5.6 km/hr (3.5 mph) at a point immediately after the signs. The adjustment data showed that the mean speed reduction for automobiles caused by the upgrade segment would be less than 1.6 km/hr (1 mph) for the 305-m (1,000-ft) travel distance.

The third comparison was between IP *d* and IP *m*, where the first speed limit signs and the second speed limit signs were located. The difference in reductions between these two IPs was 0.6 km/hr (0.4 mph), which was not significant. This indicates that automobiles drove at similar speeds at these two points.

The last comparison pair was IP *g* and IP *k*, 366 m (1,200 ft) before and 305 m (1,000 ft) after the bridge. The reduction difference between them was 1.4 km/hr (0.9 mph). This indicates that after traveling about 305 m (1,000 ft) past the bridge, vehicles attempted to reach the speed they had 366 m (1,200 ft) before the bridge.

Trucks

As for automobiles, further comparisons were made between pairs of IPs to assess the reductions at selected points (Table 4). The same IPs were selected for trucks as for automobiles. First to be considered were the points 183 m (600 ft) before and 153 m (500 ft) after the bridge (IP *h* versus IP *j*). For this pair, the speed difference for trucks was 1.1 km/hr (0.7 mph), indicating that the average speed of trucks 366 m (600 ft) before the bridge was not significantly different from that 153 m (500 ft) after the bridge. This means that the drivers reduced their speed over the bridge but increased it to the same level as before the work space.

The second comparison was between the points 153 m (500 ft) before and 153 m (500 ft) after the first speed limit signs (IP *c* versus IP *e*). The mean speed reduction for this pair was

4.8 km/hr (3.0 mph), a significant amount. This means that on the average the speed was reduced by 4.8 km/hr (3.0 mph) around the first speed limit signs. Assuming that at 153 m (500 ft) before the first speed limit signs the drivers had reached their desired speeds, the mean speed difference of 4.8 km/hr (3.0 mph) is mainly caused by the speed limit signs. A portion of this reduction may be due to the upgrade segment on the highway, but that portion cannot be determined from the available data. Thus, considering the upgrade effect, it can be concluded that the trucks on the average reduced their speeds by less than 4.8 km/hr (3 mph) immediately after passing the speed limit signs.

The third comparison was between IP *d* and IP *m*, where the first speed limit signs and the second speed limit signs were located. The difference in reductions between these two IPs was 2.1 km/hr (1.3 mph), which was not significant. This indicates that although the trucks reduced their speeds over the bridge, by the time they reached IP *m* they increased their speeds to the speed level they had at IP *d*.

The last comparison pair was IP *g* and IP *k*, 366 m (1,200 ft) before and 305 m (1,000 ft) after the bridge. The reduction difference between them was 0.02 km/hr (0.01 mph). This indicates that after traveling about 305 m (1,000 ft), vehicles attempted to reach the speed they had 366 m (1,200 ft) before the bridge.

CONCLUSIONS

Automobiles and trucks decreased their speeds to the lowest level near the work space, but after passing it they increased their speeds to the higher levels they had before the work space. The percentage of vehicles exceeding a speed level decreased as they approached the work space (bridge), but after passing the work space the percentage increased to the higher levels found before the work space. Even at the work space nearly 65 percent of automobiles and 47 percent of trucks were speeding.

Automobiles and trucks, on the average, traveled 5 to 21 km/hr (3 to 13 mph) and 5 to 19 km/hr (3 to 12 mph), respectively, slower inside the traffic control zone compared with their speeds at the beginning of the merging taper. As drivers traveled further into the traffic control zone, the speed reductions first increased, then slightly decreased, and finally reached a maximum value at the bridge. Beyond the bridge, the speed reductions continuously decreased until vehicles left the study section.

A small percentage of drivers reduced their speeds by large amounts; thus, the mean speed is influenced by these large reductions. The speed reduction frequency distribution plots were not bell shaped at most locations but had a long tail (similar to lognormal or Pearson Type III distributions).

Comparisons of speed reductions at similar distances before and after the work space indicated that vehicles attempted to reach the speeds they had before the bridge. The speed reductions before and after the first work zone speed limit signs were also compared. The speed limit signs were found to be effective in reducing the average speed of automobiles by 5.6 km/hr (3.5 mph) and that of trucks by less than 4.8 km/hr (3.0 mph) at a point immediately after the signs.

RECOMMENDATIONS

The locations at which drivers slow down or speed up are critical points in a construction zone. Knowing these points would help in placing the signs at appropriate locations. It is recommended that the placement and frequency of the work zone speed limit signs be examined using the speed reduction pattern of the vehicles. The location of the signs and the length of the section before the work space should be such that most drivers are encouraged to follow the speed limit.

The analysis indicated that location of a speed-measuring station has to be carefully selected because it will affect the outcome of the measurements. Furthermore, speed distributions, as well as the mean speeds, should be analyzed to obtain more accurate speed characteristic data. Speed profile data from other work zones should be used to further validate the findings of this study.

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Effect of Nonpermanent Pavement Markings on Driver Performance

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A study was conducted to determine the effect on driver performance of different length pavement markings commonly used in work zones. The study was conducted on a divided multilane facility, and the two nonpermanent, or temporary, marking patterns examined were 0.61-m stripes with 11.59-m gaps (2-ft stripes with 38-ft gaps) and 1.22-m stripes with 10.98-m gaps (4-ft stripes with 36-ft gaps). Both of these patterns were compared with the full complement of markings [i.e., 3.05-m stripes with 9.15-m gaps (10-ft stripes with 30-ft gaps) and edge lines]. The field data collection effort consisted of following randomly selected traffic stream vehicles through a segment of roadway marked with one of the patterns noted. The maneuvers of each of the 436 vehicles followed in this manner were recorded on videotape. The tape was then used to obtain the measures of effectiveness (MOEs) necessary to evaluate driver performance as related to the pavement marking patterns. The MOEs used included lateral placement of the vehicle on the roadway, vehicle speed within the test segment, number of edge line and lane line encroachments, and number of erratic maneuvers. For each operational measure examined, the results of the analysis indicated that drivers performed better with the 3.05-m (10-ft) markings that included edge lines. This result is reasonable and was expected. However, the analysis also indicated that drivers generally performed better with the 1.22-m (4-ft) lane lines than with the 0.61-m (2-ft) lane lines, particularly under adverse weather conditions.

Road construction and maintenance operations, such as pavement overlay projects, often require the use of temporary pavement markings. Such markings must provide a level of guidance for the driver that will ensure safe travel. Using the concepts of positive guidance (i.e., combining traffic engineering and human factors technologies), the markings provided must enable a driver to determine the appropriate path and speed (*1*). If the markings are inadequate, the driver may choose an inappropriate path or speed, which may result in an accident.

Through the conduct of a large number of research and accident studies, it has been determined that the current recommended standard for permanent broken lines, either centerlines or lane lines, meets the needs of drivers in providing the appropriate level of guidance. The *Manual on Uniform Traffic Control Devices* (MUTCD) defines this standard for a broken line as a combination of stripes and gaps, usually in the ratio of 1:3, with the most typical pattern consisting of 3.05-m (10-ft) stripes and 9.15-m (30-ft) gaps (*2*).

Whereas the standards for permanent markings are widely accepted, there are varying opinions regarding temporary, short-term, or nonpermanent markings. (The MUTCD first used the term "temporary" pavement markings. In the 1988 edition of the MUTCD, the term was changed to "short-term." Currently, the term "nonpermanent" is being used in the revision of Part VI of the MUTCD now in the process of proposed rule making for final acceptance.) In a 1986 survey conducted by the Traffic Engineering Section of the Arizona Department of Transportation, it was discovered that 15 different temporary marking patterns were in use in 50 states (*3*).

This lack of consistency among states and the need to improve safety in work zones resulted in the development of the current FHWA policy on nonpermanent pavement markings. This policy was first incorporated into the MUTCD as section 6D-3 with a compliance date of January 1989. The official ruling regarding the incorporation of the new policy indicates the intention of creating uniformity and providing additional guidance with respect to nonpermanent pavement markings.

Nonpermanent pavement markings are defined in the MUTCD as "those that may be used until the earliest date when it is practical and possible to install pavement markings that meet the full MUTCD standards for pavement markings." For nonpermanent broken-line pavement markings, the MUTCD recommends 1.22-m (4-ft) stripes and 10.98-m (36-ft) gaps, with some exceptions. It is this recommended broken-line marking that is presently being questioned. Of those 50 states surveyed, 33 used markings less than 4 ft (1.22 m) or gaps longer than 10.98 m (36 ft). It is the concern in many of these states that the newly recommended standard of 1.22-m (4-ft) stripes and 10.98-m (36-ft) gaps will significantly increase project costs while not providing any additional safety benefits.

The lack of information related to nonpermanent pavement markings and the benefits and costs associated with different marking patterns makes the decisions related to policy development difficult. This study was undertaken to determine the operational effects of different marking patterns on driver behavior so that future decisions regarding nonpermanent pavement markings can be based on sound transportation engineering research.

LITERATURE REVIEW

A number of studies have been conducted examining the retroreflectivity and reliability of permanent markings and raised pavement markers (RPMs). Likewise, a large number of efforts have been undertaken to determine the effectiveness of various work zone traffic control devices, including delin-

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eators, beacons, drums, and so forth. However, few studies have examined the effectiveness of nonpermanent pavement markings. A summary of two recent research efforts in which a number of temporary pavement marking patterns were studied is presented below.

A 1986 study by Dudek et al. (*4*) examined the effectiveness of 10 temporary marking treatments (see Table 1) on various measures of driver performance under dry weather and road conditions only. All 10 treatments were tested during the day, and the 7 most effective were examined at night. Two of the most effective treatments were Treatments 1 and 2, both of which were examined in the current study. The experiment consisted of having test subjects traverse a 9.7-km (6-mi) test track, which included several horizontal curves and simulated a two-lane, two-way roadway with 3.36-m (11-ft) lanes, including a standard centerline and edge lines outside the treatment zones. The treatments studied were placed on four horizontal curves on the track, with the edge lines being dropped 153 m (500 ft) before the beginning of the curve and continuing 153 m (500 ft) after the curve (*4*).

The measures of effectiveness (MOEs) used in evaluating the treatments included

1. Speed and distance measurements, such as maximum entry speed into the curve, minimum speed while in the curve, and magnitude of the speed change;
2. Erratic maneuvers, such as lateral deviations or completely missing the curve; and
3. Subjective comments and ratings of the treatments by the drivers.

Results found by Dudek et al. pertaining to the two nonpermanent treatments examined in the current study were as follows:

- There were no practical differences between the treatments, either daytime or nighttime, in MOEs developed from speed and distance measurements. Practical differences were arbitrarily defined as at least 6.4 km/hr (4 mi/hr) for speed measures and 0.31 m (1 ft) for distance measures. There were also no significant differences in the erratic maneuver data between the two treatments.
- Treatment 2 [0.61-m (2-ft) stripes with 11.59-m (38-ft) gaps] was associated with some drivers missing the curve and with a few wide deviations to the right of the centerline. Subjectively, this treatment was rated as the least effective.
- Treatment 1 [1.22-m (4-ft) stripes with 10.98-m (36-ft) gaps], which was the baseline condition and is the current recommended standard in the MUTCD, was rated average in terms of effectiveness by the drivers during the daytime studies. The erratic maneuver data also showed that this treatment resulted in relatively few complete misses of the curve during the day, but a relatively high frequency of deviations from the centerline.
- Although Treatment 1 [1.22-m (4-ft) stripes with 10.98-m (36-ft) gaps] was not the preferred choice of drivers during the nighttime tests, the performance data did not indicate any differences between that treatment and the preferred treatments, which included the use of RPMs.

The second study related to this topic was an NCHRP research effort in which Dudek et al. compared 0.31-m, 0.61-m, and 1.22-m (1-ft, 2-ft, and 4-ft) temporary broken-line markings in work zones during the night under dry weather conditions. The field studies were conducted at seven pavement overlay projects on two-lane, two-way roadways in four states. The sites selected had 3.66-m (12-ft) lanes, paved shoulders [1.22 to 3.05 m (4 to 10 ft)], lengths ranging from 772 to 2,044 m (2,530 to 6,700 ft), and annual average daily traffic counts ranging from 2,750 vehicles to 9,600 vehicles. Each site contained a tangent section and a horizontal curve of 2.0 degrees, with the exception of one with a 3.0-degree curve. The material used for the temporary centerline markings was yellow retroreflective tape (*5*).

Traffic stream studies conducted included comparisons of operational measures among the three sets of markings. The MOEs evaluated included vehicle speeds, lateral distances from the centerline to the left front tire, centerline encroachments, and erratic maneuvers. Results from the studies showed no practical significant differences [≥6.4 km/hr (4 mi/hr)] between the three striping patterns with respect to vehicle speeds. There were also no statistical or practical differences [≥0.31 m (1 ft)] between the marking patterns in the comparison of lateral distance from the centerline. The remaining

TABLE 1 Temporary Pavement Marking Patterns Evaluated in Proving Ground Studies (*4*)

Treatment	Description
1[a]	1.22 m stripes (10.2 cm wide) with 10.98-m gaps (control condition)
2[a]	0.61-m stripes (10.2 cm wide) with 11.59-m gaps
3[a]	2.44-m stripes (10.2 cm wide) with 9.76-m gaps
4[a]	0.61-m stripes (10.2 cm wide) with 5.49-m gaps
5[a]	Four nonretroreflective RPM's at 1.02-m intervals with 9.15-m gaps and one retroreflective marker centered in alternate gaps at 24.40-m intervals
6[a]	Three nonretroreflective and one retroreflective RPM at 1.02-m intervals with 9.15-m gaps
7	0.61-m stripes (10.2 cm wide) with 14.64-m gaps
8	Treatment 2 plus RPM's at 24.40-m intervals
9[a]	Two nonretroreflective RPM's at 1.22-m intervals with 10.98-m gaps plus one retroreflective RPM centered in each 10.98-m gap
10	0.31-m stripes (10.2 cm wide) with 5.80-m gaps

[a] Treatments evaluated both day and night.
1 m = 3.28 ft; 1 cm = 0.39 in

MOEs, centerline encroachments and erratic maneuvers, were noted as being infrequent or nonexistent.

In addition to the traffic stream studies, paid driver subjects were recruited to drive through the test segments and rate the different marking patterns. The results of this effort showed no significant differences between the ratings for the three marking patterns. However, the general trend indicated that the 0.31-m (1-ft) stripe was ranked slightly poorer and that the drivers preferred the longer 1.22-m (4-ft) stripe.

In summary, these two studies produced no strong evidence to indicate that 1.22-m (4-ft) stripes with 10.98-m (36-ft) gaps were any more effective in providing driver guidance than the 0.31-m (1-ft) stripes with 11.90-m (39-ft) gaps or 0.61-m (2-ft) stripes with 11.59-m (38-ft) gaps. However, as noted by the authors of these efforts, the research conducted was limited in scope, and thus the results obtained could only be applied to those situations tested. Their suggestions for future research related to this issue included determining effectiveness of the different marking patterns under adverse weather conditions.

GENERAL RESEARCH APPROACH

The objective of this study was to determine the effect of nonpermanent pavement markings on driver performance. Three marking patterns were tested:

- 0.61-m stripes with 11.59-m gaps (2-ft stripes with 38-ft gaps),
- 1.22-m stripes with 10.98-m gaps (4-ft stripes with 36-ft gaps), and
- 3.05-m stripes with 9.15-m gaps (10-ft stripes with 30-ft gaps) and edge lines.

The first two patterns are the temporary markings examined, whereas the third scenario is the full complement of markings recommended in the MUTCD. Data were collected for all three marking patterns during day and night and under dry and wet weather conditions. (Collection of data under the full complement of markings provided a baseline performance measure, that is, an indication of how drivers normally perform with all markings present.)

The data analysis consisted of comparing a number of operational measures collected for the three marking patterns, with the principal comparison being between the two temporary marking patterns. The MOEs selected were defined to provide a clear indication of the differences in driver performance associated with the different marking patterns and included the following:

- Lateral placement of the vehicle on the roadway: Typically, drivers will attempt to center their vehicles in the travel lane. The amount of deviation from this position provides an indication of accident potential, either a run-off-road type accident to the right or a sideswipe accident to the left when the vehicle is in the right lane of a multilane facility. The lateral placement measure used in the analysis was the distance from the lane line to the center of the vehicle.
- Vehicle speed within the test segment: The speed at which a vehicle traverses the study site provides a measure directly related to the ability of the driver to determine the appropriate travel path. The inability to perform this task may result in an accident, either into another vehicle in an adjacent lane or into a fixed object off the roadway. A difference in speed between two marking patterns indicates that drivers need to travel slower under one scenario to see the markings and determine the correct path of travel. The speed selected for the analysis was the average running speed over the test segment.
- Number of edge line and lane line encroachments: These operational measures are similar to lateral placement in that they indicate the potential of an accident resulting from inappropriate lateral position. The number of encroachments occurring during each run was the measure used in the analysis.
- Number of erratic maneuvers: Occurrences such as sudden speed or directional changes and brake applications are performance variables that measure the ability of the driver to select the appropriate travel path. Making such maneuvers while driving through the test segment is indicative of a driver's inability to select a proper path on the basis of the information available (i.e., pavement markings).

The results presented in this paper are for operations on divided multilane facilities. The temporary broken-line pavement marking has two specific applications as stated in the MUTCD: to provide (*a*) white lane lines for traffic moving in the same direction on multilane facilities and (*b*) yellow centerlines on two-lane, two-way roadways where it is safe to pass.

In this research, only the lane line application on multilane facilities was examined. Since the objective of this study was to determine the operational effects of different lane line patterns without the effect of other markings, a divided multilane facility was selected as the test segment. On the basis of FHWA policy as documented in the MUTCD, the only marking present on this type of roadway under temporary conditions would be the lane line. On an undivided multilane roadway, the permanent centerline would be marked in addition to the temporary lane lines. This centerline could obviously affect driver performance and, consequently, distort any results obtained regarding the effects of the lane line.

With regard to the application of centerlines on two-lane, two-way roadways, the study by Dudek et al. (*5*) previously examined temporary pavement markings on two-lane roadways. The missing element in that effort was the effect of adverse weather conditions on driver performance. Such conditions were studied in this project and provide insight into the effects associated with different marking patterns and adverse weather conditions.

DATA COLLECTION

Site Selection

With the help of the Virginia Department of Transportation, several divided multilane sites scheduled for pavement overlay were identified. The site finally selected was a 6.4-km (4-mi) segment of southbound Interstate 85 extending from the intersection with Virginia State Route 903 (Exit 1) to the North Carolina state line (see Figure 1). The test segment with the temporary markings began 122 m (400 ft) south of

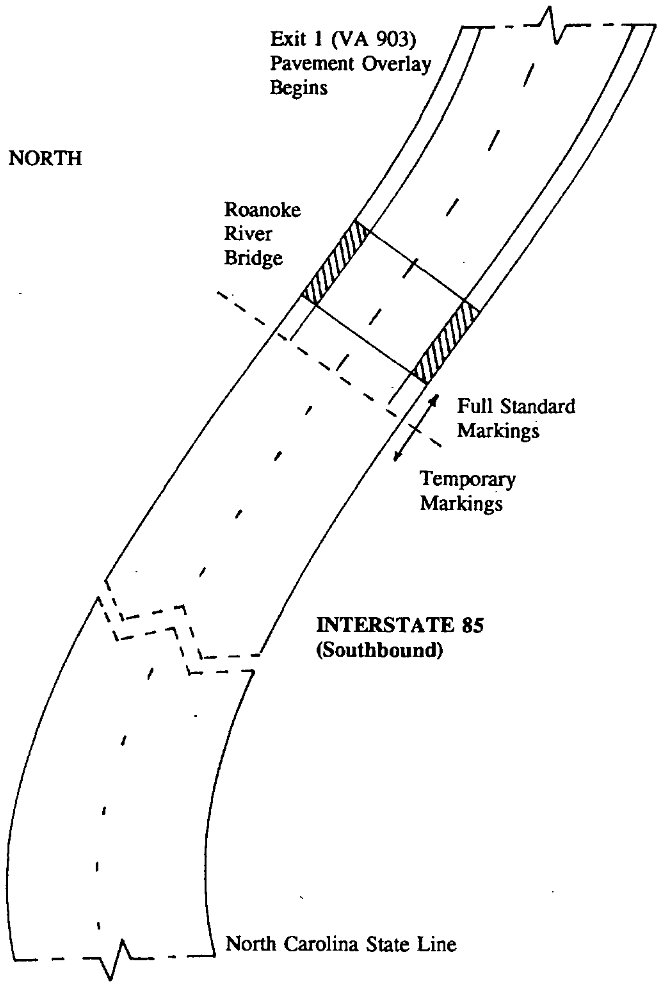


FIGURE 1 Field study site.

the Roanoke River bridge and continued to the state line with a total length of 5.0 km (3.1 mi). The terrain was relatively flat and the curvature was mild. The cross section of the roadway, once the final markings were in place, would consist of a 3.05-m (10-ft) right shoulder, two 3.66-m (12-ft) lanes, and a 1.22-m (4-ft) left shoulder.

Pavement Markings

Once the pavement overlay work was completed, the placement of the markings began. The marking material used for all three patterns tested was retroreflective paint, which was the material to be used for the permanent markings once the data collection for this research study was completed. All markings were 10.2 cm (4 in.) wide and were placed with a typical high-speed pavement marking truck in a rolling lane closure.

The first set of markings placed was the full complement of markings, as recommended in the MUTCD, from the beginning of the overlay segment to a point 122 m (400 ft) past the Roanoke River bridge. This was done as a safety measure to avoid having any temporary markings on the approach to the bridge where the shoulders tapered down to 0.61 m (2 ft) on either side. The first marking pattern [0.61-m (2-ft) stripe with 11.59-m (38-ft) gap] was then placed from the point below the bridge where the full complement of markings stopped to the state line. This pattern was left in place for 2 weeks while data were collected. The second pattern [1.22-m (4-ft) stripe with 10.98-m (36-ft) gap] was then placed over the 0.61-m (2-ft) pattern and left in place for 2 additional weeks while data were collected. The final set of data was then collected on a segment of Interstate 3.5 mi (5.6 km) long, approximately 1.6 km (1 mi) upstream of the original study site. This segment exhibited the same curvature, terrain, and cross section elements as the original study site and had also been resurfaced just before the data collection effort, which resulted in approximately the same contrast between the markings and the pavement surface as exhibited on the original test segment. The traffic characteristics (e.g., average speed, vehicle classification percentages, hourly volumes) were almost identical at the two locations. This was expected since one was immediately downstream of the other.

Field Procedures

The field data collection procedure consisted of a data collection van following and videotaping the operations of random cars in the traffic stream along the selected route. Three restrictions were placed on the vehicles selected from the traffic stream for data collection:

1. To maximize sample size, the only vehicle type selected was a passenger car (i.e., no vans, pickups, or trucks).
2. The vehicle selected had to be isolated for at least 70 percent of the segment to eliminate any influences caused by other traffic.
3. The vehicle selected had to remain in the right lane for at least 70 percent of the segment since data were collected for that lane only.

The first requirement was easily determined in the field before beginning each run. The other requirements were not determined until the data collection run was under way or completed. These requirements resulted in some aborted runs in the field and in the elimination of runs during the data reduction task.

At an on-ramp upstream of the roadway segment being used for the study, the data collection team parked and waited for traffic stream vehicles. When a passenger car of interest passed, the team pulled in behind the vehicle and closed to the necessary following distance before reaching the beginning of the study segment. Preliminary information about the vehicle, such as body style, color, and taillight description, was recorded to help match the vehicles on the videotape with the appropriate run number during data reduction.

When the data collection team reached the beginning of the study segment, the distance measuring instrument (DMI), stopwatch, and videocassette recorder were started, and they continued to run through the entire segment. The videotape provided a continuous real-time record of the operations of each vehicle followed as it traversed the roadway and allowed for the acquisition of all MOEs in the office.

For those runs conducted in wet weather, an additional variable was collected, which subjectively gauged the intensity of the rainfall or the road conditions. The two-member data collection team jointly selected one of the following factors during each run: wet road, splash/spray, light rainfall, medium rainfall, or heavy rainfall.

The total number of passenger cars for which data were collected in the above manner was 436. A breakdown of these runs by weather and light condition is given in Table 2. Whereas the goal was to obtain 45 runs for each of the 12 cells, the amount of time allowed between the deployment of the different marking patterns (2 weeks) and the sporadic rainfall limited the number of runs in the wet weather cells for the 0.61-m (2-ft) and 1.22-m (4-ft) patterns.

DATA REDUCTION

Obtaining the operational measures to be analyzed from the collected data consisted of three basic steps: recording lateral placement from the video images, recording encroachments and erratic maneuvers from the videotape, and determining average running speed.

Lateral Placement

Determining the lateral placement of each followed vehicle began with the determination of vehicle width. During the field data collection effort, several measurement points were selected for this purpose. At each of these points, lane widths, shoulder widths, distances to guardrails, and other points of reference were precisely measured. Video images were produced from two of these points for each vehicle. The widths of the vehicle and the lane (or other reference) in the video image were measured, recorded on the data reduction form, and used to determine the actual vehicle width as follows:

$$C = (W/w) \times c$$

where

C = actual car width,
W = actual reference width,
c = measured car width, and
w = measured reference width.

For each vehicle, video images were then produced for each point at which lateral placement was to be measured. A total of eight points were selected within the segment to be representative of the geometric characteristics of the roadway. From each video image, the car width and distance from the centerline to the outside edge of the left rear tire were measured as shown in Figure 2 and recorded on the data reduction form. The actual distance from the centerline was then computed as follows:

$$D = (C/c) \times d$$

where

D = actual distance from the lane line,
C = actual car width,
c = measured car width, and
d = measured distance from the lane line.

Encroachments and Erratic Maneuvers

A second run through the videotape was conducted to record encroachments and erratic maneuvers. An encroachment was defined as occurring when the outside edge of the rear tire

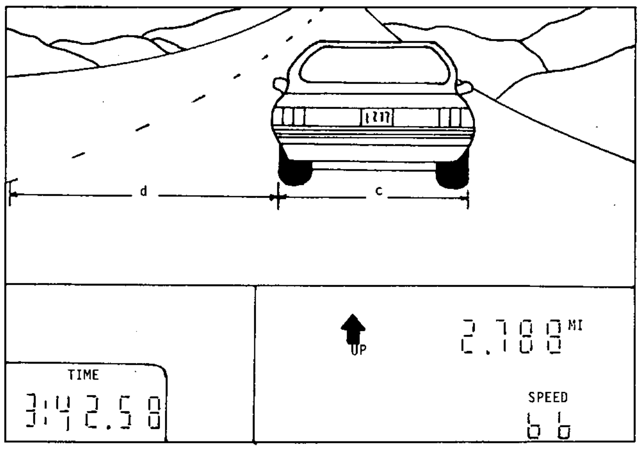


FIGURE 2 Measurements obtained from the video image for computing lateral placement.

TABLE 2 Number of Passenger Cars Followed

Stripe Length	Light Condition	Weather Condition Day	Night
0.61 m	Day	45	15
	Night	45	28
1.22 m	Day	45	20
	Night	37	21
3.05 m	Day	45	45
	Night	45	45

1 m = 3.28 ft

of the vehicle being followed crossed the outside edge of the lane line or edge line. For the temporary marking patterns, there was no edge line present. In those cases, the seam in the pavement served as a surrogate. This seam was consistently 3.66 m (12 ft) from the lane line and would eventually serve as a guide for placing the edge line. For each encroachment observed during a run, beginning and ending mileposts (from the DMI) and times (from the stopwatch) were recorded on the data reduction form along with the type of encroachment (lane line or edge line).

Whereas lateral placement and encroachments serve as objective measures of vehicle performance, erratic maneuvers are more subjective in nature. For purposes of this study, three events were classified as erratic maneuvers: brake applications, sudden speed changes of 8 km/hr (5 mi/hr) or greater, and sudden directional changes. Each event was recorded on a form indicating the type of erratic maneuver and the location where it occurred (DMI reading).

Running Speeds

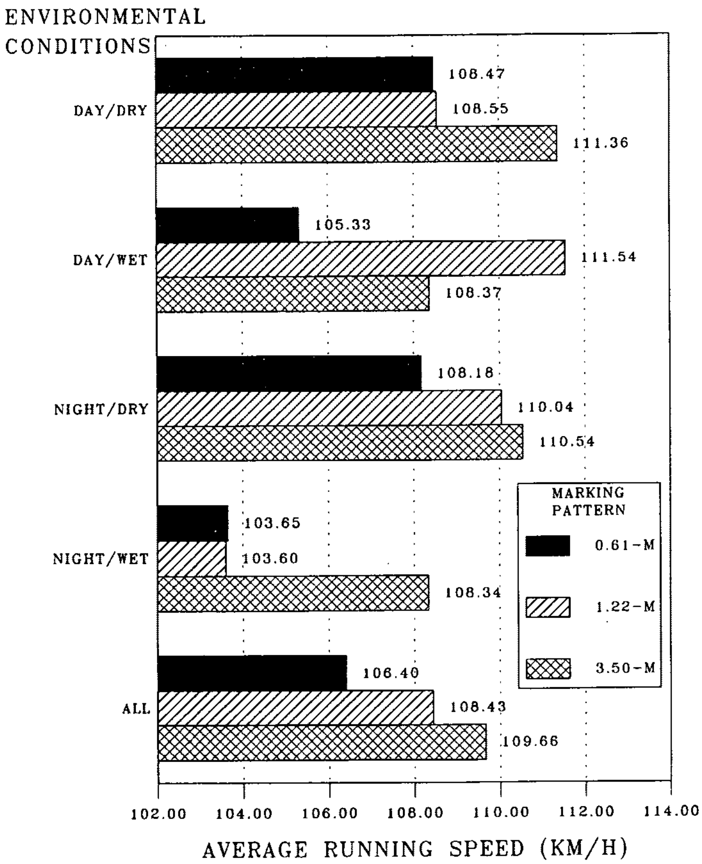
The final MOE obtained from the videotape was the average running speed. At the start of the second run through the tape, the time at which the vehicle entered the test segment (shown on the stopwatch) was recorded. The time at which the vehicle exited the segment was also recorded. Using these values and the known distance between the two points (obtained from the DMI), the average running speed was calculated for each vehicle followed.

DATA ANALYSIS AND RESULTS

The mean values for each of the collected MOEs by pavement marking pattern and environmental condition (time of day/weather) are shown in Figures 3 through 6. The statistical analysis of the data consisted of a number of procedures, including analysis of variance (ANOVA). For all analyses, a 95 percent confidence level was selected to determine whether a class variable (e.g., pavement marking pattern) had a significant effect on an MOE.

The primary issue addressed in this study was, What effect does pavement marking pattern have on driver performance? A summary of the results from the analysis indicated the following:

- There were significant differences between the average running speeds of vehicles when comparing the 3.05-m (10-ft) marking pattern with the 0.61-m (2-ft) pattern. There were no significant differences in speeds when comparing the 1.22-m (4-ft) marking pattern with either the 3.05-m (10-ft) or the



1 km/h = 0.62 mi/h; 1 m = 3.28 ft

FIGURE 3 Average running speeds.

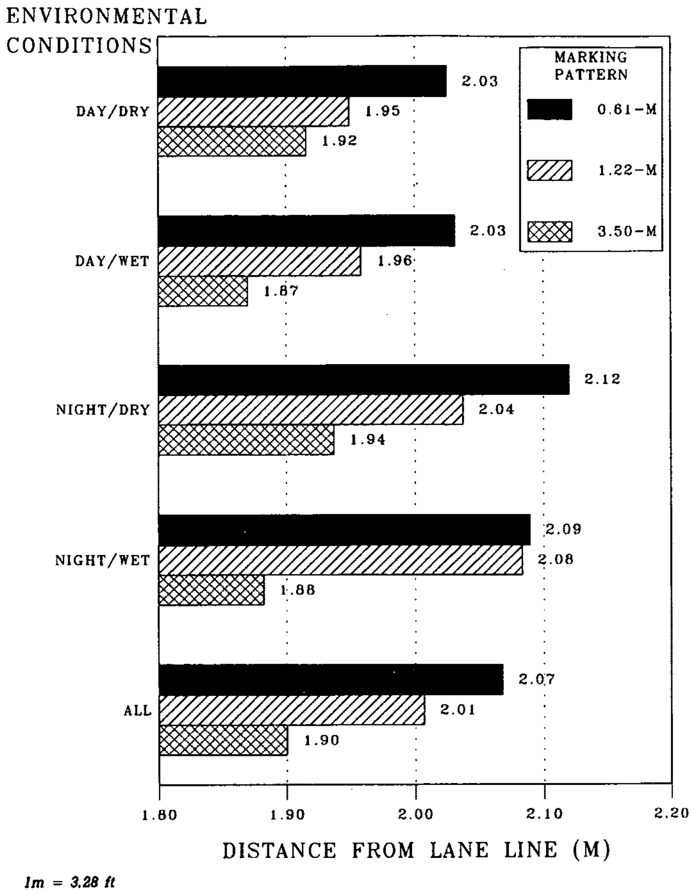


FIGURE 4 Mean distance from the lane line to the center of the vehicle.

0.61-m (2-ft) pattern. Overall, travel speeds were reduced as the length of the marking became shorter (see Figure 3).

- The lateral placement MOE, distance from the lane line to the central axis of the vehicle, proved to be significantly different for the 3.05-m (10-ft) marking pattern compared with either the 0.61-m (2-ft) or 1.22-m (4-ft) pattern. The differences for the 0.61-m versus 1.22-m (2-ft versus 4-ft) patterns, however, were not significantly different. The general trend was for drivers to position their vehicles closer to the center of the lane [i.e., 1.83 m (6 ft) from the lane line] as the length of the marking was increased (see Figure 4).
- Lane placement variance, which served as a measure of a driver's ability to traverse the roadway in a consistent manner, proved to be significantly different for the three marking patterns. The results indicated an increase in lane placement variability as the length of the marking was reduced (see Figure 5).
- The differences between the average number of edge line and lane line encroachments for the three marking patterns were significantly different and revealed that drivers tended to stray out of the travel lane more frequently as the marking was reduced in length (see Figure 6).

In addressing the primary issue above, the data collection and analyses were structured to determine the effects of marking pattern with respect to two secondary issues: day versus night and weather conditions. A summary of the results from the analysis as related to these issues is given below.

What effect does day versus night have on vehicle operations with respect to pavement marking pattern?

- Whereas the data showed speeds to be generally lower at night than during the day, there were no significant differences between the three marking patterns that could be attributed to time of day.
- Drivers positioned their vehicles closer to the center of the lane during the day than at night for all three marking patterns. However, these differences were not statistically significant with respect to the length of the marking.
- The variance in lane placement was significantly greater for night conditions. The results indicated that the difference in variance between day and night for the 3.05-m (10-ft) pattern was relatively small compared with the 0.61-m (2-ft) and 1.22-m (4-ft) patterns. The difference in variance between day and night for the 0.61-m (2-ft) and 1.22-m (4-ft) patterns was relatively the same.
- The number of encroachments per run during the night was significantly different from the number that occurred during the day. The results revealed higher values at night for each marking pattern.

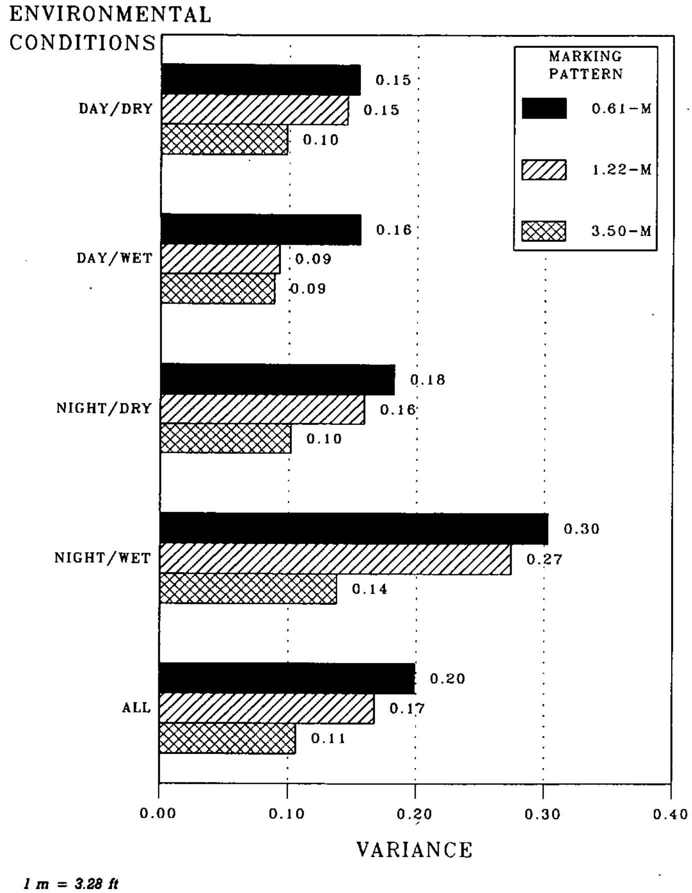


FIGURE 5 Average lateral placement variance.

What effect does adverse weather (i.e., rain and wet road conditions) have on driver performance with respect to pavement marking pattern?

- The effects of weather, specifically rain, on the differences in average running speeds between the three marking patterns were mixed. There were significant differences between the 3.05-m (10-ft) and 0.61-m (2-ft) patterns. There were no significant differences between the 3.05-m (10-ft) and 1.22-m (4-ft) patterns. Finally, the differences in speeds between the 0.61-m (2-ft) and 1.22-m (4-ft) patterns were not significant, but they did reveal some effects that could be attributed to weather conditions. In general, speeds were lower for wet weather conditions than for dry conditions for each marking pattern.
- The effects of weather on differences in lateral placement (i.e., the ability of drivers to center their vehicles in the travel lane) between the three marking patterns were insignificant. There was no consistent pattern for this measure when examining wet and dry conditions, and the actual differences were relatively small.
- The impact of weather on lane placement variance was significant. This confirmed earlier results that showed lane placement variability to increase as the length of the marking decreased and emphasized the impact of rain on this measure.
- The number of encroachments per run was significantly different for dry and wet weather conditions. The data revealed a slight decrease in the number as a result of wet weather conditions.

CONCLUSIONS

For each operational measure examined, the 3.05-m (10-ft) marking pattern generally resulted in better driver performance than either the 0.61-m (2-ft) or 1.22-m (4-ft) pattern. This result is reasonable and was expected, since the 3.05-m (10-ft) pattern consisted not only of longer stripes but also contained edge lines, the standard full complement of markings recommended in the MUTCD.

Comparisons against this scenario provided indications of the differences to be expected when drivers encounter nonstandard markings. For example, on the basis of data in this study, drivers would travel 1.22 km/hr (0.76 mi/hr) slower on a segment with 1.22-m (4-ft) stripes and 3.25 km/hr (2.02 mi/hr) slower on a segment with 0.61-m (2-ft) stripes than they would on the same roadway segment fully marked.

The differences are even more significant when examining encroachments. Drivers are likely to encroach over the lane line or edge line 66 percent more in the presence of a 1.22-

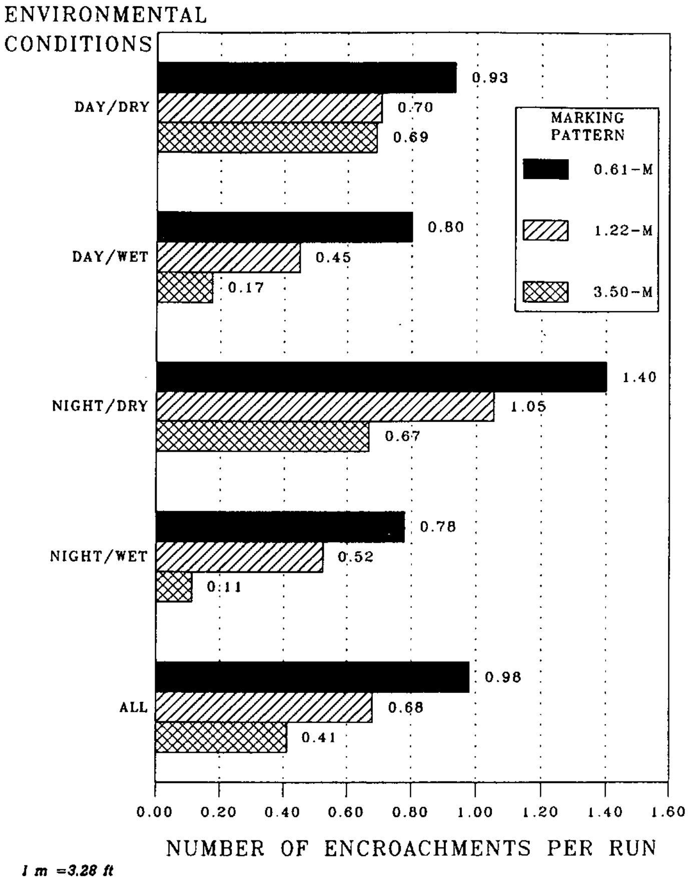


FIGURE 6 Average number of encroachments per run.

m (4-ft) temporary marking and 139 percent more in the presence of a 0.61-m (2-ft) marking than in the presence of the 3.05-m (10-ft) pattern. These values increase dramatically under night and wet weather conditions.

Overall, the results provide evidence of significant decreases in driver performance associated with either of the temporary marking patterns tested. Whereas it may not always be feasible to place full markings on a temporary basis, measures should be taken to prevent reductions in driver performance that result in increased accident potential. Such measures include the use of longer temporary markings and the appropriate use of advance warning signs to indicate a change in the pavement marking pattern.

A comparison of the operational measures for the two temporary marking patterns [0.61-m (2-ft) versus 1.22-m (4-ft)] indicates that there were not a large number of statistical differences, largely because of the small sample size. However, certain trends existed with respect to driver performance:

- The speed at which drivers traveled decreased as the length of the lane line decreased.
- Drivers positioned their vehicles closer to the center of the lane as the length of the lane line increased.
- The variability of vehicle placement within the lane increased as the length of the lane line decreased.
- The number of encroachments increased as the length of the lane line decreased.
- All operational measures were negatively affected by adverse weather conditions.

The cumulative results of these trends indicate that drivers performed better with the 1.22-m (4-ft) stripes than with the 0.61-m (2-ft) stripes. The number of encroachments per run is an operational measure that illustrates the differences in the two marking patterns (see Figure 6). Under dry weather conditions, day and night, the number of encroachments is 33 percent higher for the 0.61-m (2-ft) pattern than for the 1.22-m (4-ft) pattern. This value increases to 50 percent for nighttime/wet weather conditions and 77 percent for daytime/wet weather conditions.

As in any operational study, it is difficult to directly translate differences in operational measures to accident potential, since the link between the two has not been clearly established. However, the operational measures examined in this study provide an indication of a driver's ability to choose and maintain a correct path and speed, thus reducing the risk of being involved in an accident. The differences in operational measures associated with each of the temporary marking patterns show that drivers maintain higher speeds, position their vehicles closer to the center of the lane, have less variance in their lane placement as they traverse the roadway segment, and have fewer encroachments with the 1.22-m (4-ft) stripe than with the the 0.61-m (2-ft) stripe. Combining these results, it becomes apparent that drivers have more confidence with the longer temporary marking, resulting in a safer driving environment.

ACKNOWLEDGMENTS

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Durable Fluorescent Materials for the Work Zone

DAVID M. BURNS, LEE A. PAVELKA, AND ROBERT L. AUSTIN

Construction work zones (CWZs) are a major cause of concern for highway and safety engineers. At any given time CWZs constitute only a small fraction of the total roadway miles. However, they are the site of an increasing number of roadway accidents each year. The traffic safety industry has attempted to respond to this problem by providing brighter retroreflective signing materials to increase the nighttime visibility of the CWZ. The equally important need for increased daytime visibility has prompted traffic engineers to experiment with fluorescent colors in the CWZ. Fluorescent colors have outstanding visibility under all daylight driving conditions. Even so, fluorescent colors have never achieved widespread acceptance in outdoor signing applications. The primary obstacle has been the very poor color stability of fluorescent signing materials. A typical fluorescent roadway sign has lost most, if not all, of its color within 1 year. Often it is only a matter of months or weeks. Recent developments have resulted in a redefinition of the contribution that fluorescent colors can make toward improving traffic safety. A fluorescent retroreflective sheeting with color durability similar to existing CWZ sheetings is now available. A field study was run to compare the visibility performance of this fluorescent retroreflective sheeting with that of conventional fluorescent films and ordinary retroreflective materials. The results of the study indicate that fluorescent retroreflective sheeting provides better daytime and nighttime visibility than do ordinary signing materials.

Safely navigating the national roadway system is becoming more difficult for the average driver. The increasing visual complexity of the driving environment, increasing traffic loads at all hours, and the seemingly endless roadway reconstruction and maintenance activities are putting a higher sensory load on the driver. The motorist's task of screening out the important driving information is becoming increasingly complex. Roadway traffic control devices (signs, etc.) are the primary communication link between the driving environment and the driver. They must effectively provide the essential regulatory and safety information.

Modern traffic control devices convey information using a redundant coding system of symbols, legends, shapes, and colors, where each attribute has an assigned meaning. By combining attributes the driver is presented with a number of visual prompts, all of which convey and reinforce the message. One example is the construction work zone (CWZ) warning sign used in the United States. The orange color (construction zone warning), diamond shape (warning roadway hazard), and legend "LANE ENDS 1500 FEET" (warning) each alert the driver to a possible hazard ahead (*1*). Devices that effectively communicate with the motorist share four basic characteristics: they are conspicuous and legible and the message is easily understood and credible (*2*). Foremost among the requirements is conspicuity. When a sign or other device is not seen because it does not successfully compete for the driver's attention, it makes no difference what the message is. The transportation safety industry has four basic control mechanisms for adjusting the conspicuity of traffic control devices: size, shape, color, and retroreflective brightness, for nighttime visibility. The first attribute identified, both night and day, is color (*3–5*).

Jenkins and Cole (*2*) and Cole and Hughes (*6*) have defined two levels of conspicuity with respect to traffic control devices: (*a*) search conspicuity (the ability "to be quickly and readily located by search") and (*b*) attention conspicuity (the ability "to attract attention when the driver is unaware of its likely occurrence").

Search conspicuity is required of guide signs and informational signs. CWZ, pedestrian crossing, and school zone signs need the higher-level attention conspicuity to perform their function. It is important that these types of warning signs maintain a high level of conspicuity at all times.

As the need for higher-visibility traffic control devices has evolved, researchers have responded with improved materials and new technologies. A review of the literature indicates that the majority of that work has focused on nighttime visibility (retroreflectivity). Improving visibility for the night driver is critically important. National accident statistics indicate that 55 percent of the fatalities occur at night, when only one-third of the drivers are on the road (*7,8*). However, improving daytime conspicuity is also very important. The vast majority of all traffic accidents, including the remaining 45 percent of traffic fatalities, occur during daylight, when retroreflective performance plays no role. Of particular importance is the need to improve the daytime visibility of traffic control devices under low light (dawn and dusk) and adverse weather conditions.

A number of visibility studies conducted in the 1960s and early 1970s demonstrated the daytime superiority of fluorescent colors. Siegel and Federman (*9*) conducted laboratory and field studies on the detection and color recognition of fluorescent orange paints relative to ordinary colors. The first traffic study was a field study run by Hanson and Dickson (*10*) in which they compared a number of fluorescent and ordinary colors under typical daylight driving conditions. Asper (*11*) looked at the conspicuity of fluorescent slow-moving vehicle emblems. All of these studies concluded that fluorescent colors are significantly more visible (as determined by larger detection and recognition distances) than ordinary colors. Fluorescent colors consistently outperformed the ordinary colors over the entire range of daylight driving conditions—sunny,

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overcast, dawn, and dusk. In part on the basis of these studies, fluorescent colors are specified in a number of transportation and workplace safety signing applications (*12–14*).

Fluorescent orange is the best documented of the fluorescent colors. Its high conspicuity and exceptional warning value are unquestioned. The *Manual on Uniform Traffic Control Devices* (MUTCD) equates the "high conspicuity" of fluorescent orange directly to "an additional margin of safety" for the motorist (*1*). Fluorescent retroreflective sheetings have been known for 20 years (*15*), but they have not yet achieved widespread acceptance in outdoor signing applications. The primary reason has been the historically poor color stability of fluorescent signing materials. The complex photochemistry of fluorescence accelerates the degrading effects of solar radiation. As a result, the outdoor color durability of fluorescent sheetings has typically been an order of magnitude lower than ordinary signing colors. According to the International Commission on Illumination (CIE), changes in the color (hue) and luminance (brightness) of conventional fluorescent materials "may be readily apparent in even a few days" (*16*).

For signing applications in which the requirement for high visibility has outweighed color durability concerns, exceptionally large color limits are specified to allow for fluorescent color degradation over time. The region of CIE color space defining unrestricted fluorescent orange (*14*), a safety color for long-term outdoor use, is at least twice the size of the region allotted for orange retroreflective sheeting (ASTM D 4956-90). This type of approach attempts to balance the visibility benefits of fluorescent colors against the economics of sign replacement. Allowing even for considerable changes in color and loss of fluorescent emission intensity, fluorescent colors, and fluorescent orange in particular, typically last no more than 2 years outdoors. Specifying very large in-use color limits to compensate for marginal color durability may be practical for some color-coded systems, but it is not practical for roadway signing.

Until now only ordinary colorants systems have possessed the color durability required for retroreflective sheetings used in the CWZ. Recent technical developments are now leading to a redefinition of the contribution fluorescent colors can make toward improving the visibility of traffic control devices. A highly retroreflective fluorescent orange sheeting has been developed having color durability similar to ordinary traffic signing materials. The field study reported here was run to compare the visibility performance of the durable fluorescent orange retroreflective sheeting with both conventional fluorescent films and the ordinary orange retroreflective materials commonly used for CWZ signing. Visibility measurements were made at night using illumination from low-beam automobile headlights and under a range of typical daylight driving conditions. New and weathered samples of the fluorescent orange retroreflective sheeting were included in this study to evaluate the effect of long-term outdoor exposure on visibility performance.

FIELD STUDY: METHODOLOGY

Subjects

The study used a group of 14 adult licensed drivers (10 males and 4 females). All subjects were within normal limits for visual acuity and color vision when tested on a Bausch and Lomb Orthorater.

Site

The study was conducted during spring and late summer 1991 at the 3M Transportation Safety Center in Cottage Grove, Minnesota. This facility is a full scale mock-up of a U.S. Interstate highway. The viewings were held on a flat straightaway running north by northwest.

Driving Conditions

The daytime viewings were conducted under a wide range of natural daylight conditions. The daylight conditions (defined below) covered the range of ambient light intensities and spectral distributions typically encountered while driving:

1. Sunny—midday (11 a.m. to 2 p.m.) under a sunny clear sky,
2. Overcast—midday (11 a.m. to 2 p.m.) under heavy cloud cover, and
3. Civil twilight—the 30-min period immediately after sunset (all observations in this study were made under a clear sky).

The nighttime segment of the study was run under the illumination of low-beam automobile headlights. Nighttime viewings began a minimum of 1.5 hr after sunset.

Backgrounds/Targets

A description of the materials used as targets is provided in Table 1. The daytime chromaticity coordinates, coefficients of retroreflection (R_A), and (where appropriate) maximum spectral radiance factors were determined for each sample and background in accordance with established practices (*17*; ASTM D 4956-90; ASTM E 991-90). Color was measured on a HunterLab Labscan 6000 spectrophotometer equipped with circumferential viewing using illuminant D_{65} and the 2-degree CIE standard observer. Table 2 gives the color and R_A data. Two backgrounds were used: white and olive drab (OD). These were chosen to represent the range of background brightness and color encountered in nature. White is representative of highly reflective environments such as concrete buildings and snow, whereas the dark green OD represents the natural foliage of trees, shrubs, grass, and fields.

Nine samples were used in the study—one red, one white, and seven orange. The three ordinary orange materials (Samples 1, 2, and 3) cover the range of retroreflective brightness available for CWZ signing materials. Two of the fluorescent materials (Samples 3A and 3B) were retroreflective sheetings, and the other two (Samples C and D) were nonretroreflective films. Figure 1 shows a plot of the seven orange samples relative to the color limits for yellow and orange retroreflective sheetings set out in ASTM D 4956. Samples 3A and 3B were identical in construction, as were Samples C and D. The differences within each pair are the result of outdoor weathering. Samples 3A and C are unexposed samples. Samples

TABLE 1 Sample Identification and Construction

Samples	Color	Fluorescent	Retroreflective	Type of Retroreflector	Sample Condition
		Yes / No	Yes / No		New / Weathered
3A	Orange	Yes	Yes	Microprismatic	New
3B	Orange	Yes	Yes	Microprismatic	Weathered
C	Orange	Yes	No		New
D	Yellow	Yes[1]	No		Weathered
3	Orange	No	Yes	Microprismatic	New
2	Orange	No	Yes	Encapsulated Lens	New
1	Orange	No	Yes	Enclosed Lens	New
E	Red	No	No		New
F	White	No	No		New

[1] Initially this sample was fluorescent orange.

TABLE 2 Color/Coefficient of Retroreflection

Sample	CIE 1931 Chromaticity Coordinates[1] Y	x	y	Maximum Spectral Radiance Factor %	nm	Coefficient of Retroreflection (R_A)[2] cd/lux/m^2
3A	34.4	0.611	0.377	112.1	610	443
3B	26.5	0.593	0.385	76.3	610	451
C	44.4	0.642	0.351	205.6	610	0
D	55.2	0.449	0.461			0
3	15.5	0.603	0.389			647
2	15.5	0.543	0.401			119
1	24.3	0.575	0.398			35
E	6.8	0.600	0.317			0
F	80.9	0.312	0.331			1
Background						
White	80.9	0.312	0.331			
OD	9.2	0.331	0.364			

[1] Illuminant D65, CIE 2° standard observer
[2] R_A measured at 0.2° observation angle, beta 1 = −4, and beta 2 = 0

3B and D had been weathered outdoors for 1 year in Arizona (inclined 45 degrees from horizontal, south facing) before this field study. This type of exposure is commonly used to assess the long-term outdoor durability of signing materials. Ketola (*18*) has shown that weathering 1 year at 45 degrees is roughly equal to 2 years of vertical exposure, which is more typical of highway signing. The combination of high solar irradiance levels and hot temperatures encountered in Arizona represents a worst-case environment for fluorescent color materials.

Figure 1 shows that the nonretroreflective fluorescent film underwent a substantial color change during exposure. Sample D started out a highly saturated fluorescent orange color identical to Sample C, but after weathering it had faded to a washed-out nonfluorescent yellow. The fluorescent orange retroreflective sheeting experienced only a modest color change during the same exposure. The color difference between the weathered fluorescent retroreflective sheeting (Sample 3B) and the unexposed sample (3A) is due primarily to a decrease in the fluorescent emission intensity as indicated by a lower maximum spectral radiance factor. There was no significant change in the coefficient of retroreflection of Sample 3B compared with Sample 3A. The radical color change that occurred in Sample D during outdoor exposure rendered it unfit for

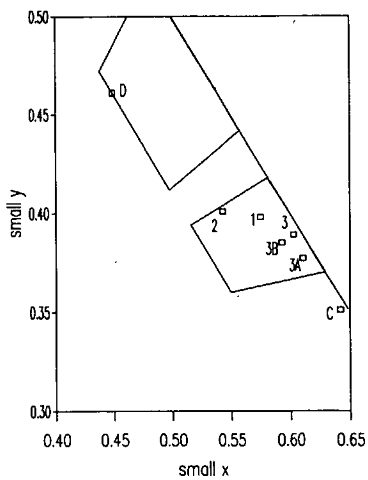

FIGURE 1 CIE 1931 chromaticity plot relative to ASTM D 4956 color limits for orange and yellow retroreflective sheetings.

CWZ signing applications in accordance with the MUTCD. For this reason Sample D was not included in the field study. All of the remaining samples used in the study were unweathered materials. The red and white targets were nonretroreflective films. Null points (white on white, OD on OD) were also run.

Protocol

The field study was conducted in a manner similar to that of Hanson and Dickson (*10*). Nine circular targets each 9.29 cm^2 (0.01 ft^2) in area were mounted in a random order onto background panels 1.22 by 1.22 m (4 by 4 ft) (Figure 2). These panels were mounted in the middle of the driving lane with the center of the panel 1.83 m (6 ft) off the ground. The subjects in automobiles drove toward the panel starting from a distance of 457 m (1,500 ft). Under daylight illumination no targets were detectible from this distance. As they approached the panel at dead slow (about 3.2 kph or 2 mph) the subjects recorded the following for each target position:

1. Detection distance—the distance at which the target can be differentiated from the background with certainty;
2. Recognition distance—the distance at which the target color (hue) can be identified; and
3. Color—the color (hue) the target was identified as possessing.

The subjects recorded their own data during each run using a data form consisting of a 3 by 3 grid with each cell corresponding to a specific target position. Subjects determined distances by referring to markings located every 3.05 m (10 ft) along the side of the road indicating the remaining distance to the background panel. Subjects were instructed to identify the target color (hue) with a single one-word name (white, green, red, etc.). Color names were not preassigned, leaving the subjects free to identify each target on the basis of their own color-naming criteria. Two practice runs were made before starting the experiment to train the subjects in how to fill out the data collection forms. Approximately equal numbers of approaches were made north- and southbound for the daylight conditions. At night where ambient illumination was independent of direction of travel, the approaches were made from only one direction for each background. The results of this study are based on 1,761 individual observations.

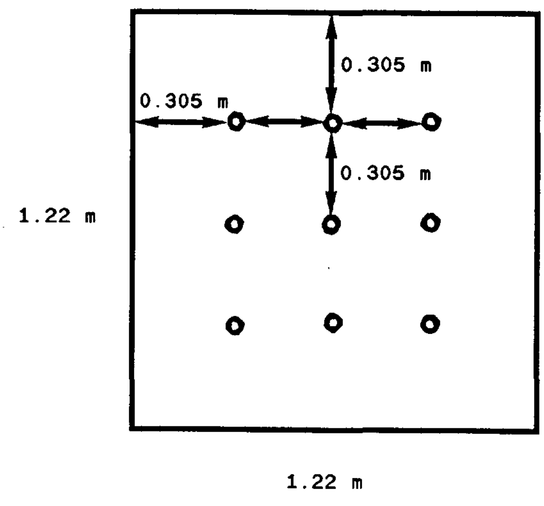

FIGURE 2 Background panel and target positions.

FIELD STUDY RESULTS

Detection and Recognition

The 85th percentile detection and recognition distance results are summarized in Table 3. Under a given set of conditions the 85th percentile distance represents the distance by which time at least 85 percent of the subjects had already detected/recognized the target. Traffic engineers commonly use the 85th percentile performance as the minimum design limits for traffic control devices. The values were interpolated from the probability-distance distributions for each condition and background. Analysis of variance (ANOVA) and two-way *T*-tests were run for each background and lighting condition. The results presented are all supported at the 95 percent confidence level by the statistical analysis.

Natural Illumination (Daytime)

Daytime Detection There were no substantial differences in the detection distances of any of the targets when viewed against the white background. Within each illumination condition the targets were all detected over the same range of distances, as shown in Figure 3. Against the dark OD background there was significant differentiation among the samples. The detection distances of the fluorescent materials were significantly larger than those of the ordinary orange samples. For example, Figure 4 compares the detection probability-distance curves for the two fluorescent orange retroreflective samples with the those of the two ordinary orange samples with comparable coefficients of retroreflection. The figure shows that under the poorest daytime visibility condition—civil twilight—the 85th percentile detection distances of the fluorescent retroreflective sheetings were 40 to 70 percent greater than the comparable ordinary orange sheetings. Under the best overall visibility condition—sunny—the two fluorescent orange sheetings still outperformed the ordinary materials by 20 to 40 percent. These results suggest that as the daytime lighting conditions deteriorate the fluorescent sheetings' visibility advantage increases relative to the ordinary signing materials.

Daytime Recognition Overall, the recognition distances of the fluorescent materials were much greater than those of the ordinary colors. The fluorescent retroreflective sheetings were recognized at significantly greater distances than the ordinary colors in almost every case. The new and weathered fluorescent orange retroreflective sheetings (Samples 3A and 3B) had equivalent recognition performance under nearly all daylight viewing conditions. One exception was the overcast/OD condition, where the 85th percentile recognition distance of the weathered sheeting was only 81 percent of the new sam-

TABLE 3 Eighty-Fifth Percentile Detection and Recognition Distances Versus Background

	Detection Distance (meters)[1]							
	Sunny		Overcast		Civil Twilight		Night	
Sample	White	OD	White	OD	White	OD	White	OD
3A	160	230	200	259	21	111	433	415
3B	223	262	209	230	37	93	442	437
C	195	344	187	270	43	116	27	37
3	191	187	216	178	43	64	442	442
2	209	154	216	166	43	66	277	276
1	212	241	215	183	43	98	154	142
E	245	105	200	87	55	14	30	11
F	--[2]	273	--	332	--	114	--	46

	Recognition Distance (meters)							
	Sunny		Overcast		Civil Twilight		Night	
Sample	White	OD	White	OD	White	OD	White	OD
3A	114	175	126	186	21	69	262	344
3B	94	186	96	151	18	64	244	218
C	134	201	133	206	24	84	15	2
3	93	143	34	99	15	23	285	218
2	27	117	43	91	3	23	180	168
1	72	139	66	122	6	43	91	72
E	14	52	27	73	3	8	3	0
F	--	78	--	44	--	38	--	0

[1] 1 m = 3.28 ft
[2] White on White null points.

ple's (Figure 5). Yet under those same conditions the weathered fluorescent sheeting was still recognized at distances up to 165 percent of the ordinary orange recognition distances.

Artificial Illumination (Nighttime—Low-Beam Headlights)

Nighttime Detection Not surprisingly, the nighttime detection distances using low-beam headlight illumination scaled directly with the coefficient of retroreflection and was independent of background color. Even the least bright retroreflective sheeting (Sample 1) was detected 3 to 4 times further away than the nonretroreflective films (Figure 6). No significant differences were found in the detection ranges of the three microprismatic sheetings (Samples 3, 3A, and 3B). This may be an artifact of the limited maximum viewing distance of 457 m (1,500 ft) used in the study.

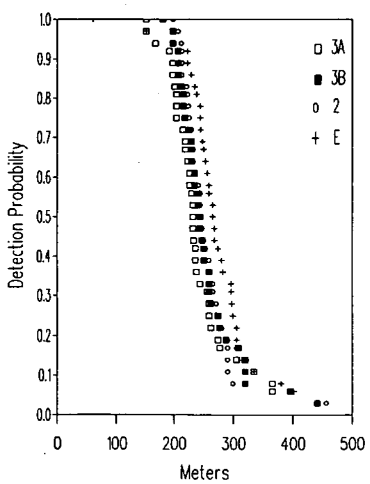

FIGURE 3 Daytime detection distance probability distribution of representative samples: overcast/ white background.

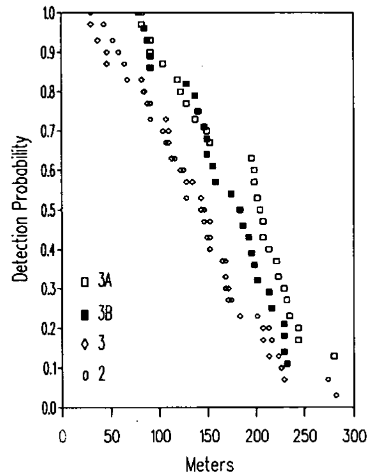

FIGURE 4 Daytime detection distance probability distribution of fluorescent and ordinary orange samples: civil twilight/OD background.

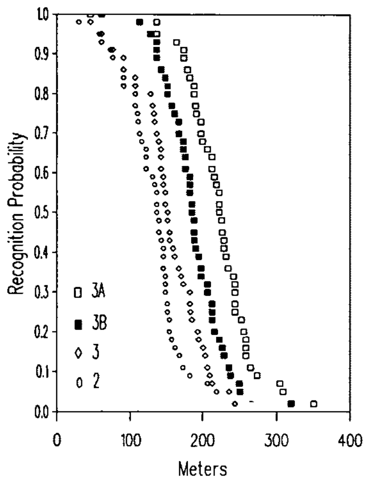

FIGURE 5 Daytime recognition distance probability distribution of fluorescent and ordinary orange samples: overcast/OD background.

Nighttime Recognition The nighttime recognition results mirror the detection results in most respects. The distances scaled with coefficient of retroreflection (Figure 7) and were generally independent of background color. Again, no significant differences were found between the three microprismatic sheetings. The recognition ranges for the microprismatic sheetings were significantly longer than for the enclosed lens or encapsulated lens sheetings. The nonretroreflective fluorescent orange and red films had extremely short recognition ranges. The subjects did not recognize the color of those films until they were practically on top of them.

Color Identification

The color identification results given in Table 4 are presented in terms of correct color recognition as a percentage of total presentations at each viewing condition. The correct color name for each sample was defined according to available industry specifications. The color of Sample C was defined in accordance with the ANSI Z535.1 color limits for unrestricted fluorescent orange (*14*). All the other samples were defined

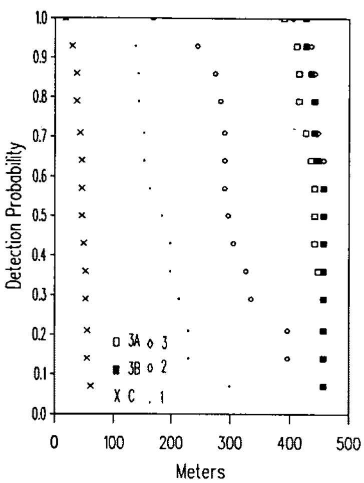

FIGURE 6 Nighttime detection distance probability distribution of samples: OD background.

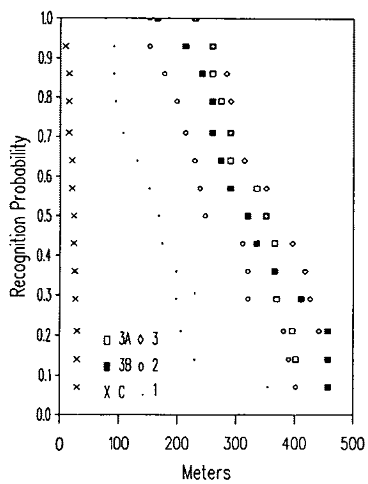

FIGURE 7 Nighttime recognition distance probability distribution of samples: white background.

TABLE 4 Color Identification Versus Background

		Correct Identification (%)							
		Sunny		Overcast		Civil Twilight		Night	
Sample	Correct Name	White	OD	White	OD	White	OD	White	OD
3A	Orange	100	95	86	95	95	100	93	93
3B	Orange	100	95	86	98	90	93	93	93
C	Orange	100	100	92	95	100	97	36	50
3	Orange	94	91	57	89	58	83	100	79
2	Orange	81	86	42	89	45	73	86	79
1	Orange	94	91	64	94	70	95	93	71
E	Red	88	95	64	100	50	97	14	57
F	White	--[1]	86	--	95	--	93	--	50

[1] White on White null points.

according to the daytime color limits set out in ASTM D 4956 for retroreflective sheeting used for traffic control devices. Under natural daylight the fluorescent orange samples were identified correctly a much higher percentage of the time and with greater consistency than the ordinary colors. The fluorescent retroreflective sheetings maintained their identifiability under poor visibility conditions, whereas the ordinary colors lost ground. Under nighttime driving conditions the retroreflective materials—fluorescent and ordinary—were typically identified correctly more than 80 percent of the time, whereas the nonretroreflective colors were correctly identified only 50 percent of the time.

DISCUSSION OF RESULTS

In 1963 the first field study to examine the visibility of fluorescent colors under typical driving conditions was conducted. That study compared the visibility of fluorescent and ordinary color nonretroreflective films under natural daylight. Retroreflective sheetings were known, but fluorescent retroreflective materials were not available. Now, almost 30 years later, this study has examined the effects of combining a fluorescent color and a high-efficiency retroreflector into a single signing material.

The ordinary orange retroreflectors each represent significant improvements in the nighttime visibility of traffic control devices. The all-weather performance of enclosed lens sheeting resulted in improved roadway safety when it was first introduced. Encapsulated lens sheeting constituted another major advance in nighttime visibility performance with a significant increase in the average nighttime visibility distance, but at the expense of some daytime conspicuity. The most recent advance in retroreflective performance is high-brightness microprismatic sheetings. At night these materials are visible at substantially greater distances than even the high-intensity encapsulated lens sheetings. In addition the microprismatic's highly saturated colors provide improved recognition day and night.

If this study only compared the performance of unweathered fluorescent sheeting with that of ordinary highway signing materials, the results would be interesting but would not have indicated the potential of durable fluorescent signing. Retroreflective fluorescent materials that are highly visible when brand new have been available for years. Limited color durability has made their use in conventional roadway signing applications impractical. When a bright fluorescent orange CWZ warning sign fades over the course of a few months to a dull yellow, it has lost almost all its warning value. Continually replacing these conventional fluorescent signs is uneconomical. However, not replacing the signs can lead to potentially dangerous situations since the reason for using fluorescent signing in the first place is to improve roadway safety by increasing the CWZ's visibility. The technology is now available to produce fluorescent colors durable enough for highway signing. After the equivalent of several years continuous outdoor exposure, the fluorescent orange retroreflective sheeting evaluated in this study still outperforms new samples of ordinary orange signing materials. Weathered fluorescent sheeting was included in this study to redefine the state of the art in retroreflective traffic control materials.

Fluorescent signing has finally developed to the point where it can substantially contribute to improving the overall visibility of traffic control devices. Because of developments in fluorescence technology it is now practical to optimize both the daytime and nighttime visibility of traffic control devices. This study reconfirms the role that retroreflective performance plays in determining nighttime visibility. It also demonstrates the daytime advantage of fluorescent signing, particularly under poor visibility driving conditions such as dawn, dusk, and adverse weather. The combination of durable fluorescent color with a highly retroreflective construction has produced a signing material with superior long-term visibility at all times, day and night, and in all types of weather.

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Correlation of the Nighttime Visibility of Pavement Marking Tapes with Photometric Measurement

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A primary visual guide for a motor vehicle driver is the use of pavement markings on the centerline and edge line of the roadway. The nighttime visibility performance of these markings is predicted by a surrogate method of laboratory or field photometric measurement. There are currently several photometric systems in use that vary widely in geometric and precision capabilities. With the advent of modern pavement markings with a variety of retroreflective optical systems and surface characteristics, common retroreflective measurements in the laboratory and field have generally been found to lack correlation with the markings visibility performance of drivers. The nighttime visibility of new, dry centerline pavement markings as viewed from a stationary automobile and semitruck are compared with laboratory and field photometric measurements. The visibility results are compared with the photometric methods. The common test geometries used in the industry today are found to have poor correlation with driver visual perception at most distances. A laboratory test method has been developed with the hope of better characterizing actual pavement marking retroreflective performance. This test method measures products at the same photometric geometries at which a driver actually observes pavement markings. Excellent agreement between driver visual observation and this test method was obtained at multiple distances.

It has long been desirable to predict the brightness of pavement markings as seen by drivers with laboratory and field photometric measurements. With the miniaturization of electronics in the 1960s it became possible to manufacture portable retroreflectometers with optics approaching actual car/driver/road geometries for distances approaching 100 m (*1*). In practice, both entrance and observation angles needed to be increased to make these portable instruments durable and portable. Increasing these angles in comparison with actual road geometries was found to give acceptable results and proved useful for determining threshold values for retroreflectivity within narrow ranges of pavement marking product construction (i.e., new and worn paints and flat preformed pavement markings with 1.5 refractive index glass beads) (*2*). With the advent of modern pavement marking systems with greatly different product constructions, it has been our experience that these portable devices and common test methods do not correlate well with human perception.

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The purpose of this study was to compare pavement marking materials differing greatly in their retroreflective performance with human observers, standard retroreflective test methods, and new laboratory test methods incorporating actual road geometries. It was hoped that human perception of pavement marking performance could be predicted by laboratory and field photometric measurements.

HUMAN PERCEPTION OF PAVEMENT MARKING BRIGHTNESS

Night viewings were held on two consecutive nights in a dark rural setting. The first night 12 observers viewed pavement markings from a Ford Taurus. All 12 observers held a valid driver's license, and none had any visual problems when questioned. The test began at 9:30 p.m. under cloudy skies and a full moon. The test concluded at 3:30 a.m. with partly cloudy skies. The Taurus's headlights were aligned the date of the test. Two subjects viewed the markings at a time. The first subject sat in the driver's seat, and the second sat directly behind with his head next to the driver's. Both viewers' eye positions were recorded relative to the dimensions of the car. Five of the 12 viewers were female. Three were over 50 years old, and three viewers were under 30. Eight of the observers wore eye glasses.

The second night of viewing was performed using a Mack semitruck. Ten subjects from the car viewing and two alternates were chosen as subjects. The test began at 9:30 p.m. with clear skies and a full moon. Two subjects viewed the samples at a time. Both subjects shared the driver's seat. Eye position measurements were taken for each subject.

Pavement marking samples 10 cm wide by 3.05 m long were viewed, two at a time. Each sample of pavement marking was applied to 0.2-cm aluminum panels of the same dimension. The two samples were viewed 10 cm apart on top of a 0.3-m-wide by 3.2-m-long viewing table with a black colored matte surface finish. The table stood 3.8 cm above the road surface. The function of the table was to keep both samples optically flat and level.

The samples were viewed at distances of 12.2, 24.4, 48.8, and 73.2 m; distances are measured from the leading edge of the stripe to the headlight. The maximum distance was chosen such that one of the markings would not be visible to some of the subjects. The samples were viewed as centerline markings. The middle of the viewing table was centered 3.7 m from the right edge of the road. Both the car and the truck were parked between the centerline and edge line. All other road markings were obliterated within the test distances and for 50 m beyond the furthest samples.

Four distinctly different commercially available white preformed pavement marking products were tested. The four products were chosen because of their different retroreflective characteristics and because they represented the range of retroreflectivity available with modern pavement marking systems. The products differed in the type of glass beads, the surface texture of the markings, and the use of a double focusing lens system. Two of the four products were duplicated for controls. Thus a total of six markings were used.

The markings were viewed statically in random pairs. Fourteen random pairings of the six lines at each location were viewed. Each distance had its own random order between the 14 pairs. The 14 random pairs viewed at each distance were selected so that it could be determined whether there was a difference between the left- and right-hand sides of the viewing table. Also, two samples at each location were displayed with their own replicates to check that the subjects rated them as equal.

The pairs were installed on the top of the viewing table and covered with a black felt cloth. The viewers then turned on the vehicle headlights and the samples were exposed for 2 sec. Each subject was allowed to write down the response while a new pair was installed.

After viewing the paired samples for 2 sec each subject was asked to write down whether the samples were equal in brightness, the right sample was brighter or much brighter than the left, or vice versa. If only one line was visible the subjects were asked to write that down. To analyze the data, a numerical weight rating of 0, ± 1, ± 2 was assigned to the ratings of equal, brighter, or much brighter, respectively. Negative numbers were used to designate the left sample being brighter; positive numbers represented the right sample being brighter.

The left and right samples and their numerical rating were entered into a data base along with personal information for each subject such as sex, age, whether the subject wore glasses, eye position information, and which group number the subject was in during the night. The data base could then be searched for any criteria chosen.

The data for each vehicle were analyzed to determine whether there was any difference between the left and right sample positions. For both vehicles it was determined that the maximum difference in rating values between the left and right sample positions was 0.5. Typically the difference between sample positions was less than 0.25. A similar analysis was performed to determine whether the replicate samples were the same. Again replicate samples always measured less than a 0.5 rating difference, typically less than 0.25. Rating values less than 0.25 to 0.5 have little difference.

The data were then analyzed for all subjects in each vehicle to determine visual differences between the four products. For this analysis the replicate samples for the products were considered to be the same material. Also, the left and right sample positions were considered to be equivalent. The four materials were ranked at each distance according to their paired differences. Table 1 gives the ranking of Products A, B, C, and D; a rank of 1 indicates the highest and 4 the lowest perceived brightness. The number in parentheses next to the product represents the average paired ranking difference versus the product of next lower ranking.

There are several important points regarding the rankings of the materials in Table 1. First, except at 73 m, there were differences in the relative rankings of materials when viewed between the car and truck. Product C ranks brighter than Product A in the truck at the two closest distances. In the car, Product A was brighter than Product C at the closer distances. Most important, depending on the distance the products were viewed in each vehicle, the relative rankings of the materials changed. The exception was Product B, which at all distances had the lowest perceived brightness. Each of the other three products changed its relative rankings.

Stated another way, the relative ranking of product brightness as seen by observers is a function of the distance the products are observed at and the type of vehicle being used. The implications of this are great. There is no single retroreflectivity measurement that can predict the relative performance of different pavement marking products at all distances used by a driver.

During the night viewing at 73 m in the Taurus, when Products B and C were viewed with Products A and D, 10 percent of the time the subjects only saw the brighter materials (A and D). This implies that the brightness of Products B and C is approaching a threshold value for automobile drivers at 73 m. In the semitruck all lines were visible at all distances.

TABLE 1 Product Rankings and Their Paired Differences

Vehicle	Distance	#1 Rank	#2 Rank	#3 Rank	#4 Rank
Taurus	12.2m	D (0.7)	A (0.2)	C (1.5)	B (0)
Taurus	24.4m	D (0.3)	A (1.1)	C (0.9)	B (0)
Taurus	48.8m	A (1.0)	D (1.7)	C (0.5)	B (0)
Taurus	73.2m	A (2.0)	D (1.2)	C (0.3)	B (0)
Truck	12.2m	D (0.0)	C (1.4)	A (0.7)	B (0)
Truck	24.4m	D (0.6)	C (0.2)	A (1.5)	B (0)
Truck	48.8m	D (0.2)	A (1.2)	C (1.2)	B (0)
Truck	73.2m	A (0.8)	D (1.4)	C (0.4)	B (0)

DEVELOPMENT OF LABORATORY RETROREFLECTIVITY TEST METHODS

The second aspect of this study was the development of laboratory retroreflectivity test methods to measure the coefficient of retroreflected luminance (specific luminance, or R_L) (ASTM D 4061-89) of pavement markings using the actual optical geometries from the night viewing.

As previously mentioned, the driver's eye height and distance behind the headlight were measured for each observer in the car and truck. The height of the center of the headlight above the ground was 66 cm for the Taurus and 114 cm for the Mack Truck. The eye height above the center of the headlight averaged 47 cm ± 2.8 cm for the Taurus and 112 cm ± 2.8 cm for the truck. The displacement of the eye behind the headlight averaged 225 cm ± 4.6 cm for the Taurus and 92 cm ± 2.0 cm for the truck.

These vehicle dimensions were used to calculate the photometric geometries during the night viewing. Table 2 gives the calculated angles using intrinsic geometry (ASTM E 808-91) to the center of each stripe for both vehicles.

The right-hand column of Table 2 gives the illuminance of each vehicle's headlight at each distance. This is the intensity of light striking an object from the vehicle headlight at that distance. A photometric range was used to make the measurements (*3*).

Measurements of R_L (coefficient of retroreflected luminance) were then made at the angles of Table 2 using a photometric range. The results for both vehicles are presented in Table 3.

The measurements of R_L in Table 3 were then multiplied by the illuminance of the headlight at each geometry in Table 2. The data for each headlight were then added together to obtain the total luminance (L) of each marking as seen by the driver. The results are given in Table 4.

Table 4 indicates the same changes in relative rankings as a function of distance as the night viewing data in Table 1. Only at 12 m in the car did the luminance measurements fail to order the materials as seen by the night viewing subjects, yet even here agreement is reasonable. It is very likely that because 12 m is such a short distance to view pavement marking materials, factors other than R_L may be influencing the driver's perception. It is also possible that because the luminance is so high at close observation distances, the eye is essentially desensitized and unable to detect differences in retroreflectivity.

Despite the truck's being disadvantaged in terms of observation angle, at large distances the observers in the truck had equal or higher retroreflected light available. This was shown by the fact that no subject viewed only one of the test lines at any distance. The reason for this appeared to be related to the intensity of the headlights on the truck.

As mentioned previously, several of the subjects were unable to view Products B and C at 73 m in the car. Using Table 4 it appears that a luminance value of 50 to 60 mcd/m² is close to a threshold value for pavement marking visibility.

TABLE 2 Intrinsic Angles for a Ford Taurus and a Mack Truck

INTRINSIC ANGLES FOR A FORD TAURUS						
Distance (meters)	Headlight	Observation (Deg)	Entrance (Deg)	Presentation (Deg)	Orientation (Deg)	Headlight Illumination (Lux)
12.2	Left	1.33	87.47	-4.44	-174.9	4.60
12.2	Right	5.07	87.50	74.36	-169.9	4.07
24.3	Left	0.87	88.66	-16.33	-177.3	0.873
24.4	Right	2.58	88.66	70.57	-174.6	1.02
48.8	Left	0.52	89.30	-21.36	-178.6	0.185
48.8	Right	1.29	89.31	67.90	-177.2	0.211
73.2	Left	0.37	89.53	-22.91	-179.1	0.0874
73.2	Right	0.86	89.53	66.87	-178.1	0.0771
INTRINSIC ANGLES FOR A MACK TRUCK						
12.2	Left	4.05	85.46	3.84	-175.6	1.42
12.2	Right	7.81	85.53	57.94	-169.2	0.969
24.4	Left	2.30	87.59	1.97	-177.7	2.17
24.4	Right	4.21	87.6	56.55	-174.3	1.83
48.8	Left	1.23	88.76	1.00	-178.8	0.379
48.8	Right	2.18	88.76	55.57	-177.0	0.384
73.2	Left	0.84	89.16	0.67	-179.2	0.152
73.2	Right	1.47	89.16	55.19	-178.0	0.123

TABLE 3 Coefficient of Retroreflected Luminance ($mcd/m^2/lx$)

			Preformed Tape Sample			
Vehicle	Distance	Headlight	A	B	C	D
Taurus	12.2m	Left	784	364	997	908
Taurus	12.2m	Right	193	244	257	341
Taurus	24.4m	Left	796	347	665	755
Taurus	24.4m	Right	357	305	334	571
Taurus	48.8m	Left	1080	308	412	619
Taurus	48.8m	Right	736	371	379	728
Taurus	73.2m	Left	1330	269	308	516
Taurus	73.2m	Right	1100	422	418	791
Truck	12.2m	Left	270	189	450	393
Truck	12.2m	Right	117	153	184	190
Truck	24.4m	Left	477	245	584	558
Truck	24.4m	Right	217	214	294	337
Truck	48.8m	Left	700	292	572	640
Truck	48.8m	Right	399	271	373	542
Truck	73.2m	Left	868	310	518	630
Truck	73.2m	Right	596	313	380	606

TABLE 4 Luminance of Pavement Marking Samples (mcd/m^2)

		Preformed Tape Sample			
Vehicle	Distance	A	B	C	D
Taurus	12.2m	4390	2670	5640	5570
Taurus	24.4m	1060	615	921	1240
Taurus	48.8m	354	135	157	282
Taurus	73.2m	200	56.0	59.2	106
Truck	12.2m	497	417	818	743
Truck	24.4m	1430	923	1804	1830
Truck	48.8m	418	215	360	450
Truck	73.2m	205	85.3	125	170

In an effort to quantify the correlation, the common logarithm of the luminance value for each product in Table 4 was plotted against the cumulative rankings from Table 1. For example, the log(luminance) of Products B, C, A, and D for the car at 12.2 m was plotted against the cumulative rankings of 0, 0 + 1.5, 0 + 1.5 + 0.2, and 0 + 1.5 + 0.2 + 0.7. The results for the car and truck are presented in Figures 1 and 2, respectively. A correlation was taken to exist if the luminance of the marking increased as the cumulative ranking increased, that is, if the graph had a continuously positive slope. No mathematical analysis of the data was performed, and no mathematical relationship was assumed. As can be seen, excellent correlation exists for both vehicles at 24.4 m and beyond. At 12.2 m for the truck the overall rankings as predicted by luminance values matches with subject perception, but the correlation is not as strong as at the greater distances. At 12.2 m in the car the correlation is poorest.

CORRELATION OF VISUAL OBSERVATIONS TO COMMON TEST GEOMETRIES

The third purpose of this experiment was to compare the nighttime visual observations with commonly used retroreflectivity test geometries. Table 5 gives the photometric angles of the various test methods used in the industry throughout the world. The test geometries given represent common laboratory test methods as well as geometries used in portable instrumentation.

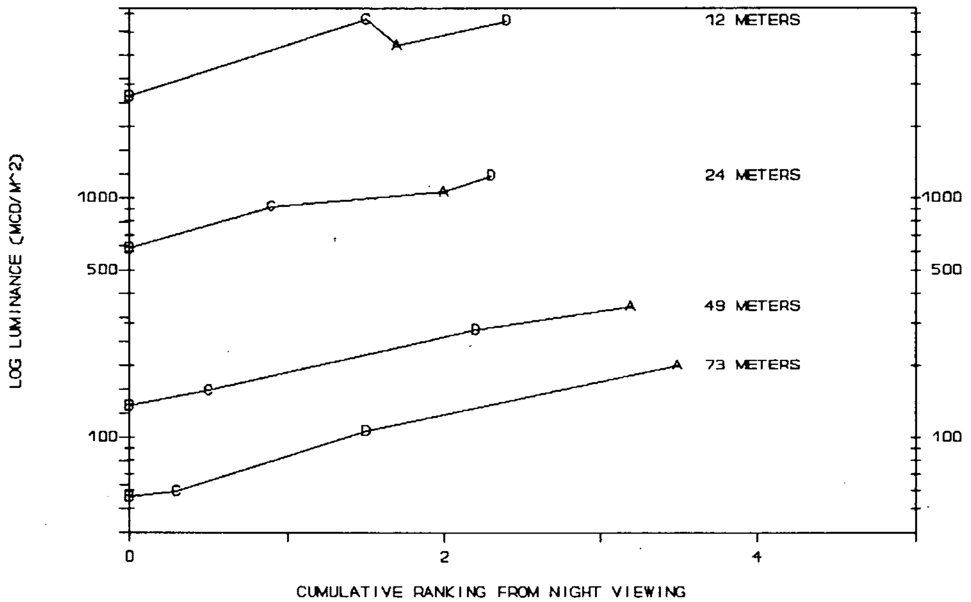

FIGURE 1 **Correlation of luminance with cumulative ranking for a Taurus.**

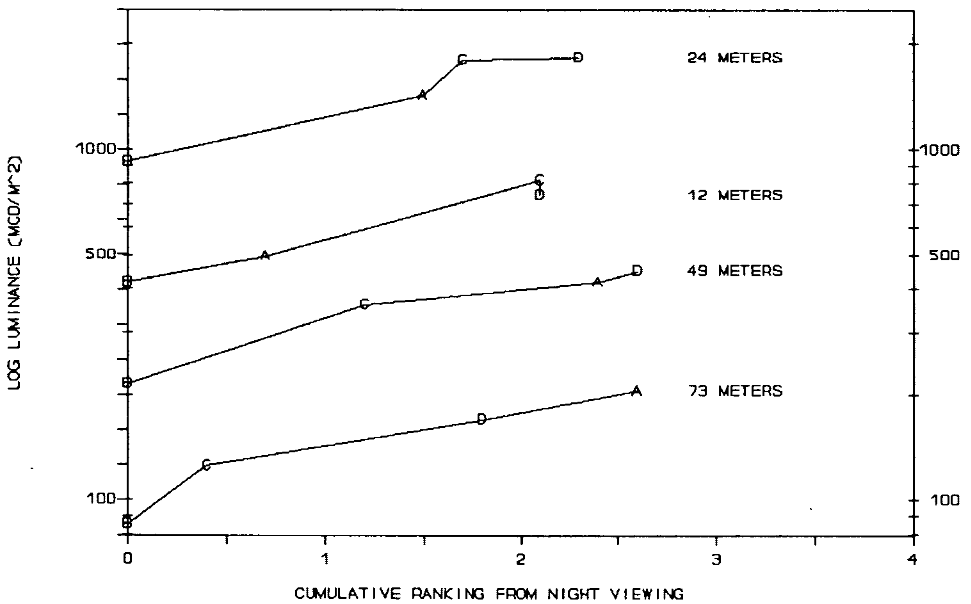

FIGURE 2 **Correlation of luminance with cumulative ranking for a Mack Truck.**

Table 6 gives the values of R_L at the photometric angles of Table 5 for each of the four products used in the night viewing. The samples were measured in a photometric range.

Graphs similar to Figures 1 and 2 were prepared for each of the common test geometries to determine whether there was a relationship to the visual observations. A geometry was stated to correlate with observation if measurement values of products increased as product ranking increased, that is, if the slope of the graph was continuously positive. Table 7 summarizes at what distance a specific test geometry correlated with the visual observations of Table 1. A question mark following a specific distance means that the correlation was not perfect; that is, the order of the products as measured by R_L at that geometry did not exactly match the order as ranked by the observers. No geometry correlated well at both short and long distances for either vehicle. In general, only one geometry correlated well at long distances for both vehicles. That geometry has an entrance angle of 89.3 degrees and an observation angle of 0.63 degrees.

CONCLUSIONS

Four different white pavement marking tapes were viewed by 12 subjects as isolated center skip lines at 12.2, 24.4, 48.8, and 73.2 m from the front of an automobile and semitruck

TABLE 5 Commonly Used Pavement Marking Test Geometries

	Intrinsic Geometry (Degrees)			
Instrument/ Test Method	Entrance (Beta)	Observation (Alpha)	Presentation (Gamma)	Orientation (Omega)
ASTM D 4061-89	86.0	0.2	0.0	180
ASTM D 4061-89	86.0	0.5	0.0	180
Ecolux	86.5	1.0	0.0	180
Mirolux	86.5	1.5	0.0	180
ART LLR IV	88.5	1.0	0.0	180
CEN	88.8	1.05	0.0	180
LTL800	89.3	0.63	0.0	180

TABLE 6 Coefficient of Retroreflected Luminance at Common Geometries

Intrinsic Geometry (Degrees)				Preformed Pavement Marking Sample			
Beta	Alpha	Gamma	Omega	A	B	C	D
86.0	0.2	0.0	180	1820	726	3950	2310
86.0	0.5	0.0	180	1400	586	2790	1830
86.5	1.0	0.0	180	1010	479	1720	1420
86.5	1.5	0.0	180	781	394	1240	1150
88.5	1.0	0.0	180	900	473	1060	1050
88.8	1.05	0.0	180	869	444	900	957
89.3	0.63	0.0	180	1170	571	887	1080

TABLE 7 Correlation of Common Geometries with Observed Ranking

Intrinsic Geometry (Degrees)				Distance that Correlates (meters)	
Beta	Alpha	Gamma	Omega	Taurus	Truck
86.0	0.2	0.0	180	none	none
86.0	0.5	0.0	180	none	none
86.5	1.0	0.0	180	none	12?
86.5	1.5	0.0	180	none	12
88.5	1.0	0.0	180	none	12,24?
88.8	1.05	0.0	180	12?	24
89.3	0.63	0.0	180	49,73	49?73

cab. Differences between the brightness of the four products were observed. The relative rankings of the four products from brightest to dimmest were found to be a function of both distance and vehicle type.

The retroreflectivity of the four pavement marking tapes was measured using common test method geometries. The four tapes were found to have greatly different retroreflectivities depending on the method or equipment used. The relative product rankings of retroreflectivity by many test methods was found not to correlate with the relative brightness rankings assigned by the subjects. Whereas a few of the common retroreflectivity test methods produced rankings similar to those of the subjects at specific distances, no one method was effective at ranking the relative product brightness under all viewing conditions.

Because of the changes in order of ranking of the products with distance and vehicle type, it appears that current instrumental methods are inadequate for ranking retroreflective differences between different classes of products. Current instrumental methods for measuring retroreflective brightness may have some value in monitoring variability within a single product, as well as monitoring changes in that product's performance throughout its useful life; however, this remains to be established.

When laboratory darkroom measurements of pavement marking retroreflectivity were made at actual driver geometries at the distances used in the human viewing experiment, excellent correlation was observed. It is now possible to predict relative brightness performance of unworn pavement marking stripes under static viewing conditions with laboratory measurements at appropriate geometries.

ACKNOWLEDGMENTS

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Physical Testing of Traffic Stripe Paint Durability

FRAZIER PARKER, JR., AND W. LEE SHOEMAKER

Two laboratory tests were used to evaluate several physical properties of traffic stripe paints. The objective of this evaluation was to provide a better understanding of these properties as they relate to predicting durability of paint stripes in the field. Tensile tests of free film specimens of paint yielded several properties derived from the stress-strain curves. Abrasion tests provided results for paint specimens tested both dry and submerged in distilled water. The tests produced consistent and repeatable results. Ten paint samples representing several different volatile and nonvolatile vehicles were evaluated using the two laboratory tests. The paints were different as reflected in the tensile properties. The water-based paints were considerably more ductile than the organic solvent–based paints. The load rate used for the test had significant effect on the results because of increased viscous creep introduced at slower load rates. The 10 paints were evaluated in the field for 5 years. Three water-based paints exhibited superior performance. The performance of these water-based paints correlated well with modulus of toughness computed from tensile tests and wear indices from dry Taber abrasion tests.

Highway pavement markings are subjected to traffic and environmental forces that cause their deterioration and the need for a continuous remarking effort. Pavement markings that become unacceptable because of deterioration are safety and operational hazards. Any improvements in durability or means of evaluating durability will reduce maintenance costs.

Possible modes of failure for pavement markings include poor adhesion, chipping, abrasion, loss of reflectivity (poor bead retention), and discoloration. Factors that affect the performance of traffic paint include paint formulation, substrate, surface preparation, humidity and temperature, traffic volume, and striping equipment.

Field tests have been used in the past to evaluate durability, reflectivity, and other characteristics of pavement marking materials. However, the rapid introduction of new materials is not conducive to lengthy and loosely controlled field evaluations.

Physical properties of paint, determined by a laboratory testing procedure, that can be used to predict paint durability in service would be useful to highway officials in selecting pavement marking materials. The tests could also be used in quality control programs to ensure that delivered and applied products meet specifications. To this end, a laboratory testing program was developed and carried out on 10 traffic stripe paints. Physical properties derived from tensile tests and abrasion tests were obtained. A field study was implemented to develop performance data for the 10 paints for correlation with laboratory-measured properties. Performance was judged on the basis of paint stripe area loss under the action of environmental forces and traffic, but evaluation of reflectivity was not included.

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BACKGROUND

Traffic Paints

Pavement marking paints contain pigments, volatile vehicles, and nonvolatile vehicles. The type of vehicle used has considerable effect on the physical properties of interest. The volatile vehicle is either an organic solvent or water. Organic solvents evaporate into the atmosphere and have been criticized for being an unacceptable form of air pollution in some areas. This has been one of the factors in a change to more water-based paints or modified solvent-based paints with a greater percentage of nonvolatile vehicle. The most widely used type of nonvolatile vehicle (resin) is a drying oil that cures by oxidation after the evaporation of the solvents. Alkyds have been the most commonly used resin in traffic stripe paints. To increase reflectivity, glass beads are added to the paint. These can be premixed into the paint, dropped onto the freshly painted stripe, or both.

The selection of paint formulation is a trade-off between (*a*) drying time, (*b*) airborne organic solvent regulations, (*c*) flexibility, (*d*) hardness, (*e*) bead retention characteristics, and (*f*) adhesion of the substrate. One of the primary considerations in selecting a pavement marking paint is the drying time required. The organic solvent–based paints dry the fastest, but this subsequently reduces the flexibility of the paint. The water-based paints dry more slowly and have greater flexibility and lower abrasion resistance. The slower curing time required by alkyd resin paints has been improved by the addition of chlorinated rubber, but this has been known to decrease flexibility. Water-based paints require more attention to surface preparation to achieve a good bond, whereas organic solvents are much better in this respect, especially on asphalt surfaces.

All of these characteristics of paint are generally known, but better ways of quantifying the relevant properties that can be used to predict the service life of the paint are still sought. The results reported in this paper will advance the knowledge of the physical properties of typical traffic stripe paints.

Past Research

Several research efforts (*1–4*) have been conducted to develop accelerated laboratory testing procedures to predict paint durability. General information regarding testing to determine durability of organic coatings can be found in ASTM STP 691 (*5*) and ASTM STP 781 (*6*).

The literature revealed numerous laboratory test methods for traffic stripe paint, including tensile tests on thin films and various abrasion resistance tests. Field tests are regarded as the most thorough method for assessing stripe durability with mixed correlation reported between laboratory-measured paint properties and field performance. Chipping and abrasion are identified as predominant painted traffic stripe failure modes and are related to film tensile properties and abrasion resistance. Placement conditions and quality, particularly as related to bond development, affect stripe durability.

EXPERIMENTAL DESIGN

Two testing methods were selected on the basis of the findings and recommendations cited in the literature review, types of failure modes expected of paint, and tests most likely to predict susceptibility to these failure modes. Tensile tests of free film samples were used to quantify susceptibility to cracking, and abrasion tests were used to quantify susceptibility to normal wear.

Paints Tested

Four traffic paints that were in inventory were provided by the Alabama Highway Department as follows (identification code is in parentheses):

- Water base (white) (WB-2-W),
- Water base (yellow) (WB-3-Y),
- Organic solvent base (white) with premixed beads (SB-1-W), and
- Organic solvent base (yellow) with premixed beads (SB-2-Y).

In addition, three companies provided the following five paints for the study:

- Water base (white) (WB-1-W),
- Modified alkyd-chlorinated rubber (white) (CR-1-W),
- Modified alkyd-chlorinated rubber (yellow) (CR-2-Y),
- Alkyd resin (yellow) (AR-1-Y), and
- Modified alkyd resin (white) (MAR-1-W).

For an extreme data point, a sample of white latex (water base) house paint (HP-1-W) was tested.

Tensile Tests

Free film tensile tests, used successfully in other cited research efforts, involve applying the paint to a backing material from which the paint can be separated after drying. Cut into the shape of a tensile coupon (bone shape), the sample can then be tested while load-deformation data are recorded for evaluation of several tensile properties.

The following procedure was used to produce the tensile test coupons:

1. A 5-cm-wide Teflon tape was applied to a metal plate.
2. The paint was puddled at one end of the taped plate and a Bird film applicator was used to draw a 0.38-mm wet film thickness across the surface. A dip-coater (ASTM D 823-84 Method B) was also tried to obtain a uniformly coated surface, but this method did not work well for all of the paints because the tape tended to "shed" some of the paints when placed in a vertical position to dry.
3. The samples were allowed to dry for 24 hr at room conditions and then cut to the desired shape. The die cutter was manufactured to specifications that provided a gauge length of 3.8 cm and a throat width of 0.6 cm.
4. After being cut, the paint coupon was peeled from the Teflon tape and hung vertically to cure for an additional 48 hr.

A Tinius Olsen universal testing machine fitted with rubber-lined, low-capacity grips was used for the tensile testing of the paint samples. A load cell with a capacity of 44.5 N was installed between the crosshead and the upper grip for greater sensitivity and resolution. Tests were conducted under a controlled deformation rate mode. The thickness of each paint sample was measured with a micrometer to the nearest 0.001 mm and recorded for use in the conversion from force to stress, calculated as follows:

$$s = 1{,}000P/(t_w * t) \qquad (1)$$

where

s = stress,
P = force,
t_w = throat width, and
t = thickness of paint.

Strain was calculated as

$$\varepsilon = \delta/g \qquad (2)$$

where

ε = strain,
δ = LVDT deflection, and
g = gauge length.

Abrasion Tests

Abrasion tests performed on paints typically use either an abrading rotating wheel or sand falling onto a painted surface. A Taber Abraser, of the former variety, was selected because of its wide use by other highway departments and research studies. The reliability of Taber Abraser results has been questioned in the past. However, it was believed that the Taber Abraser offered the best available method when procedures for preparing the abrading wheels are carefully followed. ASTM D-4060 and Federal Test Method Standard 141a provide testing procedures for using the Taber Abraser.

The following procedure was used to produce the specimens used for the abrasion tests:

1. Paint thinner was used to clean 10- × 10-cm steel specimen plates.
2. A Bird applicator was used to apply a 0.38-mm wet film thickness of paint. Five specimens were prepared for each paint sample.
3. The prepared specimens were allowed to dry for 24 hr at room conditions.

The three testing parameters for the Taber Abraser are the choice of abrasive wheels, the weight placed on the wheels, and the number of cycles to which the specimen is subjected. The wheels selected were CS-17 resilient wheels composed of rubber and abrasive grain, which produce a harsh abrasive action. The arms were weighted each with 1000 grams. The specimens tested dry were subjected to 2,000 cycles, and the specimens tested submerged were subjected to 500 cycles. The following test procedures were followed:

1. The specimen plates were weighed.
2. The weighed specimen plate was mounted on the Taber Abraser and the CS-17 wheels lowered into position.
3. The automatic counter was set to the appropriate number of cycles and started. A vacuum pickup was used to remove loose particles from the specimens during dry testing.
4. At the end of the cycles, the specimen plate was weighed and the wear index computed as

$$\text{Wear index} = [(A - B) \times 1{,}000]/C \qquad (3)$$

where

A = weight before abrasion,
B = weight after abrasion, and
C = number of cycles.

Field Evaluation

Transverse test stripes were placed across the outside lane of US-29 east of Auburn, Alabama, in March 1987 at the junction of a 1-year-old and a 3-year-old asphalt surface. The AADT for the four-lane facility is approximately 13,000. Two stripes each of the 10 paints were placed on both the 1- and 3-year-old surfaces. Stripes were placed by experienced Alabama Highway Department personnel with a walk-behind unit. Calibrations were done to try to achieve a film thickness of about 0.25 mm. The average for all stripes was 0.26 mm. There was, however, considerable between- and within-stripe film thickness variability, but no apparent relationship with performance. Placement temperature was approximately 21°C.

The condition of the stripes was observed periodically to assess their performance. Estimates of paint surface area retained form the basis for evaluation of performance and comparison with measured paint properties. The paint areas lost in 1.2-m-wide strips across wheelpaths were estimated and used to compute a percentage of stripe area remaining. The reported results are the average from two raters rounded to the nearest 5 percent. Results from individual raters were always within 15 percent. Initially observations were made frequently, but the interval between observations increased with time. A final 5-year evaluation was made in March 1992.

RESULTS AND DISCUSSION

Tensile Tests

Tensile tests were performed on nine paints [samples of (CR-2-Y) could not be prepared because of extreme brittleness]. Samples cured at room conditions were tested at load rates of 1.25 cm/min. The properties of the paints are given in Table 1. At least three samples were tested for each paint. The mean and coefficient of variation are reported for film thickness, ultimate strength (peak of stress-strain curve), modulus of toughness (area under stress-strain curve to ultimate strength), failure strength (breaking strength), and elongation at failure and initial modulus (initial slope of stress-strain curve).

Representative stress-strain curves for three paints are shown in Figure 1. These plots show the significant differences in the tensile characteristics between the paint formulations. The solvent-based alkyd resin paint (AR-1-Y) is very brittle, whereas the modified alkyd resin paint (MAR-1-W) has considerably more ductility but a lower ultimate strength. The water-based

TABLE 1 Tensile Properties (Cure, Room Condition; Load Rate, 1.25 cm/min)

	Thickness (mm)		Ultimate Strength (kPa)		Mod. of Toughness (kPa)		Failure Strength (kPa)		% Elongation at Failure		Initial Modulus (MPa)	
Paint	Mean	CV	Mean	CV	Mean	CV	Mean	CV	Mean	CV	Mean	CV
HP-1-W	0.22	0.45	2737	0.13	724	0.31	2427	0.19	30	0.29	74	0.24
WB-1-W	0.38	0.18	4082	0.05	1345	0.06	2882	0.06	39	0.11	83	0.10
WB-2-W	0.36	0.25	1738	0.09	2055	0.25	1365	0.08	133	0.19	26	0.35
WB-3-Y	0.31	0.32	1744	0.06	1441	0.25	1179	0.07	100	0.27	26	0.17
CR-1-W	0.32	0.21	8605	0.26	138	0.32	8605	0.26	2	0.30	414	0.37
MAR-1-W	0.50	0.09	1207	0.03	421	0.07	883	0.06	40	0.10	20	0.14
SB-1-Y	0.40	0.15	2930	0.04	48	0.13	2930	0.04	2	0.23	288	0.56
SB-2-Y	0.24	0.05	4675	0.09	124	0.12	4675	0.09	4	0.15	138	0.14
AR-1-Y	0.38	0.09	4709	0.06	103	0.12	4709	0.06	3	0.15	191	0.21

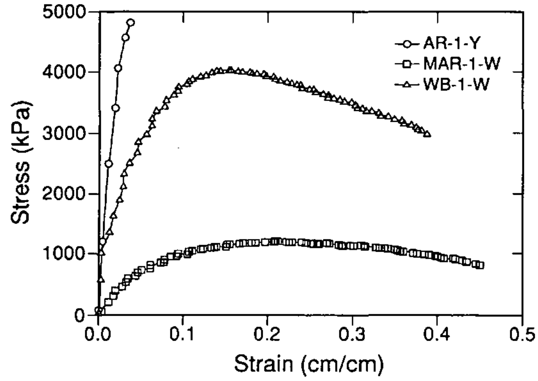

FIGURE 1 Representative stress-strain curves.

paint (WB-1-W) has good ductility, high ultimate strength, and a failure strength considerably lower than the ultimate strength.

Abrasion Tests

Abrasion tests were run on five specimens of each of the 10 paints as previously described. The specimens were tested dry for 2,000 cycles. Abrasion tests were also run on the same number of specimens submerged in distilled water for 500 cycles. Results for the house paint (HP-1-W) were not useful because the paint lost adhesion to the specimen plate. The mean of the wear indices and coefficient of variation for each series of tests are given in Table 2. Note that the smaller the wear index, defined by Equation 3, the more abrasion resistant is the paint and that the index approaches zero for no wear. Table 2 also contains the relative ranks of the paints tested. The lower the rank, the better the performance for that test.

The most significant findings of these tests were the relatively poor performance of the water-based paints when submerged in water. The alkyd resin paint was the best performer overall considering both conditions. The generic solvent-based paints (SB-1-W and SB-2-Y) performed poorly in the dry test but did better in the submerged test. The modified alkyd resin paint performed poorly for both abrasion test conditions.

Field Evaluations

The transverse stripes were observed periodically to assess their performance for comparison with paint properties. The early response of the stripes provides interesting insight into the influence of bond and paint stress-strain characteristics. Long-term performance appears to be controlled primarily by wear and abrasion resistance.

Table 3 contains a summary of the performance of the stripes after 5 years. These data indicate that except for the paint (WB-1-W), the stripes on the 3-year-old pavement surface performed at least as well as the stripes on the 1-year-old pavement surface. Replicate samples on the 1- and 3-year-old surfaces performed similarly.

The house paint (HP-1-W) was incompatible with asphalt pavement surfaces. The paint did not adhere to the pavement surface and the stripes were completely obliterated in 2 or 3 days.

The water-based paint (WB-1-W) experienced severe early chipping on the 3-year-old surface and accounts for the poor long-term performance (Table 3). The water-based paint (WB-2-W) experienced limited early chipping, but this did not increase with time. The performance of these two paints suggests that the effects of poor adhesion will be apparent early in stripe life.

The solvent-based (SB), chlorinated rubber (CR), and alkyd resin (AR) paints were brittle, and the water-based (WB) paints tended to be more ductile. The more brittle paints also tended to be stiffer and have higher tensile strength. The brittle paints experienced early cracking within and around the periphery of the stripes that progressed in size and number for about 1 year. These cracks penetrated the asphalt surface.

Brittleness is primarily responsible for the transverse stripe cracking. As the paint dries and shrinks and as the pavement/paint expands and contracts with temperature, shear stresses at the paint-pavement interface induce tensile stresses in the paint film that cause cracking. The more ductile paints are able to develop the necessary strain without exceeding tensile strength, whereas brittle paints are not able to accomplish this even though their tensile strength may be greater. As for the periphery cracks in the asphalt, the expansion and contraction of the pavement in response to temperature gradients are inhibited by the stripe. The more flexible and ductile paints are better able to conform to pavement movements.

TABLE 2 Taber Abraser Results

	Wear Index					
	Dry			Submerged		
Paint	Mean	CV	Rank	Mean	CV	Rank
HP-1-W	0.419	0.048	7	–	--	10
WB-1-W	0.167	0.045	1	0.586	0.220	6
WB-2-W	0.198	0.053	2	0.458	0.045	5
WB-3-Y	0.222	0.022	5	0.758	0.058	8
MAR-1-W	0.574	0.026	8	0.762	0.198	9
CR-1-W	0.221	0.201	4	0.366	0.063	2
CR-2-Y	0.350	0.045	6	0.611	0.260	7
SB-1-Y	0.691	0.234	9	0.455	0.178	4
SB-2-Y	0.761	0.052	10	0.383	0.112	3
AR-1-Y	0.220	0.041	3	0.284	0.057	1

TABLE 3 Summary of Paint Performance After 5 Years

Paint	% Paint Surface Remaining 3-yr old	1-yr old	Average	Comments
HP-1-W	0	0	0	–
WB-1-W	20	90	55	Early chipping on 3-yr old surface
WB-2-W	95	95	95	–
CR-1-W	0	0	0	–
MAR-1-W	55	15	35	Diff. 3- and 1-yr surface
SB-1-W	50	10	30	Diff. 3- and 1-yr surface
CR-2-Y	70	30	50	Diff. 3- and 1-yr surface
WB-3-Y	100	100	100	–
AR-1-Y	40	20	30	–
SB-2-Y	70	30	50	Diff. 3- and 1-yr surface

Other studies have observed the same periphery cracking around traffic stripes and sealers on asphalt. This has also been explained as being caused by the traffic paint deforming the asphalt underneath the edges (7).

The influence of early cracking on long-term performance could not be definitely ascertained. Progressive loss of area appeared to be due to wear or abrasion and some chipping that did not appear to be directly related to the early cracking. However, comparison of long-term stripe performance with paint stress-strain properties indicated that paints that cracked early lost area faster.

Correlations

Average percent remaining paint surface from Table 3 and various paint stress-strain properties from Table 1 were plotted to examine relationships between tensile properties and stripe performance. The plots do not show any strong continuous relationships, but there is a distinct grouping of the two water-based paints (WB-2-W and WB-3-Y) that exhibited superior performance. This grouping is strengthened if the performance of the third water-based paint (WB-1-W) on the 3-year-old surface is discounted and the performance on the 1-year-old surface used instead of the average. The plots indicated that high ductility, as measured by percent elongation at failure, may be important but that ultimate tensile strength may not be important. Figure 2 indicates that modulus of toughness, which combines strength and ductility, is definitely important. The water-based paints with large toughness performed well, whereas the SB, CR, and AR paints with small toughness performed poorly. This suggests that the early cracking may be a factor in performance, although not discernible with periodic visual observations.

Dry wear index values from Table 2 and average percent remaining paint surface from Table 3 are plotted in Figure 3. Again the three water-based paints that had superior performance are grouped. However, the chlorinated rubber paints (CR-1-W and CR-2-Y) and the alkyd resin paint (AR-1-Y), which had poor performance, also have low abrasion loss. This suggests that stripe performance is a function of both paint stress-strain and abrasion characteristics, and that physical tests for both should be included for evaluation. Although additional testing and evaluation are needed to establish definitive criteria, the data from this study suggest that a modulus of toughness greater than 1400 kPa and a Taber dry wear index less than 0.25 are required to ensure long-term stripe performance. Wear indices from submerged abrasion testing were also compared with stripe performance but did not differentiate the superior-performing water-based paints.

Bond is also important but may be controlled through chemical compatibility with pavement surface materials. Bond

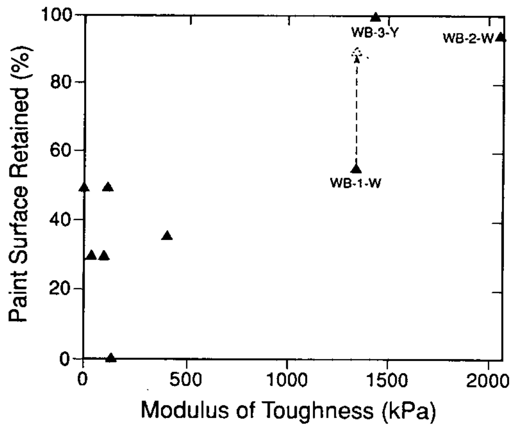

FIGURE 2 Stripe performance versus paint toughness.

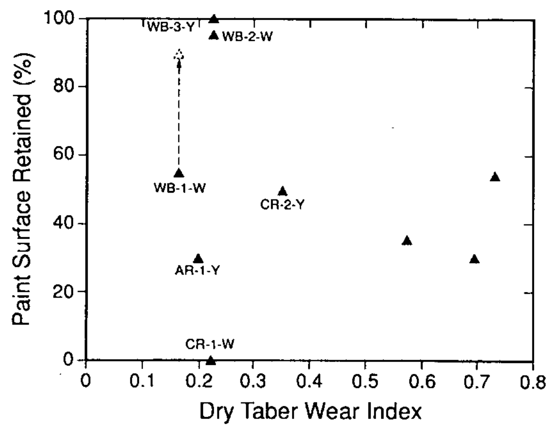

FIGURE 3 Stripe performance versus wear index.

development is also sensitive to application conditions. It is suspected that improper application was the cause of poor adhesion of the paint (WB-1-W) on the 3-year-old surface, which led to early chipping. To evaluate paints for acceptance and possibly for quality control purposes, some form of bond test is needed to complement toughness and abrasion tests. A possibility would be to modify the Taber abrasion tester to test samples applied directly to pavement surfaces rather than metal plates. This modification would provide an evaluation of adhesion as well as resistance to abrasion.

CONCLUSIONS

Two laboratory tests were performed to evaluate physical properties of traffic stripe paints. Tensile tests of free film specimens of paint yielded several properties derived from stress-strain curves. Abrasion tests provided abrasion resistance for both wet and dry conditions. The tests produced consistent and repeatable results that varied widely for the different types of paints tested.

Ten paint samples representing a variety of volatile and nonvolatile vehicles were tested and evaluated using the laboratory tests developed. The paints were quite different as reflected in the tensile properties. The water-based paints were considerably more ductile than the organic solvent-based paints.

The abrasion tests also produced a wide variation of results among the 10 paints tested. Dry testing produced lower wear indices, and water-based paints performed poorly when tested wet. The alkyd resin paint tested had the highest abrasion resistance considering both wet and dry test results. Dry wear indices correlated best with field stripe performance.

The 5-year field performance of the ductile water-based paints was superior to the more brittle solvent-based, chlorinated rubber, and alkyd resin paints. Correlation of stripe performance with modulus of toughness and dry Taber abrasion wear index indicated that paint properties measured by both are important and should be considered in paint evaluation.

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Evaluating the Potential of Remote Sensing Rural Road and Travel Conditions

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Communication of current road and travel conditions may reduce the number of accidents attributable to winter driving conditions in rural mountain states. During the last 5 years, 61.1 percent of the total yearly accidents at the study site occurred during the relatively small percentage of time (9.8 percent) that the road and travel conditions were poor. Use of real-time remote weather information for updating road and travel information was evaluated. Spot speed surveys for different road and travel conditions, road user surveys, snowplow operator reports, and remote weather information system (RWIS) data were analyzed to evaluate the effectiveness of the real-time weather information system. The existing RWIS did not correlate well with the road conditions reported by road users or snowplow operators. An upgrade of the RWIS is needed to improve reliability. The addition of visibility measuring equipment (particle counter) is needed. Additional RWIS sensor locations and automatic speed monitoring should also be considered.

The number of accidents attributable to winter driving conditions on the Interstate road system is a significant problem in Wyoming and other mountain states. One possible solution for addressing the winter accident problem is communication of current road and travel conditions. The main communication objectives are (*a*) to provide the road user with information about the severity of the road and travel conditions so that the road user may determine whether to proceed with a trip and (*b*) to provide the road user with adequate warning so that driving habits may be adjusted.

This paper provides an evaluation of a remote weather information system (RWIS) to assist governmental bodies in updating road and travel information. Real-time road and travel information can be communicated to road users using a variety of devices including changeable message signs (CMSs), road and travel telephone numbers, road and travel information on public radio, and linear radio systems. The key need is to obtain real-time road and travel conditions on rural roads. Presented in the following section are results of a University of Wyoming survey of departments of transportation concerning use of RWIS and adverse road advisory messages.

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EXISTING USE OF RWISs

Several states currently use RWISs for maintenance purposes. The RWIS is used by several agencies to predict when snow/ice control measures will be required. California, Florida, South Carolina, and Wyoming are a few states that have used RWIS for updating or supplementing weather data to determine the road and travel advisories for road users.

The California Department of Transportation (Caltrans) currently uses RWIS in conjunction with changeable message signing to regulate traffic. Road closure information and expected delays are the types of information concerning poor road and travel conditions provided to road users by Caltrans.

The Florida Department of Transportation (FDOT) has used fog detection and warning devices in the past, but these were discontinued due to fog detection device malfunctions. FDOT currently uses wind detection devices and related travel advisories posted on CMSs. South Carolina currently uses a fog detection and warning system (*1,2*).

The Wyoming Transportation Department (WTD) currently uses remote weather information systems to detect strong and gusty winds on Interstate 80 near Laramie, Wyoming. An automatic wind warning system consisting of a remote wind speed measuring device and CMSs is currently being used. Strong and gusty winds are measured and compared with predetermined wind speed criteria. If the wind speed criteria are surpassed, a high wind warning message is automatically displayed on CMSs. The criteria used by WTD are wind speeds of 35 mph (56 kph) for dry pavement conditions and 25 mph (40 kph) for icy or snowy pavement conditions. The message that is displayed is wind gusts to xx mph—advise no light trailers. Other states, such as Nevada, also use the CMS to provide wind-related messages. Accurate knowledge of pavement conditions is important to the criteria associated with the Wyoming system. In order to aid in determining conditions other than wind speed, a RWIS was installed at the study site. The RWIS was located approximately 13 mi (21 km) east of Laramie in the 41-mi (66-km) section between Laramie and Cheyenne, which was determined to have the most severe conditions. To evaluate the RWIS, an investigation of accident data was undertaken to determine whether certain user groups needed to be targeted for RWIS information.

ACCIDENT ANALYSIS

The accident data on Interstate 80 between Laramie and Cheyenne were evaluated to determine trends in winter accidents. For 1986 to 1991 there was an average of 193 accidents per year. Of those, 118 accidents occurred when roadway conditions were poor (icy, snowy, or slushy). During this period, the average accident rate for all road users during poor road and travel conditions for the study site was 11.63 (number of vehicles involved per 1,000,000 mi of travel). This was about 13 times greater than the accident rate for all road users during favorable road conditions (0.90). Accident rates were also estimated for local Wyoming, other Wyoming, and out-of-state passenger vehicles and trucks. Traffic volume data, vehicle classification data, snow/ice maintenance data, and accident data were used to estimate accident rates for each combination of vehicle type, driver proximity, and pavement condition (see Table 1). The average accident rate for out-of-state road users was 19.04 (number of vehicles involved per 1,000,000 mi of travel) during poor road conditions. The average accident rate for local Wyoming road users was 12.57 during poor road conditions.

The accident rates for local Wyoming road users in poor road conditions was 10 to 25 times higher than in favorable conditions. The average accident rate for all passenger vehicles was 1.06 for favorable road conditions and 13.99 for poor road conditions. The average accident rate for trucks was 0.74 for favorable road conditions and 9.74 for poor road conditions. The numbers of accidents that occurred with different roadway conditions are given in Table 2.

The yearly number of accidents when road conditions were poor was about 60 percent higher than the yearly number of accidents when road conditions were favorable. The accidents that occurred during poor road conditions happened in a time span that amounted to approximately 10 percent of the total time during the year. Using this knowledge, adverse road and travel conditions were classified and spot speed surveys were conducted during the 1990–1991 and 1991–1992 winters.

SPEED SURVEYS

Speed surveys were incorporated to determine how varying degrees of visability, wind, and pavement conditions affect road user behavior. The speed surveys were conducted adjacent to the RWIS site for eastbound traffic and separated into two categories—passenger vehicles and trucks. The stopwatch method of measuring time over a distance was used to determine the spot speeds. Speed data were obtained for both the passenger vehicle and truck classifications. In total, more than 5,600 independent speed observations were made. The weather-related road and travel conditions were determined as the speed surveys were being conducted. Speed surveys were recorded by time and combinations of visibility, wind, and pavement conditions.

The visibility condition was classified as either favorable or poor. The visibility condition was considered favorable if there was at least 600 ft (183 m) of sight distance. The visibility condition was classified as poor if less than 600 feet (183 m) of sight distance was available. The wind condition was classified as either calm or strong and gusty. The pavement conditions were classified as favorable (dry or wet), slick in spots, or poor (slushy, snowpacked, or icy). The pavement condition was checked at regular intervals during each observation period.

The spot speed surveys were analyzed to determine whether varying degrees of visibility, wind, and pavement conditions

TABLE 1 Estimated Accident Rates[a]

Passenger Vehicles

	Dry or Wet			Icy, Snowy, or Slushy		
Year	Local Wyoming	Other Wyoming	Out of State	Local Wyoming	Other Wyoming	Out of State
1986	0.61	1.36	1.78	11.30	3.74	14.76
1987	1.16	0.47	2.28	15.50	3.22	27.02
1988	1.03	0.45	2.19	15.19	8.49	19.93
1989	1.04	0.14	1.39	10.46	12.00	23.66
1990	0.95	0.28	1.35	7.53	5.61	23.25
1991	0.49	0.68	1.39	12.47	9.52	28.16

Trucks

	Dry or Wet			Icy, Snowy, or Slushy		
Year	Local Wyoming	Other Wyoming	Out of State	Local Wyoming	Other Wyoming	Out of State
1986	0.00	0.00	0.38	8.19	0.00	12.64
1987	2.76	0.00	0.71	21.16	0.00	15.43
1988	3.25	0.69	0.92	31.85	0.00	17.53
1989	0.62	0.00	0.61	5.25	0.00	20.49
1990	0.62	0.00	0.77	0.00	5.24	9.48
1991	1.19	0.00	0.79	11.90	0.00	16.08

[a] Accident rates are number of vehicles involved in accidents per 1,000,000 miles of travel

TABLE 2 Number of Accidents

	Conditions		
Year	Dry or Wet	Icy, Snowy, or Slushy	Total
1986	58	75	133
1987	86	112	198
1988	84	122	206
1989	71	151	222
1990	74	106	180
1991	69	142	211

were significant factors affecting motorist behavior during periods of poor road and travel conditions. The speed survey data were then analyzed using regression analysis procedures. Average speed and percent in the 10 mph pace were the dependent variables used to evaluate the effect of poor road and travel conditions on road users. The average cell sample size contained 333 observations for passenger vehicles and 231 observations for the truck classification.

The dependent variable, average speed, was modeled against the predictors visibility, wind, pavement, vehicle type, and interaction effects of visibility*wind, visibility*pavement, visibility*vehicle type, wind*pavement, wind*vehicle type, pavement*vehicle type, and pavement*pavement. Stepwise model building procedures (forward addition, backward elimination, and maximum R^2) were used to determine the best predictors. The consensus of the three stepwise regression models was a model including visibility, visibility*wind, and pavement as the best predictors of average speed.

The resulting best model for predicting average speed was

$$\overline{S} = 62.5083 - 8.9833(V) + 7.4583(V)(W) - 4.7417(P) \quad (1)$$

where

$\overline{S}$ = average speed (mph) (1 mph = 0.62 kph),
V = visibility (0 = favorable, 1 = poor),
W = wind (0 = favorable, 1 = poor), and
P = pavement (0 = favorable, 1 = slick in spots, 2 = poor).

The resulting coefficient of determination for the model was $R^2 = 0.92$, showing a good relationship between the predictors and average speed. Pavement and visibility were the most important factors affecting average speed. The interaction between visibility and wind was also a significant factor. Vehicle type was not a significant factor affecting average speed during periods of poor road and travel conditions.

The dependent variable, percent in pace, was also modeled using the same predictors. Using the model building procedures, only pavement was found to have an appreciable effect on the percent of road users traveling in the 10-mph pace.

The best model for predicting percent in pace was

$$\text{Percent in pace} = 68.9580 - 7.9072(P). \quad (2)$$

where P is pavement (0 = favorable, 1 = slick in spots, 2 = poor).

The resulting coefficient of determination for the model was $R^2 = 0.72$, showing a fair relationship between pavement and percent in pace. The regression model indicates that as pavement conditions became more adverse, the percent of road users traveling in the 10-mph pace was reduced. The ability of the RWIS to reflect road and travel conditions was investigated using the same classifications.

RWIS, ROAD USER SURVEYS, AND SNOWPLOW OPERATOR REPORTS

Real-time weather data from RWIS were collected from December 1990 to January 1992. The core of the RWIS is a surface sensor and a set of atmospheric condition sensors. The output from each of the sensors is fed to a microprocessor called a remote processing unit (RPU), which converts the output into identifiable conditions and then stores the conditions in memory. The measured weather data recorded by the RPU include presence of precipitation, surface pavement temperature, air temperature, relative humidity, wind speed, and wind direction. The RPU then determines the pavement status and dew point on the basis of the measured parameters. All of the data are updated when a predetermined significant change is measured for any of the seven parameters.

Local commuters and interstate truckers were surveyed to determine their evaluation of road and travel conditions. Local commuters completed questions in a travel diary and Interstate truckers were surveyed using citizens band radio. Road user characteristics and their classification of the road and travel conditions by visibility, wind, and pavement conditions were obtained.

Information concerning road and travel conditions was also obtained from WTD for the study site from December 1990 to January 1992. Snowplow operators described road and travel conditions in terms of visibility, wind, and pavement conditions to radio dispatchers. The radio dispatchers kept a log of road and travel conditions by date and time of day. The results of these data comparisons are summarized in the following section.

RWIS RESULTS

There was little correlation between the visibility reported by the road users and precipitation measured by the RWIS. Precipitation is the only possible indicator of visibility with the present system. The visibility condition was reported as clear 520 times (71 percent), as limited 176 times (24 percent), and as very limited 41 times (5 percent) when no precipitation was present. When precipitation was present, clear visibility was reported 303 times (35 percent), limited 368 times (43 percent), and very limited 185 times (22 percent).

The majority of road users (70 percent) rated the winds as strong and gusty when the RWIS measured wind speeds of 15 to 20 mph (24 to 32 kph). WTD currently uses wind speed criteria for posting advisory wind messages of 25 mph (40 kph) for poor road conditions and 35 mph (56 kph) for dry road conditions.

Pavement conditions reported by the road users did not correlate well with the pavement status provided in the RWIS data. Table 3 contains summaries of these results, which indicate the difficulty of applying a spot detection of road condition to estimate conditions over the entire roadway.

The visibility conditions reported by the snowplow operators also did not correlate very well with the presence of precipitation as measured by the RWIS. Pavement conditions reported by the snowplow operators also did not correlate well with the pavement status provided by the RWIS. These results are contained in Table 4.

Road users generally tended to report less favorable visibility conditions than did the snowplow operators. Road users also reported strong and gusty winds at lower wind speeds as measured by the RWIS than did the snowplow operators. Road users, in general, indicated poorer pavement conditions than the snowplow operators, who most often describe adverse pavement conditions as slick in spots. Although these results indicate overall poor correlation, additional capabilities are possible for RWIS monitoring.

CONCLUSIONS

Safety improvements are needed to reduce the number of winter accidents. Possible solutions for addressing the winter accident problem are education, improved communication of current road and travel conditions, and restriction of travel.

Poor visibility and pavement conditions had the most effect on the average speeds of road users traveling during inclement road and travel conditions. Road users adjusted their travel speeds depending on their perception of the severity of the conditions. Strong and gusty winds should be reported when wind speeds greater than 15 mph (24 kph) are reached, correlating with road user ratings. Current road and travel information needs should be conveyed to the road users so that they can make an informed decision concerning making a trip during varying degrees of adverse winter conditions.

There was very little correlation between the conditions described by RWIS, road user surveys, and snowplow operator reports. The conditions described by the present RWIS did not relate to the overall conditions of the roadway as described by either the road users or the snowplow operators. Therefore, the existing RWIS should not be used solely to determine poor road and travel conditions. The current RWIS does not provide adequate information to accurately determine the road and travel conditions for Interstate 80 between Laramie and Cheyenne. If use of the RWIS to determine road and travel conditions is to be continued, the RWIS should be upgraded to include more weather sensor locations and visibility measurement devices.

RECOMMENDATIONS

A project to develop information on safe winter driving strategies should be performed. The information should address safe advisory speeds to be recommended during specific poor road and travel conditions. Information concerning necessary travel, safe following distances, emergency or evasive maneuvers, and emergency preparedness should be included. Road users should be advised of the risk of traveling during poor road and travel conditions to determine whether their trip purpose justifies the risk. Safe following distances for specific road and travel conditions should be recommended on the basis of available stopping sight distance and pavement condition. Emergency or evasive maneuvers should be recommended concerning the safest places to stop when conditions deteriorate to a level that road users should stop and

TABLE 3 Pavement Conditions, RWIS Versus Road User Surveys

	Road User Surveys						
RWIS Status	Dry	Wet	Slushy	Snow-packed	Slick-in-spots	Icy	Total
Dry	411	62	11	19	236	40	779
Wet	5	12	15	4	34	9	79
Chemical Wet	1	4	3	4	66	58	136
Snow/Ice Alert	39	14	7	44	285	165	554
Total	456	92	36	71	621	272	1548

TABLE 4 Pavement Conditions, RWIS Versus Snowplow Operator Reports

	Snow Plow Operator Reports						
RWIS Status	Dry	Wet	Slushy	Snow-packed	Slick-in-spots	Icy	Total
Dry	86	21	0	5	134	4	250
Wet	1	7	0	5	39	2	54
Chemical Wet	0	0	0	6	46	0	52
Snow/Ice Alert	8	1	0	8	147	28	192
Total	95	29	0	24	366	34	548

wait for conditions to improve. Emergency preparedness information should be assembled so that stranded road users know what to do and have the proper supplies in case of emergency. Information concerning these safe driving strategies should be conveyed to the traveling public in drivers' license examination procedures, port-of-entry handouts, and local media to maximize exposure.

Permanent traffic speed monitoring stations could be installed with additional improved RWIS stations that include particle counters to measure visibility. Reductions in the average speed or percent in pace of road users should be used in conjunction with the expanded RWIS data to determine the road and travel conditions being encountered by the road users.

The additional weather sensor stations would improve the system by sensing poor road and travel conditions at more than one location. This would improve reliability by indicating poor road and travel conditions that would be applicable to a wider segment of the roadway between Laramie and Cheyenne.

The wind speed criteria currently used by WTD for high wind warnings should be lowered to levels consistent with the road user ratings. The RWIS pavement status can be used to supplement other sources of information but should not be used alone because of the poor single point correlation of the pavement status and actual conditions over the 41-mi (66-km) distance between Laramie and Cheyenne. More RWIS sensors would provide the pavement status at more locations, improving the reliability of the system.

ACKNOWLEDGMENTS

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PART 3

Roadside Maintenance

Applicator Training Materials on Use of Chemicals for Vegetation Management

HARVEY A. HOLT

A set of training materials, manuals and videos, has been produced for those applying herbicides and plant growth regulators on roadsides. The events leading to their development, the content, and indications of known usages are described.

Roadside applicators use herbicides and plant growth regulators that can control plants within and without the target area, the narrow roadside right-of-way. This area is in constant contact with adjacent sensitive, nontarget sites. The roadside applicator is subject to continuous public exposure and scrutiny. These applicators must be well trained. However, the Environmental Protection Agency/U.S. Department of Agriculture (EPA/USDA) core and category manuals are too broad. The core manual covers all pests and pesticides, whereas herbicides and plant growth regulators are the primary chemicals used on roadside rights-of-way. The category manual contains very little material specifically identified with roadside vegetation management. Since most states do not have right-of-way extension counterparts at the university level, few states have adequate training materials and comprehensive programs. A large number of roadside applicators do not have access to any education after certification because of county and state restrictions against employees traveling outside their governmental boundaries. Most temporary summer employees receive just enough training for certification.

MODULE BACKGROUND

A coalition of basic manufacturers recognized the seriousness of the training problem and provided the initiative that resulted in new applicator training materials. The manufacturers, American Cyanamid, DowElanco, DuPont, and Monsanto, in cooperation with USDA's Extension Service (USDA-ES) and EPA have provided pesticide applicator training materials to the states that should not only enhance the technical competence of the roadside applicator but also be of value to all rights-of-way applicators. The person becoming initially certified should be better educated, and the training materials should also serve as continuing education material for people already certified.

The National Curriculum Committee was organized by USDA-ES with representatives of state departments of transportation, commercial applicators, and pesticide training coordinators. This committee established the modular format and developed the technical outlines for each module. USDA-ES then sent a request for proposal to all states for development of the training materials. Purdue University prepared a proposal and was awarded the contract in summer 1989.

The module outlines were distributed at the 1989 meeting of the National Roadside Vegetation Management Association (NRVMA). Comments were incorporated into the final outlines. Written texts for each module were distributed at the 1990 NRVMA meeting for review and comments. The corrected documents were the basis from which scripts were written for video production. Video footage was shot nationwide during spring and summer 1991. The final product, videos and manuals, was presented at the 1991 meeting of NRVMA, again with the opportunity for comments and corrections. Corrections were made and final product distributed to all states in winter 1991–1992.

MODULE CONTENT

Each module is prefaced with learning objectives and followed with test questions so that the applicator knows what is expected to be retained.

Module 1: Roadside Vegetation Control Is Necessary

Control of roadside vegetation is necessary to meet safety and legal requirements, for road structure maintenance, and for appearance. Safety and legal requirements include maintenance of a safety recovery zone (clear zone), sign visibility, sight distance, and noxious weed control. Vegetation control improves drainage, slows roadbed degradation, prolongs the life of roadside hardware, and affects snowdrift control, fire hazard reduction, and erosion control. Vegetation control also enhances the delineation and beauty of the roadside. Control measures make use of mechanical, manual, chemical, and biological means; native plant materials; and cultural practices. Roadside vegetation control contributes to safer, more relaxing travel for the motoring public. Controlling this vegetation requires that a variety of methods be used. Each control method has advantages and disadvantages, and no single method can be used for all weed control problems. Integration of the control methods gives the most cost-effective program with the least environmental disturbance. The text is 10 pages and the video is approximately 12 min.

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Module 2: Plant Biology for Roadside Vegetation Managers

Plant biology includes plant types (grasses, broadleaves, woody plants), growth stages, life cycles (annual, biennial, perennial), and conducting tissue (xylem and phloem). Factors influencing plant growth include water, soil, temperature, relative humidity, and light. Plants can be grouped into similar sets for vegetation management purposes. All plants, whether they are grasses, broadleaves, or woody plants, go through similar growth stages and have very specific life cycles. They respond similarly to environmental influences, although some plants may be more adapted to environmental extremes than others. Knowing their biology helps plan effective management programs, be it to suppress or release the plants. The text is 10 pages and the video is approximately 10 min.

Module 3: Characteristics of Chemicals Used for Roadside Vegetation Management

After studying this module, the applicator should (*a*) understand terms used to describe characteristics and actions of herbicides and plant growth regulators and (*b*) know some important characteristics of herbicides and plant growth regulators that determine the use and application of these chemicals for roadside vegetation management. The herbicides and plant growth regulators registered for use on roadside rights-of-way present the opportunity to control almost all plant species or to selectively manage for broad groups of plants. Most programs will use only a small number of the products available. Each product has its unique advantages and problems. The label is the best source of use information. The text is 12 pages and the video is approximately 16 min.

Module 4: Weed Control Programs for Roadside Vegetation Management

The objectives of vegetation management programs can be grouped into nonselective and selective control. Each has its place in roadside vegetation management. Nonselective weed control is the control of all weeds. Selective control means that some plants are released to grow as a result of the treatment method chosen. Nonselective weed control is important around guide rails, median barriers, signposts, delineators, fences, structures (abutments, headwalls, inlets), storage yards, road shoulders, median islands, and ditches. Selective vegetation control is involved in broadleaf weed control, ditches, special grass control, woody plant control, and the use of plant growth regulators. Some parts of the roadside are universally managed to stay free of weeds, and some parts are managed to promote some type of plant cover. Each road managing agency will have specific objectives and programs that reflect the plants and climatic conditions of that locale, budget, available equipment, and public perception of what constitutes acceptable management. The text is 10 pages and the video is approximately 7 min.

Module 5: Application Equipment for Roadside Vegetation Management

This module presents some of the equipment used to make broadcast and directed applications of liquid sprays of herbicides and plant growth regulators on roadsides. Broadcast equipment includes booms with conventional and Raindrop spray tips, boomless spray equipment (off-center tip, straight stream tip, Boom Buster nozzle, Radiarc, Directa-Spra, CDA), and computer injection sprayers. Equipment for directed applications include hand gun, backpack, trigger pump, spot gun, wiping applicator, and Visko-Rhap invert emulsion applicator. Some of the equipment used to apply dry herbicide formulations, such as granules and pellets, is also discussed. The array of equipment ranges from very cheap to very expensive. Each has advantages and disadvantages. Excellent results can be obtained with poor equipment, and poor results with excellent equipment. The text is 12 pages and the video is approximately 9 min.

Module 6: Equipment Calibration for Roadside Vegetation Management

Calibration is the process of measuring and adjusting the amount of chemical a piece of spray equipment will apply to the target area. Proper calibration is essential. After studying this module, one should be able to determine that the correct amount of product is being applied for a variety of application situations and equipment types by being able to determine area, speed, gallons per acre, amount of product to add to each tank, mix percent by volume, altered equipment speed and application rate per acre, amounts for partial mixes, and amounts for granule and pellet application. This module is a workbook with example problems as well as problems to test understanding. The text is 35 pages and the video is approximately 22 min.

Module 7: General Problems Encountered in Chemical Application for Roadside Vegetation Management

That an entire module describes things that can go wrong should make it apparent that an applicator is the person primarily responsible for the success of the vegetation management program. Attention must be paid to changing conditions on the roadside during application. Nothing teaches like experience, but bad experiences with chemicals can change entire weed control programs. After studying this module, the applicator should be able to recognize potential problems related to chemical use, including (*a*) registered for site of application; (*b*) physical barriers and obstructions (highway traffic, obstructions, accessibility, terrain, ditches); (*c*) environmental effects such as brownout, leaching, lateral movement, adjacent water and wells, nonapplicator exposure, off-site vegetation, backflash, and roadside loading and mixing; (*d*) climate/weather-related factors, such as dirt and rain; (*e*) equipment limitations that result in overapplication and pattern variances; and (*f*) plant factors associated with broad spectrum control, no vegetation to release, invasion by other weeds, plant size, timing, and layered vegetation. General environmental concerns include spills, nontarget species, drift, endangered species, groundwater, and surface water contamination. The text is 12 pages and the video is approximately 11 min.

Module 8: Applicator/Operator Safety for Roadside Vegetation Management

Weed control is not without occupational hazards, whether the job is done with chemicals or with mechanical equipment. Study of this module should provide (*a*) an understanding of basic concepts of toxicology as they relate to exposure to herbicides and plant growth regulators, (*b*) awareness of the need for and types of protective clothing related to the use of these products, (*c*) the ability to make a rational decision on the need for protective clothing, (*d*) the ability to deal with emergency exposure occurrences, and (*e*) awareness of other personal safety practices related to mechanical and manual weed control practices. The text is 15 pages and the video is approximately 9 min.

Module 9: Public Relations for Roadside Vegetation Management

Public relations is a personal and professional responsibility. It is essential to the management of issues on the public mind. The most effective public relations programs are always in progress long before there is an apparent need. The techniques of public relations can produce fewer complaints, quicker resolution of conflicts, and improved support for roadside vegetation management. Study of this module should provide knowledge concerning (*a*) the importance of public relations, (*b*) the importance of the applicator in public relations, (*c*) how to inform the public and special interest groups, (*d*) how to deal with the media both reactively and proactively, (*e*) how to deal with complaints, and (*f*) planning for crisis management. The text is 10 pages and the video is approximately 9 min.

MODULE DISTRIBUTION AND USAGE

Two sets were sent to each state, one to the state lead agency responsible for regulation and the other to the pesticide applicator training coordinator. The national distribution of these materials required that they be applicable at the state, regional, and national level. The modular format was designed to encourage states to adopt individual modules that they believe to be particularly applicable. The modular format also makes it easier for states to update training programs. The masters are to be maintained by Purdue University for 5 years.

Since states are encouraged to duplicate and disburse copies of the training materials, it is impossible to know where and how the materials have been used. On the basis of personal communications, it is believed that the materials are being used in a variety of situations. Some state highway departments have placed copies in each regional office. A number of cities, counties, and commercial applicators have purchased the training materials. Several universities have purchased additional sets. The manuals have also been used as the primary training document for the rights-of-way category for some states.

The modules come in a multicolored set with each tape and manual color coordinated in a single binder. The manuals are saddle stitched for easy duplication. The interior print colors of red and black were chosen for duplication clarity. These training materials can be ordered from Agricultural Communication Service, Purdue University, Media Distribution Center, 301 South Second Street, Lafayette, Ind. 47905-1092 (317-494-6794). Tapes and manuals cost $400 per set; sets of manuals only cost $50 per set. A 10 percent discount will be given on orders of 10 or more of either; the discount is 25 percent on orders of 50 or more. Checks should be made payable to Purdue University.

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Postemergence Control of Johnsongrass, Dallis Grass, and Purpletop in Tall Fescue

S. W. BINGHAM, W. J. CHISM, AND P. L. HIPKINS

Selective control of Johnsongrass was evaluated in tall fescue roadsides. Initially, two application techniques were evaluated: spot handgun applications and broadcast boom application using 281 L/ha (30 gal/acre). The selective herbicides showing good promise were fenoxaprop, sethoxydim, and primisulfuron. The standard glyphosate gave 100 percent control of both Johnsongrass and tall fescue (nonselective). Fenoxaprop contains isomers, and a preparation for the more active isomer was evaluated during later experiments (HOE 46360 05H, Hoechst-Roussel Agri-Vet Co.). Fenoxaprop (active isomer), nicosulfuron, and fenoxaprop were effective for Johnsongrass control with acceptable tall fescue quality for roadside cover. Nicosulfuron caused more injury than the fenoxaprop formulations. During 1991, fluazifop was tank mixed with fenoxaprop for excellent Johnsongrass control and caused low tall fescue injury, which resulted in improved turfgrass quality on the roadside. Dallis grass and purpletop were effectively controlled out of tall fescue highway turf with fenoxaprop plus fluazifop and imazethapyr plus imazapyr. Fenoxaprop and imazethapyr alone provided less-than-desirable control of both species.

During the last 10 years, Johnsongrass and Dallis grass have become important weeds along Virginia highways and have continued to increase in severity in recent years. Currently, glyphosate is being widely used in tall fescue and bermudagrass roadsides for control of these weeds. Handgun foliar applications are primarily used, and severe damage on the actively growing turfgrasses has resulted in recurring Johnsongrass and Dallis grass. Thus, the aggressive Johnsongrass and Dallis grass are not completely controlled with current herbicides (*1*). With severe damage to the tall fescue and bermudagrass, regrowth of Johnsongrass and Dallis grass has occurred with little competition.

The timing for herbicide application to provide excellent control of Johnsongrass and Dallis grass has been during June and July in Virginia (*2*) and Texas (*3*). Complete tall fescue control and 90 percent control of bermudagrass is encountered at this time with glyphosate. Even though Johnsongrass is controlled well, new seedlings emerge as well as some new plants from rhizomes that escaped treatment or where rainfall occurred soon after application, reducing effectiveness (*4,5*).

Imazapyr alone and in tank mixtures with other herbicides has provided substantial Johnsongrass control (*5,6*); however, this herbicide has failed to be widely accepted by departments of transportation. Fenoxaprop and sethoxydim were apparently promising for Johnsongrass control and required only a short time (4 hr) between application and rainfall to be efficacious (*4*). The objectives of these studies were to evaluate selective herbicide treatments for Johnsongrass and Dallis grass control while allowing tall fescue to fill in the space to reduce regrowth of weeds from seed or underground structures.

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MATERIALS AND METHODS

Several tests were conducted on Virginia primary and Interstate highway roadsides in tall fescue infested with 25 to 50 percent Johnsongrass or Dallis grass. The herbicides selected for these studies have shown promise for control of Johnsongrass in crop situations and included fenoxaprop, an active isomer of fenoxaprop, nicosulfuron, sethoxydim, fluazifop, primisulfuron, quizalofop, imazethapyr, imazethapyr plus imazapyr, and fenoxaprop plus fluazifop. Glyphosate was used as a standard.

Except for one study, the applications were made with a CO_2 backpack sprayer with a boom providing 281 L/ha (30 gal/acre). One study used a handgun technique to spray to wet the weed foliage. Three to four replications were used in a randomized complete block design with plots 183 by 366 cm (6 by 12 ft) or larger. The data collected included control ratings on a 0 to 10 scale where 0 = no control, 1 to 3 = slight symptoms, 4 to 6 = definite control but generally not acceptable, 7 to 9 = acceptable control to excellent, and 10 = complete control; injury ratings with similar scale; rhizome counts in 30.5 by 30.5 by 15.2 cm (1 ft by 1 ft by 6 in.) deep in soil; percent control; and quality ratings with a 1 to 9 scale where 5 = acceptable, 9 = best, and below 5 = less than acceptable turfgrass quality for roadsides.

RESULTS AND DISCUSSION

Selective Johnsongrass Control

Using the handgun technique to wet the foliage of Johnsongrass was the standard procedure for herbicide application at the department of transportation in Virginia. This technique would be most effective when the weed occurs in clumps and the clumps are scattered widely. Glyphosate was very effective for control of rhizome Johnsongrass; however, the tall fescue

TABLE 1 Johnsongrass Control in Tall Fescue Using Handgun Application

Treatments 7/6/88	Rate[b] g ai/379 L	Control Ratings[a] Johnsongrass Shoots 7/30	9/30	Rhizomes 9/30
Fenoxaprop	181	1.3 c	8.7 b	6.7 b
Sethoxydim + Crop oil con.	340 0.50% v/v	9.3 a	7.3 a	6.3 b
Primisulfuron + X-77	36 0.25% v/v	6.7 b	7.0 c	4.7 c
Glyphosate	2722	9.3 a	10.0 a	10.0 a
Check		1.0 c	0.0 d	0.0 d

[a] Control rating scale: 0 = no control, 1-3 = slight symptoms, 4-6 = definite control but generally not acceptable, 7-9 = acceptable control to excellent, 10 = complete control.
[b] To obtain lb ai/100 gal, multiply g ai by 0.002205, then L by 0.2642.

was completely controlled using 2722 g ai/379 L (6.0 lb ai/100 gal) (Table 1). Fenoxaprop was slightly more effective than sethoxydim and primisulfuron. These herbicides were not completely efficacious; regrowth was apparent even during the same year except for glyphosate, which allowed seedlings to reestablish during the next season.

In the second study, the rate of glyphosate was reduced to 561 g ai/ha (0.5 lb ai/A), which is tolerated by the tall fescue (Table 2). The best rhizome control was by fenoxaprop and sethoxydim, and poor results were obtained with primisulfuron. Sethoxydim caused severe injury to the tall fescue. Fenoxaprop contains isomers, and a more active isomer was evaluated in 1990 and 1991 (Table 3). The active isomer appeared to require only about one-half to three-fourths as much active ingredient where 95 percent control of Johnsongrass was obtained at 2 months after treatment. The second study during 1990 was initiated after the second mowing (August 3), and the results were very poor compared with those of the June application. Nicosulfuron was very effective for Johnsongrass control; however, a definite injury level occurred on tall fescue. This injury may still be acceptable to many managers of highway tall fescue.

During 1991, fluazifop was used to boost the effectiveness of fenoxaprop for Johnsongrass control and provided a high-

TABLE 2 Selective Johnsongrass Control in Highway Tall Fescue Using an Over-the-Top Application

Treatment 7/26/89	Rate[b] g ai/ha	Control Ratings[a] Johnsongrass Shoots 8/24	9/29	Rhizomes[b] Number per 14,158 cm^3	Turf Injury[a]
Fenoxaprop + X-77	140 0.25% v/v	7 a	7 a	4	0
Sethoxydim + Crop oil con.	213 0.50% v/v	9 a	7 a	5	10
Primisulfuron + X-77	22 0.25% v/v	5 ab	7 a	17	5
Glyphosate + X-77	561 0.25% v/v	9 a	7 a	8	3
Check		0 b	0 b	21	0

[a] Control rating or injury scale: 0 = no control or injury, 1-3 = slight discoloration, 4-6 = definite control or injury but generally not acceptable control, 7-9 = acceptable control or unacceptable injury, 10 = complete control or dead turfgrass.
[b] To obtain lb ai/A, multiply g ai/ha by 0.000892. To obtain ft^3, multiply cm^3 by 0.0000353.

TABLE 3 Selective Johnsongrass Control in Highway Tall Fescue Using 281 L/ha (30 gal/acre) Broadcast Sprayer

Treatment Time/Herb.	Rate[a] g ai/ha	Johnsongrass shoot Percent control		Tall fescue ratings Injury[b]	Quality[c]
6/14/90[d]		7/19	8/9	6/27	7/19
Fenoxaprop	294	88 ab	78 abc	2.0 cd	6.7 ab
	392	98 a	60 abc	2.0 cd	7.7 a
Fenoxaprop	140	62 bc	57 abc	1.0 de	5.7 bc
(act. isomer)	280	97 a	95 a	2.0 cd	7.0 ab
Nicosulfuron +	41	93 a	95 ab	3.0 abc	2.3 e
X-77(0.25% v/v)	81	97 a	100 a	4.0 a	2.0 e
Primisulfuron +	35	7 d	3 d	2.7 bc	4.0 cd
X-77(0.25% v/v)	70	50 c	45 cd	3.0 abc	4.3 cd
Check		0 d	7 d	0.0 e	4.3 cd
8/3/90[d]			9/3	9/3	
Fenoxaprop	294		30 e	0.0 f	
	392		43 b-e	0.3 ef	
Fenoxaprop	140		35 de	0.0 f	
(act. isomer)	280		55 abc	0.3 ef	
Nicosulfuron +	41		53 a-d	2.7 bc	
X-77(0.25% v/v)	81		67 a	4.7 a	
Primisulfuron +	35		32 e	0.7 ef	
X-77(0.25% v/v)	70		40 cde	20 cd	
Check			0 f	0.0 f	

[a] To obtain lb ai/A, multiply g ai/ha by 0.000892.
[b] Injury rating scale: 0 = no injury, 1-3 = slight discoloration, 4-6 = definite injury, 7-9 = unacceptable injury, 10 = dead turf.
[c] Quality rating scale was 1-9, where 5 = acceptable, 9 = best, and below 5 = unacceptable quality.
[d] No mowing was done prior to 6/14 and johnsongrass was 30.5 to 61 cm (12 to 24 inches) tall. Test site treated 8/3 was mowed twice before treatment, the second just one week before treatment.

TABLE 4 Selective Johnsongrass Control in Highway Tall Fescue Using Broadcast Sprayer

Treatment Time/Herb.	Rate[a] g ai/ha	Turf ratings Injury[b]	Quality[c]	Johnsongrass shoot Percent control	
6/4/91[d]		6/18	8/2	8/2	9/20
Fenoxaprop	294	0.3 ab	5.3 ab	83 ab	83 abc
	392	1.0 ab	4.3 abc	75 abc	75 abc
Fenoxaprop	140	0.0 b	4.7 abc	90 ab	90 ab
(act. isomer)	280	1.3 ab	5.3 ab	90 ab	90 ab
Fenoxaprop(act.)+	35	0.7 ab	5.7 ab	68 abc	83 abc
Fluazifop	140				
Fenoxaprop(act.)+	70	1.7 a	6.0 a	93 ab	95 ab
Fluazifop	280				
Fluazifop	211	1.7 a	4.3 abc	42 a-d	38 cde
Check		0.0 b	3.7 bc	20 cd	7 de
6/4/91 + 7/5/91[d]		6/18	8/2	8/2	9/20
Fenoxaprop	197+197	0.7 a	6.0 ab	90 a	87 ab
	294+294	0.3 a	7.0 a	100 a	77 ab
Fenoxaprop	140+140	1.0 a	6.3 ab	98 a	97 ab
(act. isomer)	280+280	0.3 a	6.3 ab	98 a	88 ab
Fenoxaprop(act.)+	17+70	1.0 a	6.3 ab	97 a	90 ab
Fluazifop	17+70				
Fenoxaprop(act.)+	35+140	0.7 a	5.3 abc	98 a	93 ab
Fluazifop	35+140				
Fenoxaprop(act.)+	70+280	0.7 a	5.0 bc	100 a	97 ab
Fluazifop	70+280				
Check		0.0 a	4.0 c	3 c	0 c

[a] To obtain lb ai/A, multiply g ai/ha by 0.000892.
[b] Control and injury rating scale: 0 = no control or injury, 1-3 = slight discoloration, 4-6 = definite control or injury but generally not acceptable control, 7-9 = acceptable control or unacceptable injury, 10 = complete control or dead turfgrass.
[c] Quality rating scale was 1-9, where 5 = acceptable, 9 = best, and below 5 = unacceptable quality.
[d] No mowing of either test prior to treatment.

TABLE 5 Selective Dallis Grass and Purpletop Control in Highway Tall Fescue Using Broadcast Sprayer

Treatment Time/Herb.	Rate[a] g ai/ha	Tall fescue Quality[d]		Dallisgrass % Control		Purpletop % Control
6/6/91[b]		7/18	8/22	7/18	8/22	8/22
Fenoxaprop	294	4.0 c	5.0 bc	22 d	7 de	83 a-d
	392	4.7 abc	5.0 bc	33 cd	37 b-e	60 a-e
Fenoxaprop	140	5.0 abc	5.0 bc	67 ab	53 a-e	42 d-e
(act. isomer)	280	4.7 bc	5.0 bc	57 bc	27 b-e	87 a-d
Fenoxaprop(act.)+	35+140	6.0 a	5.0 bc	88 ab	78 ab	65 a-e
Fluazifop	70+280	6.0 a	5.0 bc	97 a	63 a-d	93 ab
Fenoxaprop(act.)+	41+140		5.7 a		95 a	93 ab
Fluazifop	81+280		5.7 a		98 a	100 a
Nicosulfuron	62	5.3 ab	5.0 bc	85 ab	30 b-e	75 a-d
(X-77 0.25% v/v)	81	5.3 ab	5.0 bc	82 ab	35 b-e	48 b-e
Imazethapyr	70	4.0 c	5.0 bc	32 cd	28 b-e	58 a-e
(X-77 0.25% v/v)	140	4.3 bc	5.0 bc	22 d	45 a-e	88 a-c
Imazethapyr +						
Imazapyr	70+9		5.0 bc		83 ab	97 a
(X-77 0.25% v/v)	140+9		5.3 ab		83 ab	100 a
Primisulfuron	62	4.3 bc	5.0 bc	13 d	32 b-e	50 b-e
(X-77 0.25% v/v)	70	4.0 c	5.0 bc	17 d	67 abc	27 e
Quizalofop	35	4.0 c	5.0 bc	8 d	32 b-e	62 a-e
	70	4.0 c	5.0 bc	20 d	20 cde	70 a-e
Check		4.0 c	4.7 c	5 d	58 b-e	42 de
6/25/91[b]		7/24 Injury[c]	8/7 Quality[d]	8/7 %Control		
Fenoxaprop	294	0.3 de	3.7 abc	23 e-h		
	392	0.0 e	3.3 abc	25 e-h		
Fenoxaprop	140	1.0 cde	3.7 abc	27 d-h		
(act. isomer)	280	0.7 cde	4.3 ab	68 abc		
Fenoxaprop(act.)+	41+140	0.3 de	4.3 ab	70 abc		
Fluazifop	81+280	0.3 de	4.0 ab	95 a		
Nicosulfuron	62	5.0 a	3.0 bc	88 ab		
(X-77 0.25% v/v)	81	23 bc	3.0 bc	62 abc		
Imazethapyr	70	1.0 cde	3.7 abc	43 c-f		
(X-77 0.25% v/v)	140	2.0 bcd	3.3 abc	65 abc		
Primisulfuron	56	0.0 e	3.7 abc	12 fgh		
(X-77 0.25% v/v)	70	0.7 cde	3.3 abc	17 fgh		
Quizalofop	35	2.0 bcd	4.7 a	57 b-e		
(X-77 0.25% v/v)	70	3.7 ab	3.3 abc	53 cde		
Check		0.0 e	3.3 abc	0 h		

[a] To obtain lb ai/A, multiply g ai/ha by 0.000892.
[b] No mowing of either test prior to treatment.
[c] Injury rating scale: 0 = no injury, 1-3 = slight discoloration, 4-6 = definite injury, 7-9 = unacceptable injury, 10 = dead turfgrass.
[d] Quality rating scale was 1-9, where 5 = acceptable, 9 = best, and below 5 = unacceptable quality.

quality tall fescue (Table 4). Very little injury was encountered with the tank mixture of fenoxaprop and fluazifop. Repeat applications of the tank mixture were very effective and allowed reduced rates of fenoxaprop and more effective control than fluazifop alone.

Selective Dallis Grass and Purpletop Control

Dallis grass control in tall fescue was obtained with fenoxaprop plus fluazifop and imazethapyr plus imazapyr (Table 5). Fenoxaprop and imazethapyr alone were not effective on Dallis grass. A more active isomer of fenoxaprop provided some control early; however, this isomer appeared to control shoots for a short time, and regrowth from crowns was apparent. The mixture of the active isomer of fenoxaprop at one-fourth the rate with fluazifop appeared to provide a synergistic response to reach up to 98 percent control of Dallis grass.

Nicosulfuron gave initial shoot control of Dallis grass; however, regrowth was apparent after 75 days. Nicosulfuron, the high rate of imazethapyr, and quizalofop cause significant injury to the tall fescue. However, this injury was temporary and may be acceptable in management of tall fescue. Thus, the Dallis grass control was acceptable with fenoxaprop (active isomer) plus fluazifop and imazethapyr plus imazapyr, while some improvement was obtained in tall fescue highway turf quality.

Purpletop was controlled in tall fescue with fenoxaprop plus fluazifop and imazethapyr plus imazapyr. The results were variable with a trend toward good control with fenoxaprop or imazethapyr alone.

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Ohio Native Wildflower Seed Nursery

ROBERT E. TATMAN

The Ohio Department of Transportation (ODOT) started a roadside wildflower program in 1984. Seed sources for Ohio native wildflowers did not exist. After attempts to interest the private sector in developing a native wildflower nursery failed, ODOT entered into agreement with the Park District of Dayton and Montgomery County to develop such a nursery. Both agencies developed criteria to proceed with the nursery as a research effort. Much emphasis was placed on record keeping and testing every phase of development. Currently the Ohio Native Seed Nursery is producing approximately 250 lb of seed each year consisting of eight different species. Field test of seed has resulted in satisfactory germination and establishment in test plots. ODOT plans to continue the development of the Native Seed Nursery, at the same time encouraging private seed growers to develop nurseries of their own.

During fall 1984, the Ohio Department of Transportation (ODOT) initiated a roadside wildflower program. ODOT received much public support for the efforts; however, one area of concern was expressed. There was no source of Ohio native wildflower seed available in sufficient quantity.

ODOT requested in 1987 that the Ohio Department of Natural Resources (ODNR) explore the possibility of a joint program to produce Ohio native wildflower seed for use by both agencies. ODNR did not have the resources for a program at that time and suggested that contact be made with the various organizations interested in establishment of native wildflower areas.

In June 1988 ODOT entered into an agreement with the Dayton–Montgomery County Park District to establish a production nursery for wildflower seed. This was a first for Ohio, and much new ground had to be broken. Wildflower seed production is a competitive industry, so nurseries already in business were reluctant to share technical information. However, the Department of Natural Resources in Wisconsin had a native seed nursery and was very helpful in the early planning days.

Whenever a new venture such as this is begun, certain resources must be available (i.e., labor, equipment, and material). Skilled labor, specialized equipment, and ideal growing conditions are required. ODOT and the Park District addressed these issues early in the program to ensure eventual success.

The year the nursery was started followed the driest year on record for Ohio. Seedling wildflower plants must have water, and if the natural rainfall was lacking, the efforts would fail. To ensure adequate moisture, ODOT constructed an irrigation pond on the site, and the Park District set up an irrigation system before the first seed was planted.

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The pond construction was carried out concurrently with the site preparation for the nursery itself. The nursery was to be located in an old abandoned field, which had become overgrown with small brush and weeds. Mowing and the selective use of herbicides soon had the site ready for further preparation.

It was decided that, for maintenance reasons, the nursery would be laid out in strips 4 ft wide. The strips were rototilled, and grass, which could be mowed, was left between the planting beds. The nursery was now ready for seed planting. However, before native wildflower seed can be produced, a native seed source must be found. Fortunately the Park District had volunteer persons knowledgeable in the collection of wildflower seed. The selection of wildflowers for roadside and Park District use was based on several criteria. Visibility, color, growing habit, and availability were the primary reasons for selecting a plant type for harvesting. The volunteers collected the initial seed stock from locations all over Ohio and kept detailed records on this process. Enough seed was collected, by hand, to start the nursery.

Plants that could be useful on certain special areas, such as the shale cuts in southeastern Ohio, were collected. During fall 1989, an annual wildflower growing on the shale cuts had been observed. It belonged to the Asteraceae family (*Bidens polylepis*), and if it could be grown successfully, it would not only be an attractive flower but also would probably survive after planting on shale cuts. One lb of the Bidens seed was harvested for use in the nursery.

The next technical question to be addressed was how to break dormancy of the collected seed. Reports indicated a wide range in wildflower seed dormancy, which initially led us to believe that some problems in germination would exist.

Several techniques for breaking dormancy were tested: cold-dry, cold-wet in vermiculite, and cold-wet in flats with planting soil. The seed was subjected to the various treatments and observations were recorded. It was found that cold storage over winter in flats provided adequate germination.

The first problem encountered after initial seed planting was weed intrusion into the planting beds. Once the old vegetation cover was removed, the weed seed already present in the soil on the site quickly germinated, and weeds proliferated. Because of this, many of the seedbeds were failures that first spring.

Because of the weed problem, it was decided to use plants instead of seed to establish the planting beds. The herbicides Round-up and Surflan provided a virtually weed-free site. A small greenhouse was built, which provided all the plant material needed to fill the nursery. At planting time, volunteers were used once again to transplant the seedlings.

The production of seedlings for transplanting also underwent a series of experimental procedures, much the same as that occurring with the seed dormancy problem. Wildflowers have, in many cases, a well-developed root system. Perennials, in particular, develop the root system before much top growth takes place. We found that because of this growth habit, some plants became root bound in the planting trays before they could be transplanted.

It was fairly easy to solve this problem. Seed was sown in the planting trays and after germination was transplanted to growing tubes. This allowed plenty of room for root growth and also made transplanting easy. The success rate for the transplants in the nursery improved dramatically as a result.

Table 1 gives the seed harvest totals from the nursery for 1990. Nearly 200 lb of seed was harvested on less than ⅓ acre of the cultivated area. We were pleased with the total seed harvest, especially since this was the first year of seed production from the perennials.

With harvest time came the next set of problems: how to pick, clean, and store the seed. The problem in connection with harvesting the seed of wildflowers was compounded by the fact that seed developed in different stages and varied in height, density, and ability of the plant to hold the seed without shattering. In some plants, like the *Bidens*, seed ripened almost overnight and fell from the plant. Other species such as purple coneflower ripened gradually and then held the mature seed for an indefinite time before it fell from the plant. This required that the nursery manager keep a close watch on the plants by monitoring progress to avoid loss of seed by shattering.

Several methods of picking the seed were tried, including handpicking and use of a vacuum and hand-held gas-powered harvester. The preferred method has not been determined. It is hoped that less labor-intensive methods can be found.

The method used for cleaning the harvested seed was fairly successful. After the seed heads had dried, they were processed through a shredder. The product of the shredder was then sent through a fan mill. The finished product, although not commercially clean, was clean enough to pass through the planting equipment much of the time.

Seed will not be stored after the harvest if it is at all feasible to carry out planting. Thus we will not be required to provide cold, vermin-proof storage over winter. This should work well, since the natural planting period for many plants in Ohio is late fall or early winter.

Since the Ohio Native Wildflower Nursery is research oriented, it was decided to find out as much as possible about the quality of the seed produced. Several of the species harvested were selected to test for percent of viable seed. Samples were collected and sent to a commercial seed-testing laboratory, where they were tested for viable seed using the Tetrazolium method of determination. Table 2 gives the results of these tests. They indicated that much of the seed lots were of a good quality.

Other research data have been compiled concerning plant height, color, soil preference, bloom period, and planting requirements. This information has been placed on charts and will be made available for use by our field crews (Table 3).

Another report showing groups of wildflowers to be planted together in specific soil types has also been developed (see Table 4). This should also greatly assist the field crews at planting time.

Comprehensive data about each wildflower variety are compiled as information becomes available from the nursery. This information will be maintained at a central data base and updated as observations are made.

The 1990–1991 seed harvest has been planted along Ohio's roadsides and throughout the Dayton–Montgomery County Park District. ODOT and Park District staff conducted field reviews of the wildflower plots during the first half of 1992. All plots showed a very acceptable germination rate.

In this paper two plots will be described. Plot A is located in northern Ohio in Lorain County. The soil in this area is largely shale and has a low pH. Past efforts by ODOT to establish vegetation on this site have not been successful.

The soil was lightly raked and hand seeded to the *Bidens polylepis* at a rate of approximately 10 lb/acre. No further site treatment was performed. Observations of this site determined that there was an extremely high germination rate, and a solid mass of yellow flowers was reported at bloom time.

TABLE 1 Seed Harvest, 1990

FORBES	HARVESTABLE SQ. FT.	WEIGHT OF SEED
Bergamot	100 sqft	2 lbs
Bur-Marigold	1800 sqft	60 lbs
Blackeyed Susan	400 sqft	1.25lbs
Greyheaded coneflower	300 sqft	16 lbs
Liatris	1100 sqft	12.25 lbs
New England Aster	300 sqft	18.75 lbs
Nodding Wild Onion	200 sqft	6.5 oz
Prairie Drop Seed	100 sqft	2.6 lbs
Purple Coneflower	1300 sqft	25.75 lbs
Orange Coneflower	400 sqft	2.37 lbs
Oxeye	700 sqft	9.3 lbs
Stiff Goldenrod	1600 sqft	28.75 lbs
Whorled rosinweed	200 sqft	10.4 oz.

TABLE 2 Seed Test Results (Test Performed by Seed Technology, Inc.)

Kind	Percent germination
Bur-Marigold	80
Oxeye	90
Orange Coneflower	87
Purple Coneflower	70
Liatris	88
Bergamot	55
Grey-Headed Coneflower	91

Note: Testing with Tetrazolium (Tz) is based on the principle that respiration processes within living tissues release hydrogen, which combines with the colorless Tetrazolium solution and produces a red pigment. Strong, healthy tissues develop a normal red strain. The Tz Test is especially useful in evaluating dormant seed at harvest. It was for this reason that this test was chosen over conventional germination tests for our wildflower seed.

TABLE 3 Earliest Bloom to Latest Bloom (Harvested Fall 1990)

Botanical Name	Common Name	Height (ft)	Flower Color	Soil Type	Bloom Period
Rudbeckia hirta	Blackeyed susan	1–3	Yellow	Mesic-dry	June–October
Ratibida pinata	Greyheaded cone	3–5	Yellow	Mesic-dry	June–September
Monarda fistulosa	Bergamot	2–4	Lavender	Mesic-dry	June–September
Echinacea purpurea	Purple coneflower	2–3	Reddish-purple	Mesic-dry	June–October
Heliopsis heliauthoides	Oxeye	2–5	Yellow	Mesic-dry	July–August
Allium cernuum	Nodding wild onion	1–2	White	Mesic-dry	July–August
Liatris spicata	Blazing star	2–5	Rose-purple	Wet-mesic	July–September
Rudbeckia fulgida	Orange coneflower	1–3	Orange-yellow	Mesic	August–October
Aster novae-angliae	New England aster	3–7	Violet-rose	Wet, mesic-dry	August–October
Bidens polylepis	Bur-marigold	1–3	Yellow	Wet-mesic	August–October
Solidago rigida	Stiff goldenrod	2–5	Yellow	Wet-dry	August–October
Sporobolis heterolepis	Prairie dropseed	1½–3½	Tan	Mesic-dry	August–October

TABLE 4 Seed Distributed to ODOT, 1991 (Site Selection Based on Soils of Southwestern Ohio)

Soil Type	Seed
Wet	Bur-Marigold Liatris New England Aster Stiff Goldenrod
Mesic	Bergamot Blackeyed Susan Bur-Marigold Greyheaded Coneflower Liatris-Blazing Star New England Aster Orange Coneflower Oxeye Prairie Dropseed Purple Coneflower Stiff Goldenrod Whorled Rosinweed Nodding Wild Onion
Dry	Bergamot Prairie Dropseed Blackeyed Susan Grey-headed coneflower New England Aster Oxeye Purple coneflower

Plot B is located in southeastern Ohio in Athens County. The site was vegetated with Kentucky 31 fescue and various other plant types. ODOT crews sprayed the site with Roundup and then, approximately 10 days later, mowed the treated grass as close as possible. A disc was lightly pulled over the planting area, and the following native wildflower seed was planted: purple coneflower, grey-headed coneflower, oxeye, liatris, nodding onion, and stiff goldenrod.

All species planted have shown satisfactory germination. Since the site was laid out in strips, it will be easy to continue the review process into the next growing season, at which time the plants should be in bloom.

We believe that the Ohio Native Wildflower Seed Nursery has been successful. Since the nursery can only produce a small amount of the seed needed for the Park District and ODOT needs, we plan to carefully select future planting sites. Areas close to nature preserves, rest areas, and, in the case of the *Bidens*, critical erosion sites will be chosen as first priority. It is hoped that our success will encourage private growers to become interested in growing native wildflowers for commercial use.

Our agreement with the Dayton–Montgomery County Park District expires in June 1993. What will be the future of the Ohio Native Wildflower Nursery? At this time, we have every reason to believe that an extension of the program will be approved. Certainly ODOT, the Park District, and the people of Ohio have much to gain by the continued success of this program.

LITERATURE SEARCH

An extensive literature search was conducted through ODOT library services. TRIS and DIALOG computer searches did not locate published data on growing wildflowers commercially in Ohio.

The purpose of this study was to investigate the possibilities of growing wildflowers in Ohio in commercial quantities. We realize that similar work may have been done in other states. However, their data were not used as a reference because of Ohio's differences in geology, climate, and so forth, which could affect growing procedures in Ohio.

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Correction and Repair of Road Edge Scour for Grassed Shoulders on Parkways

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The problem of road edge scour on grassed shoulders is discussed. The focus is on the edge scour problem located in the National Park Service units, especially the roads making up the category of parkways. The literature pertaining directly to edge scour is limited to only a few items. In contrast, the literature that addresses the roadside landscape is considerable. Many of the findings came about in the 1930s. The origin of the parkways in the 1920s was the basis for many of the design philosophies followed today. Although design standards and maintenance practices have been refined, edge scour still occurs. Edge scour is most prevalent in parkways that were designed in the 1930s and that have not been improved to accommodate today's traffic conditions. With an almost 50-year construction cycle, many parkways and similar park and recreational roads exhibit the problem. To illustrate the current problem and current maintenance practices in the national park system, maintenance practices and the extent and treatment of edge scour are described for several national parks. The description includes the type of study zone—tropical, low altitude temperature, eastern high altitude, and the western high altitude. These four zones form the basis for site-specific considerations and recommendations.

Road edge scour damage is the erosion of unpaved turf shoulders and adjacent roadsides caused by vehicle traffic. It is characterized by destruction of the vegetative cover, rutting of the shoulder, and development of a turf dike. If it is not corrected, it may be followed by fractures and failure of the pavement edge. Edge scour damage is primarily associated with older roadways that are experiencing an increased volume of longer, wider, and heavier vehicles. These roads are most frequently those that have not been updated to reflect modern design standards.

Edge scour damage was extensively studied in the 1950s. There is, however, an increase in observed damage. Also there have been advances in design standards, development of new products and techniques, and a large volume of research concerning vegetation selection, establishment, and maintenance. All of these factors may contribute to the correction or prevention of edge scour damage. This paper focuses on the edge scour problem located within the national park system, specifically, the "parkway" (*1*).

HISTORICAL OVERVIEW

The first recreational parkway, the Bronx River Parkway, was completed in 1923. Skyline Drive, completed in 1934, became the first parkway to be incorporated into the national park system (*2*).

The earliest studies concerned with the performance of turf shoulders appeared in the 1950s. Dubois (*3*) discussed results of extensive tests on rutting of turf shoulders. He concluded that properly stabilized shoulders with adequate load-bearing capacity served to resist rutting on sites with moderate vehicle use. In such cases it was possible to establish turf on the stabilized shoulder, and the turf itself helped resist rutting (*4*).

Brant (*5*) identified the difference between stabilized turf shoulders and turf shoulders as the ability of stabilized turf shoulders to carry the weight of a vehicle. He further identified the potential for use of stabilized turf shoulders as a satisfactory, economical alternative to other types of shoulder, providing the shoulder is properly compacted and vehicle use is moderate.

Buchanan (*6*) discussed design and construction of stabilized turf shoulders along the Natchez Trace Parkway. The project's goal was a satisfactory compromise between shoulder stabilization and establishment of turf coverage. Turf was successfully established on a stabilized shoulder able to accommodate the weight of passenger vehicles without rutting, even after heavy rain.

Edge scour damage and associated problems are visually unappealing and may be a safety problem. For example, Vance (*7*) indicates that the greatest number of state liability cases concerning highway shoulders occur because a shoulder has not been brought up level with the resurfaced pavement. However, the small shoulder width and associated edge scour condition exhibited by some parkways appeared not to increase the frequency of accidents.

Newton (*2*) describes the early historical development of recreational parkways. The first parkways were designed to be extremely pleasant to drive and yet to be functionally efficient for commuting traffic. Manning (*8*) describes the intrinsic beauty of these parkways.

Skyline Drive is a highway built in this early scenic parkway tradition. The design standards for this parkway were developed to accommodate traffic moving at approximately 40 mph and presented a new set of design and construction issues.

The 1930s technical information was concerned with the roadside. Waugh (*9*) described special landscape ecological characteristics of roadways. He noted nine basic physical zones in a cross section of highway from the centerline to the surrounding countryside. He described the interaction of vehicle disturbance with vegetation and the resulting zonation of vegetation near the pavement and on the unpaved shoulder.

The distribution of vegetation along the roadside was affected by regional differences. In 1936, Waugh continued his

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roadside landscape ecological descriptions with an article concerning Californian roadside ecology (*10*). He noted that the drier western landscape did not contain the striking zonation of the northeastern American landscapes. Boddy and Taylor (*11*) wrote one of the earliest published descriptions of landscape architectural specifications for roadside improvement.

By the 1940s, there appeared articles addressing specialized roadside applications. Curtiss (*12*) wrote about roadside concerns in national forests. Bell (*13*) described roadside standards for western scenic areas. Dupre (*14*) reviewed accomplishments and progress of roadside development in Ohio. After the 1930s and early 1940s, the literature addressing roadside issues decreased in frequency (*15*).

In 1966, the Department of Commerce published a booklet describing a program for scenic highways (*16*). Pragnell (*17*) authored a report concerning scenic roads in forested lands.

The Highway Research Board (*4*) described a research experiment where traffic disturbances were tested on a turf shoulder. The study noted that after 32 passes with a load bearing 3-ton dump truck, points in the shoulder had deflections ranging from 0.5 to 0.875 in. Dunbrook (*18*) indicates that turf shoulders require regrading once every 3 to 4 years. He also found the most effective method to repair the shoulder was to trench out the shoulder and then fill the shoulder with pit-run or pressed gravel with an asphalt cutback or emulsion. The filled trench must be rolled and cured. Power et al. (*19*) discuss cost-effective methods to repair shoulders through reshaping without adding material, reshaping with material added, and pavement widening in selected locations.

Traffic lane design, roadway alignment, and paved shoulder width are other design issues that may alleviate or aggravate the edge scour condition. Gericke and Walton (*20*) indicate that wider shoulders are required to accommodate longer vehicles.

By 1980 standards, the effects of edge scour are diminished by designing new roads with wide lanes and road alignments that keep fast and large moving vehicles in the center of the lane. However, on national park roads and older roads that are designed using different alignment criteria, and where the existing parkway alignment and roadside character are of national historic value, the highway cannot be readily improved to 1980 standards. These national park roads exhibit the effects of edge scour.

From its modern inception in the mid-1920s and early 1930s, roadside development has progressed from the application of many impractical vegetation treatments to the use of proved standard implementation and maintenance practices. In the future, practical site-specific vegetation treatments and maintenance practices may be further refined.

Considering the large area of American landscape devoted to the roadside, the volume of journal literature specific to vegetation treatments and shoulder conditions is relatively small.

DEVELOPMENT OF EDGE SCOUR DAMAGE

Edge scour damage is caused by vehicles traversing the unpaved shoulder surface. The cause of this vehicle movement varies from intentional pulloff to vehicle wandering as a driver responds to road conditions.

A substantial portion of the edge scour damage is caused by large, heavy vehicles such as recreational vehicles (RVs). An increased number of vacationers are taking large vehicles on parkways designed for smaller, slower vehicles. These large vehicles are often not well suited to the roadway design of the 1930s. Also, there is a high presence of older drivers.

The problem of edge scour is associated with the disturbance of the vegetated highway shoulder by vehicle tires, which destroy the vegetation and contribute to the development of ruts within the shoulder. Repetitive vehicle damage deepens the ruts. Shoulder material may be pushed toward the roadside and eventually enough material may be collected to form a turf dike. This may restrict proper drainage, causing the shoulder to become wet and soft and aggravating the damage. Eventually, ruts may become deep enough to allow vehicle axles to drag on the pavement edge, causing fractures or destruction of the pavement. This in turn can be aggravated by moisture. This rutted shoulder condition, with or without pavement damage, is defined as road edge scour. Edge scour damage tends to occur in the same location repeatedly. Figure 1 shows the physical characteristics of edge scour.

The situation reflects a more extensive problem than simply edge scour and rutting on National Park Service roads. The edge scour damage and rutting arise from the vehicle tire leaving the roadway and traversing the grassed area. A number of repetitive tire passes over the same point kills the grass. Repetitive passes also compress the soil and with the presence of moisture can result in rutting. The grass and soil are unable to support the vehicle tire loads with moisture present.

The edge scour problem is a conflict between old roadway design standards and modern roadway use. The issue has not

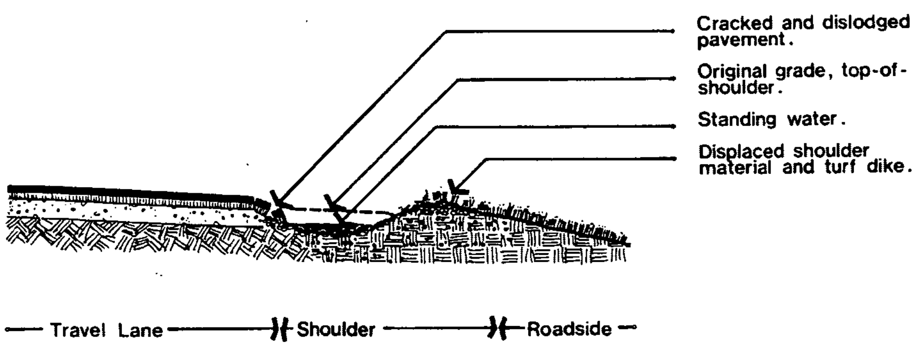

FIGURE 1 Typical edge scour section.

been extensively studied. With the exception of older, intentionally, and historically preserved parkways, the problem is easily solved by improving the roadway to match modern performance criteria.

DESIGN STANDARDS

From the period starting with the early development of the parkway system to the 1980s, design standards were constantly improved and revised. Thus, the design standards of the 1930s were quite different from the design standards exemplified by the Federal Highway Administration (*21*). The design standards of the 1980s allow for highways to accommodate greater vehicle speeds, greater traffic volumes, and greater vehicle sizes.

ROADWAY DEVELOPMENTS

Complementing the changes in general roadway standards, other issues concerning roadway development were addressed from the 1960s through the 1980s. These issues include vegetation selection and maintenance research, erosion control research, vegetation preservation, and vegetation prescriptions.

Vegetation Selection and Maintenance Research

Interest in roadside vegetation treatment and maintenance has led to improved methods in revegetating the roadside. In the 1930s, roadside design transformed from casual landscaping attempts to install roadside vegetation. These solutions emphasized low maintenance treatments that could endure the harsh conditions of the roadside landscape. In time, roadside maintenance methods were developed to manage these landscape treatments (*22*).

It was not until the late 1960s and early 1970s that literature concerning turf selection and vegetation maintenance was well documented. In 1968, White and Bailey (*23*) documented issues associated with maintenance equipment and the management of roadside turf grasses. From this work, Smithberg and White published reports describing recommendations for roadside turf methods and materials (*24,25*).

Erosion Control and Slope Stabilization Research

Vegetation (softscape material) and pavement (hardscape material) have been recently combined in various grid patterns to create a durable surface with the aesthetic features of vegetation. Several of these products are commercially available. These erosion control and slope stabilization techniques may have useful applications in reducing edge scour.

Vegetation Preservation

Preservation of existing stands of vegetation has been given some attention in the literature. Presently, vegetation preservation along roadsides has been primarily concerned with protecting woody plants and may have little application in reducing edge scour.

Vegetation Prescriptions

General roadside vegetation treatments may not always be appropriate for some roadside conditions. Therefore, site-specific vegetation prescriptions have been prepared and implemented. With increased understanding of native vegetation associations, these relationships have been applied to roadside vegetation prescription situations.

Each prescription is confined to the landscape ecological setting of the region. Bailey (*26*) describes the basic biological regions in the United States. Within each region a basic set of native biological associations exists. These biological associations make up the regional plant material palette.

These approaches are considered prescription approaches because they prescribe native plant materials to specific environmental conditions such as wetlands or xeric landscapes. Further developments and research in site-specific roadside prescriptions may assist in reducing localized edge scour damage.

CURRENT PRACTICES IN THE NATIONAL PARK SYSTEM

To study the current practices in the national park system, the park system was divided into study zones. The study zones are based on the concept that the national park system units can be classified according to edge scour characteristics. The parks can be divided into four basic types. The first type is the tropical national park (i.e., parks with tropical vegetation). During rainy seasons, the vegetation can grow very quickly and revegetate disturbed ground. The second type is the low altitude temperature zone. In this zone, vegetation grows at a moderate pace. The third zone is the eastern high altitude zone. Vegetation is restricted by cold stress. Vegetation of the fourth zone, the western high altitude zone, is restricted by cold and low moisture stress.

Parks and parkways in the four zones were visited in 1987 and 1988. Roadsides were examined for edge scour symptoms and the extent of edge scour. Discussions were held with maintenance personnel to obtain their perception of the issue, and current practices to correct edge scour were documented.

Tropical Zone: Everglades National Park

Edge scour is present. The most prominent scouring occurs on the outside edge of some roadway curves and at intersections of roadways. The edge scour disturbance occurs annually in the same locations.

Edge scour can also occur on roadway horizontal tangent lines near sign locations. Drivers may inadvertently let their vehicles drift as they read the sign. Edge scour is associated with both informational directional signs and with posted speed limit signs that require a reduction of vehicle speed.

The repair technique is to fill edge scour ruts with marl material. This material is obtained from a local crusher operation that produces excess material ranging in size from 3/

4 in. to fines. The ruts that occur associated with edge scour are filled in before the ruts get deeper than 3 in. Often the material is not compacted during installation. If compaction is performed, maintenance truck wheels are used to compact the marl. The marl is not reseeded. Existing nearby grass, next to the disturbed area, quickly covers the marl during the rainy summer season.

Placing sod over the marl fill can be effective. Sod is often torn apart by vehicle wheels before the sod can be established.

The color of the marl material is white and contrasts noticeably with undisturbed shoulder material. This contrast can be aesthetically unappealing. However, there is no evidence that the public considers these repairs unsightly.

As edge scour develops and ruts occur, the removed shoulder material is deposited further from the centerline of the roadway and creates a raised shoulder condition. This raised shoulder is removed approximately every 3 years.

Several alternatives have been explored concerning the repair of consistently recurring edge scour locations. One approach is to use concrete blocks (turfstone). The blocks have been effective in reducing edge scour. Grass cover in these blocks has been good. The color of these treated areas is noticeably different from that of existing turf shoulders.

Another approach has been to widen the paved road surface with a 1-ft-wide linear patch. This patch can be visually unappealing.

The Everglades National Park experiences occasional off-road informal use of vehicles. Damage to shoulders appears minimal because the use occurs during the dry winter season, and the shoulders are relatively hard and resistant during this period.

Low Altitude Temperature Zone and Eastern High Altitude Zone: Blue Ridge Parkway, Great Smoky Mountains National Park, and Shenandoah National Park

These parks are in the eastern high altitude zone with portions in the low altitude temperature zone. Edge scour is present. In some locations, the presence is extensive. Both inside and outside curve edges exhibit scour damage. Often the vertical curve alignment and the superelevation geometrics determine the location of the scour. Curves that throw the vehicle to the outside will have outside radius damage. Some curves throw the vehicle into the center of the curve. These curves will have inside damage.

In addition, shoulders near bridges exhibit edge scour. Edge scour near bridges occurs where the bridge lane width is greater than the nonbridge lane width. Once the vehicle is on the bridge, the vehicle may drift as the driver adjusts to the wider lane width. Once past the bridge, the vehicle may not be in a position to traverse the pavement properly. Before the driver can correct the vehicle, the vehicle may traverse the shoulder, resulting in edge scour.

Edge scour ruts often appear substantial. These ruts may be greater than 18 in. wide and greater than 8 in. deep. The large width of the rut is caused by trailer or motor home dual tires drifting off the pavement. In addition, the shoulder may be wet and therefore soft. The frequent impact of the tires can cause a deep rut. Once the rut has substantial depth, the vehicle's axle prevents further depth penetration; however, the dragging axle then begins deteriorating the edge of the pavement (Figure 2). Numerous edge scour locations exhibit pavement edge deterioration from fatigue cracking. The deep ruts also collect more moisture and aggravate the problem.

There is concern that these edge scour ruts may lead to vehicle accidents and park liability. At present, no identifiable claims have been made.

Edge scour repair is conducted by placing a 50 percent loam and 50 percent aggregate mix into the edge scour rut. The material will then be seeded. Repairs are made during the spring and late fall. Edge scour repairs made during the summer can pose a danger to maintenance crews because of the number of traffic conflicts and heavy visitation.

Turfstone has been used in some locations. However, on soft shoulders the turfstone can be ineffective. The turfstone is forced down into the soil and is often demolished. On shoulders with ample support and with only occasional edge scouring, the turfstone has been effective in reducing damage. Several park personnel mentioned that the turfstone does not blend visually with existing grassed shoulders.

Rolled asphalt curing has been used on roadways near metropolitan areas or for drainage. Near metropolitan areas, traffic volumes can be very high, vehicle loads can also be high, and traffic speeds are often at their legal maximum. This type of edge scour treatment has been very effective. However, this

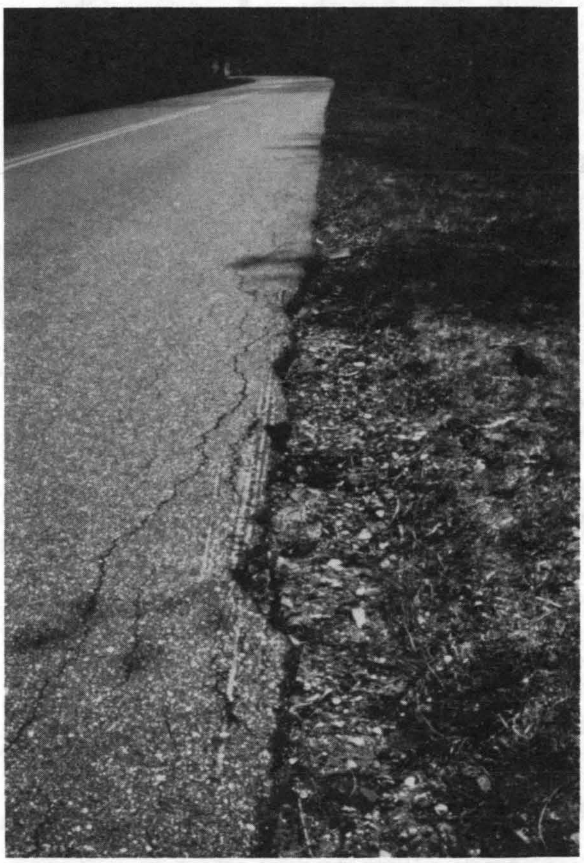

FIGURE 2 Extreme edge scour conditions, Blue Ridge Parkway.

treatment requires additional drain inlets and subsurface drainage. To reduce the need for drainage structures, split-block curbing has been suggested. Although curbing will not prevent informal off-road use, it seems to be effective in reducing vehicles drifting past the pavement.

Turf diking is also a problem that can aggravate edge scour by softening the shoulder. Shoulder accumulations are scraped off every 5 to 7 years.

In these national parks and parkways, the design standards were developed in the 1930s. The design speed, expected vehicle loads, and expected traffic volumes are different from what the parks are experiencing today. With the development of the RV, vacationers are taking large vehicles on parkways designed for leisurely experiences. These large vehicles are often not well suited to the design standards of the 1930s. Roadway geometrics and RV driver habits may contribute to edge scour development. With oncoming traffic, the RV driver will reposition the vehicle such that runoff on the shoulder is common.

Western High Altitude Zone: Rocky Mountain National Park

Edge scour was not present. Several conditions may contribute to the absence of edge scour. First, many of the roadways are edged with rolled bituminous curbing. The curbing can prevent vehicles from drifting off the pavement. Also, continuous paving from the pavement edge to the paved ditch has been used. This means that the roadside vegetation begins on the nontraffic side of the ditch. Vehicles and vegetation rarely come in contact with a continuous paving roadside treatment. Thus there is no edge scour problem.

Second, because of infrequent rainfall, gravelly substrate may not contain high moisture levels during periods of high use, and the shoulder may have a structurally high bearing capacity.

Third, the design standards for some of these roads are similar to modern 1980 standards. These modern standards can assist the vehicle driver in preventing vehicles from drifting off the pavement.

In addition, groundplain vegetation is often present in low surface coverage levels. Thus, it is natural to see shoulders and adjacent landscapes with little or no groundplain vegetation. Repair of shoulders in a visually sensitive manner can often be easily accomplished.

Revegetation recover rates can be extremely slow in alpine tundra life zones. Even the plant growth rates in the lower elevations of the lodgepole pine and ponderosa pine forests can be slow. However, the careful placement of substrate, boulders, and natural groundplain mulches and debris can visually integrate the shoulder with the parkland.

SUMMARY

The scientific literature pertaining directly to edge scour is limited to only several articles. The literature addressing the roadside landscape is considerably larger. Modern roadside development was initiated with the construction of parkways in the 1920s and 1930s.

Edge scour is more prevalent in 1930 parkways that have not been improved to accommodate 1980 traffic conditions. Numerous corrective measures have been implemented to reduce edge scour. Typical corrective measures include allowing the shoulder's turf to recover in the off season, shoulder regrading, shoulder paving, shoulder and ditch paving, installing turf-concrete matrixes, and curbing.

Design recommendations to correct edge scour conditions are site specific as indicated in the discussion of the site visits.

All of these treatments that are nonstructural have not been successful or have visual trade-offs. The presence of shoulder edge scour is common. In the site visits, observations of adjacent state and county highways and discussion with park maintenance personnel indicate that edge scour occurred on these roads also.

Shoulder edge drop-off/depressions have been a major reason for state liability suits, but this has not been experienced yet in the national parks. Two reasons probably account for this. One is the lower speed, especially in the peak visitation periods. The other is the presence of the roadside clear zone adjacent to the pavement, which probably causes the drivers to ride off any drop-off instead of attempting to return to the roadway immediately.

Thus, a reason for the longer edge scour locations is that drivers, once they drop a tire off the pavement edge, will stay in the rut until they can naturally come back on the pavement.

Once the shoulder has been disturbed by tire action, the shoulder material is soft and subject to deeper and more extensive rutting. As edge scour develops and fill is added and then displaced by the tire action, ruts occur. The dislodged shoulder material is deposited further from the pavement edge and creates a raised shoulder condition or contributes to the creation of turf diking.

This raised shoulder cumulative condition presents problems in terms of ruts and road drainage. Constant maintenance results.

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