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Contents

Foreword

Papers in this Record are sponsored by the Committee on Traffic Control Devices and also by the Committee on Railroad-Highway Grade Crossings. The leadoff paper by Dewar, "Criteria for the Design and Evaluation of Traffic Sign Symbols," reports the results of an international expert opinion survey on the important issue of symbol use for traffic signs.

The next two papers examine the use of different devices at intersections and their effects. Eck and Biega used a before-and-after analysis to compare two-way and four-way stop sign control on the basis of delay, violation rates, and road user costs, concluding that four-way stops were less effective in these respects. In "Traffic Control and Accidents at Rural, High Speed Intersections," Agent reports on a study of 65 rural intersections and the measures (such as providing adequate warning) that can be applied to reduce accidents.

Pavement markings are the subject of papers by Dudek et al. and by Cottrell, who deal with the topics of work zones and the use of edgelines, respectively. Both papers attracted formal discussions that are presented with the papers.

The Committee on Railroad-Highway Grade Crossing sponsored "Reliability and Risk Assessment in the Prediction of Hazards at Rail-Highway Grade Crossings," by Faghri and Demetsky, who report on successful use of a new model that can be applied to evaluate crossings and prioritize improvements. The second paper sponsored by this committee, by Richards and Heathington, reports on a Tennessee survey of motorist understanding of grade crossing control devices, which revealed deficiencies in both public and police officer comprehension of some devices.

In "CALSIG: An Integration of Methodologies for the Design and Analysis of Signalized Intersections," Cassidy and May present a procedure that can be used for several levels of analysis and that can aid in identifying deficiencies and implementing improvements. Sharp and Parsonson describe another new procedure, this one for dealing with signal malfunctions, by using an expert systems approach in responding to telephone calls about signal problems. A third paper on signalized intersections is presented by Najafi: "Sketch Planning Process for Urban Arterial Signalized Intersection Improvements." The suggested process incorporates benefit/cost techniques and a broad range of factors, including excess fuel consumption and safety considerations.

The last two papers address the delineation needs of guiderails and barriers. In "Guiderail Delineation," Campi reports on the value of delineation and research that led to a preferred design for such installations. Ullman and Dudek report on a study of five treatments for a concrete barrier, recommending cube-corner delineators at 200-ft spacing as the most cost effective.

Criteria for the Design and Evaluation of Traffic Sign Symbols

ROBERT DEWAR

Several criteria for traffic sign symbols were examined through a questionnaire survey that allowed determination of the importance, or weighting, that should be assigned to each symbol in the design and evaluation of signs. The survey sample included traffic sign experts (members of national traffic control device committees) and practicing traffic engineers from Australia, New Zealand, Canada, and the United States. Separate ratings were assembled for symbols in general and for warning, regulatory, and information symbols in particular. Understandability was the factor rated most important, with conspicuity second. Learnability was considered least important, while reaction time, legibility distance, and glance legibility were rated equally but were determined to be more important than learnability.

The use of symbols (pictographs) to convey information has become prevalent in the past two decades. This is particularly evident in the case of traffic signs, on which symbols are used to convey dozens of different messages. Some of those responsible for traffic control devices believe that almost any message that needs to be conveyed to drivers can be expressed in this form, while others feel that the proliferation of symbolic traffic signs on our highways does more to confuse drivers than to inform them. Recent efforts to develop new symbolic messages indicate that not all messages can be translated into symbols. Research sponsored by the Federal Highway Administration (FHWA) (*1*) indicates that significant proportions of drivers have difficulty understanding symbolic messages that are presently included in the *Manual on Uniform Traffic Control Devices*. In 1985 the FHWA proposed deleting the word message alternates for several traffic signs in the belief that the symbolic versions were well enough understood that the word messages were no longer necessary. It is reasonable to assume that once a symbol has been used on a highway system for many years, drivers will come to know its meaning. This is apparently not the case, however, for many of the symbols presently used.

A task force of the National Committee on Uniform Traffic Control Devices has concluded that certain messages are well understood and do not need to be conveyed with words. The Task Force also believes, however, that evidence on the majority of symbols in the manual is either lacking or indicates that these symbols are not well understood.

Research on traffic sign perception indicates that symbolic messages have a number of advantages over written ones. The most obvious is perhaps the fact that the driver need not be able to read the language of the country in which the symbolic signs are used, which is a benefit for international travelers. Other advantages include greater legibility distance (*2*, *3*), easier recognition under degraded visual conditions such as fog (*4*), readier visibility at a glance (*4*, *5*), possibility of a more rapid response (*4*), and greater conspicuity than word signs (*6*).

It should be noted that these various advantages of symbolic messages reflect several criteria for their effectiveness. Unfortunately, the development of symbolic messages has frequently been hampered by poor research and in some cases no research, as outlined by Dewar and Ells (*7*). Another problem is the tendency to use a single measure of traffic sign adequacy (e.g., understandability, reaction time, or glance legibility) rather than a battery of tests. In some instances, multiple measures have been used (*8*), but even in these studies there has been no indication of the relative importance or weight that should be attributed to each of the measures employed.

In a series of experiments, Dewar and his colleagues used the same set of eight traffic sign symbols and took several measures—legibility distance on the roadway, reaction time, glance legibility, semantic differential ratings, and a preference measure (ratings of clarity of the sign's meaning). Roadway legibility distance was found to be correlated with reaction time (*9*), and semantic differential ratings were correlated with preference ratings (*10*); however, glance legibility was not found to correlate with any of the other measures.

In another series of experiments on traffic signs, Roberts et al. (*11*) used understanding time, accuracy of comprehension, certainty of comprehension, preference, and identification time. An "efficiency index" of each sign's overall effectiveness was calculated on the basis of these five measures. The only meaningful correlation found was that between understanding time and certainty of the accuracy of the response ($r = +0.28$). It appears that the five procedures used by Roberts et al. measured quite different aspects of perception and comprehension of traffic sign symbols.

Another series of experiments, carried out at the University of Melbourne, Australia, also employed several techniques in an extensive evaluation of signs bearing turn restriction messages (*12–15*). Measurements included comprehension, reaction time, glance legibility, legibility distance, and short-term memory for traffic sign messages. Results from the various measures were not always in agreement. Analyses were not performed to determine how the various measures correlated with one another, but they appeared to be measuring different aspects of traffic sign effectiveness.

The various types of research mentioned previously used a number of techniques to measure traffic sign effectiveness. An examination of the results makes it clear that the various

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measures are not always closely related. This suggests the need to use more than one method in evaluating traffic sign symbols, but this choice of technique still leaves open the questions of how to combine the data from various measures and what relative importance should be assigned to the different measures. On the basis of previous research on traffic signs, and on the general requirements for a good sign, it is suggested that the following criteria are important in evaluating and designing traffic sign symbols:

- *Legibility distance.* The greatest distance at which the symbol can be clearly "read";
- *Understandability.* The ease with which the symbol can be understood;
- *Conspicuity.* The extent to which a sign can be easily detected or seen in a visually complex environment;
- *Learnability.* The extent to which the meaning of a symbol can be learned and remembered;
- *Glance legibility.* The ease with which the symbol can be "read" when it is seen for only a fraction of a second; and
- *Reaction time.* How quickly the meaning of the sign can be identified.

The study described here examined the relative importance of each of these criteria for the development and evaluation of traffic sign symbols.

METHOD

A questionnaire survey was conducted with eight sample groups of subjects. Four of the groups consist of individuals who can be considered experts in the design and development of traffic signs, and four consist of practicing traffic engineers. The questionnaire asked the subjects to rate, on a 10-point scale from very important to very unimportant, the importance of six criteria for the development and evaluation of traffic sign symbols. Definitions of the criteria, which were listed earlier, were provided on the first page of the questionnaire. The subjects initially rated these criteria without reference to any particular class of traffic sign message. They then rated the same criteria as applied specifically to warning, regulatory, and information signs, assigning separate ratings to each type of sign. Finally, an open-ended question solicited comments on any additional criteria that the subjects might consider important in the design of traffic sign symbols, without reference to sign classification. The questionnaires were distributed by mail to all sample groups except Groups 4 and 8 (described later). For Group 4, the questionnaires were distributed and collected with the assistance of Alan Forbes of the Psychology Department of the University of Wellington (Wellington, New Zealand); in the case of Group 8, the questionnaires were administered at a traffic safety workshop in Sydney, Australia.

SUBJECTS

A total of 153 subjects participated in the survey. All were considered to be knowledgeable about traffic signs and their use on the basis of experience and/or membership on a committee responsible for national traffic sign standards. The sample consisted of four groups of experts and four groups of practicing traffic engineers, as follows:

Group 1. 20 members of the U.S. National Committee on Uniform Traffic Control Devices (NCUTCD);
Group 2. 30 members of the Council on Uniform Traffic Control Devices for Canada (CUTCDC);
Group 3. 11 members of the Standards Association of Australia (SAA) Committee (MS/12), responsible for the Australian Manual on Uniform Traffic Control Devices;
Group 4. 16 New Zealand professionals involved with traffic control devices to varying degrees; five members were on the National Roads Board Committee on Traffic Signs, employees of the Road Transport Division, Ministry of Works and Development of New Zealand;
Group 5. 29 practicing traffic engineers from the United States;
Group 6. 12 practicing traffic engineers from Canada;
Group 7. 21 traffic engineers from Victoria, Australia, who were responsible for traffic control devices in their particular jurisdictions;
Group 8. 14 local government traffic engineers and consultants from various locations in New South Wales, Australia, who were attending a traffic safety workshop.

The sample provides a broad representation of experts and practicing traffic engineers who are highly knowledgeable about the development and design of traffic sign symbols and/or their application to traffic control on the roadways.

RESULTS

The frequency of occurrence of responses to each questionnaire item was determined, and the mean importance ratings were calculated (Tables 1 and 2). Before conducting the major analysis, the reliability of the rating measure and the nationality differences were examined. These preliminary analyses indicated no significant differences between the two Australian samples of practicing traffic engineers, suggesting reliability of the measure, and no differences between the groups of experts from Australia and New Zealand, suggesting that there were no important nationality differences between these two groups. Furthermore, there were no substantial differences between the opinions of practicing traffic engineers and experts. Likewise, differences were minimal between the Canadian and U.S. samples, but for the North American sample the practicing traffic engineers rated four criteria (understandability, glance legibility, and reaction time for warning signs, as well as glance legibility for regulatory signs) as being of greater importance. These small differences indicated good overall consistency in the ratings.

The statistical test used was the median test, which allowed comparison of the particular pairs of samples and pairs of criteria that were of interest. For each analysis the data were divided at the center of the distribution and the chi-square value was calculated. Separate analyses were done for the ratings on traffic signs in general (the first question) and for the individual types of signs—warning, regulatory, and informational. Figure 1 shows the mean ratings of the sample from Australia and New Zealand, as well as those of the Canadian/U.S. sample. To allow comparison of ratings among the six criteria, data from

TABLE 1 IMPORTANCE RATINGS FOR AUSTRALIA/NEW ZEALAND SAMPLE GROUPS

	SAA Committee MS/12	New Zealand Sample	N.S.W. Traffic Engineers	Victoria Traffic Engineers	TOTAL
N	11	16	14	21	62
GENERAL					
Legibility Dist.	3.00	3.69	3.35	3.00	3.26
Understandability	2.27	1.44	1.86	1.76	1.79
Conspicuity	2.64	1.69	1.93	2.14	2.06
Learnability	4.27	3.38	3.86	4.00	3.85
Glance Legibility	3.45	3.31	1.64	2.95	2.84
Reaction Time	2.82	3.06	2.36	2.57	2.69
WARNING					
Legibility Dist.	3.18	3.25	2.36	2.52	2.79
Understandability	2.36	1.75	1.57	1.81	1.84
Conspicuity	2.55	1.69	1.50	2.00	1.90
Learnability	4.18	3.38	3.64	4.05	3.81
Glance Legibility	3.91	3.31	2.14	2.86	3.00
Reaction Time	2.82	3.00	2.07	2.29	2.51
REGULATORY					
Legibility Dist.	2.55	2.63	3.14	3.00	2.85
Understandability	2.27	1.50	2.71	2.00	2.08
Conspicuity	1.82	1.44	2.14	2.10	1.89
Learnability	3.73	2.75	3.57	3.95	3.52
Glance Legibility	2.91	3.00	2.43	3.14	2.90
Reaction Time	2.55	2.38	3.50	2.76	2.79
INFORMATION					
Legibility Dist.	2.91	3.63	2.29	3.95	3.31
Understandability	3.18	2.00	3.36	3.10	2.89
Conspicuity	3.64	2.31	3.36	3.52	3.19
Learnability	4.91	4.50	5.14	4.76	4.81
Glance Legibility	3.55	4.00	3.93	4.81	4.18
Reaction Time	4.27	4.06	4.14	4.62	4.31

* low ratings indicate high degree of importance

all four Australia/New Zealand samples were combined because the preceding analyses had shown essentially the same trends for the four groups of subjects. Within each set of data (general, warning, etc.), all possible combinations of the pairs of criteria were compared. Legibility distance ratings were compared with ratings on understandability, conspicuity, and so on. Similar comparisons were made between the ratings on understandability and the remaining criteria. The same analyses were then carried out for the combined data from the four North American samples.

Results of these analyses (using the median test) are summarized in Table 3, in which only the significant differences are presented. Because of the subjective nature of the measures and the large number of tests carried out, a relatively stringent criterion of $p < 0.002$ was selected as the index of statistical significance for these comparisons.

It is evident that understandability is a particularly important criterion for a traffic sign symbol. Conspicuity ranks a close second behind understandability. Otherwise, the trends were consistent for the Australia/New Zealand sample, although this was not so for the North American sample. The other striking feature is the consistently low rating of learnability. When all the data are considered, the criteria of glance legibility, legibility distance, and reaction time are rated equal to each other in importance, below understandability and conspicuity but above learnability. It should be noted that all criteria are found to be of some importance, if the rating values of 5 and 6 are taken to represent a neutral point on the scale. Three of the criteria approach this neutral rating, however, in the case of information sign symbols.

Some differences can be seen between classes of traffic signs. Understandability appears to be particularly important for warning and regulatory symbols.

The most frequently mentioned additional criteria were sign location (mentioned 18 times), uniformity (18), color (10), night visibility (10), size (6), and shape (6). Note that the first of these (location) is not actually a criterion for sign design but is rather for implementation.

TABLE 2 MEAN IMPORTANCE RATINGS FOR CANADA/UNITED STATES SAMPLE GROUPS

N	NCUTCD 20	CUTCDC 30	U.S. ENGINEERS 29	CANADIAN ENGINEERS 12	TOTAL 91
GENERAL					
Legibility Dist.	2.95	2.90	3.00	3.25	3.06
Understandability	2.00	2.87	2.07	1.58	2.41
Conspicuity	2.75	2.74	2.86	3.25	2.88
Learnability	3.95	3.97	3.93	3.25	3.90
Glance Legibility	3.15	3.10	2.69	3.00	3.00
Reaction Time	2.60	3.03	2.46	2.08	2.66
WARNING					
Legibility Dist.	2.95	2.60	2.76	3.25	2.82
Understandability	1.84	2.60	2.00	1.25	2.08
Conspicuity	2.70	2.83	2.41	2.58	2.64
Learnability	3.80	3.40	3.62	3.42	3.56
Glance Legibility	3.00	3.70	2.34	2.17	2.92
Reaction Time	2.90	3.03	1.90	2.33	2.54
REGULATORY					
Legibility Dist.	3.20	3.00	3.07	2.92	3.06
Understandability	2.10	2.53	2.00	1.17	2.09
Conspicuity	2.65	2.67	2.62	2.00	2.56
Learnability	3.85	3.30	3.90	2.58	3.52
Glance Legibility	3.25	3.40	2.79	2.92	3.05
Reaction Time	3.00	3.53	3.24	2.50	3.19
INFORMATION					
Legibility Dist.	4.00	2.93	3.79	4.50	3.65
Understandability	2.80	3.16	2.83	2.67	2.91
Conspicuity	3.95	3.57	3.69	4.17	2.67
Learnability	5.32	4.77	4.90	5.17	4.98
Glance Legibility	4.90	3.63	4.10	4.83	4.22
Reaction Time	4.75	4.33	3.72	4.75	4.29

* low ratings indicate high degree of importance

DISCUSSION

The high degree of importance placed on symbol understandability is not surprising. The regularity with which this criterion is incorporated into studies of traffic signs (it is frequently the only variable measured) attests to its importance among researchers. Understandability is dependent not only on how clearly the symbol conveys its intended message but also on the time available for processing it (*2*) and the distance from which it is viewed (*9*). A simple design is recommended because small elements of a symbol cannot be distinguished at the distance usually required in traffic sign perception. In contrast to understandability, the highly rated criterion conspicuity has received very little attention from researchers, except in Australia (*6*). This regional bias may account for the relatively greater importance placed on conspicuity by the researchers in the Australia/New Zealand samples. It could be argued that this criterion is the most fundamental of all, for the other characteristics of a traffic sign symbol become irrelevant if the sign is not seen by the driver. It should be noted that conspicuity per se may not be considered a function of symbol design but is determined more by symbol size, color, shape, and contrast between the symbol and the background of the sign panel on which it appears.

The consistent rating of learnability as less important than the other criteria may be seen by many as a surprise. The low rating of this factor by Group 3, the SAA Committee MS/12 members, is particularly surprising in view of the use at the time of this criterion, along with understandability, to evaluate symbols proposed for the Australian Manual of Uniform Traffic Control Devices. It could be suggested that these results reflect the realization that this criterion is not particularly important in symbol design, especially if a symbol is high in understandability (the most important criterion). In addition, sign designers may feel that learnability is the criterion least

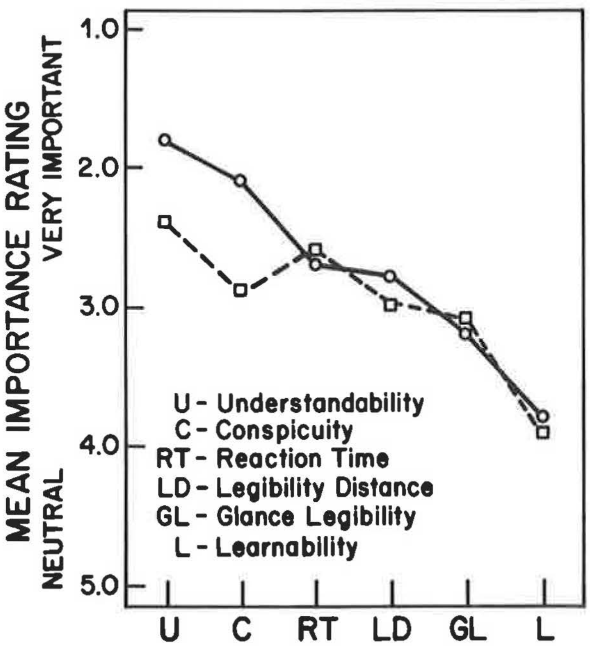

FIGURE 1 Mean importance ratings of six symbol criteria. Dashed line shows ratings from Australia/New Zealand; solid line indicates ratings from Canada/United States.

TABLE 3 SUMMARY OF SIGNIFICANT DIFFERENCES BETWEEN CRITERIA

Data Set	Australia/New Zealand	Canada/United States
General	U > LD, L, RT C > LD, L	LD > L U > L, GL C > L GL > L RT > L
Warning	LD > L U > LD, L, GL RT > L C > L	U > LD, L, GL RT > L
Regulatory	U > L C > LD, L, GL, RT	U > LD, C, L, GL, RT C > LD, GL
Information	LD > L U > L, GL, RT C > L	U > LD, L, GL, RT C > L

NOTE: U = understandability; C = conspicuity; RT = reaction time; LD = legibility distance; GL = glance legibility; and L = learnability.

under their control, since education of drivers is not their responsibility. However, simplicity of design is often suggested as a worthwhile criterion.

The importance of conspicuity is reflected by the large number of times that sign location or placement is indicated in the spontaneous responses to the open-ended question. Location is not a criterion for symbol adequacy but instead relates to implementation of signing standards by practitioners of traffic engineering. In view of the stress that has been placed on conspicuity, it may be that some subjects see good conspicuity of signs as partial compensation for poor placement. The importance of uniformity of symbols, both within and among traffic sign systems, is also evident. If the stress that has been placed on this issue in the literature is considered, it is surprising that symbol uniformity was not mentioned more often. Visibility under conditions of darkness was of some concern as well. These comments have revealed only one additional criterion (uniformity) that relates directly to design of symbols on traffic signs.

Although committees composed largely of traffic engineers are responsible for determining the designs of symbols for traffic signs, it would be valuable to know the relative importance assigned to symbol criteria by a representative sample of drivers as well. User input has been incorporated into the design of a variety of systems and machines, and the same should be done with visual communication systems used on highways.

The present analysis has shed some light on the issue of the relative importance of the various criteria for traffic sign symbols. The measurement was subjective in nature, and the sample was small and limited to four countries. The overall consistency of the data across the samples, however, permits conclusions to be drawn about the views of traffic sign experts and practicing traffic engineers. It is tempting to suggest the use of a formula with differential weightings applied to each of those criteria, but this would be premature in view of the limited data gathered. However, this study does emphasize the need to take a number of factors into account in the design of symbols. It also provides those who develop traffic signs with information on the relative importance of six criteria for traffic sign symbols.

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Field Evaluation of Two-Way Versus Four-Way Stop Sign Control at Low-Volume Intersections in Residential Areas

RONALD W. ECK AND JAMES A. BIEGA

This study was conducted to acquire data that would assist in resolving the conflicting opinions and research results that exist about two-way versus four-way stop sign control at low-volume intersections in residential areas. A unique opportunity to compare operational issues at such intersections existed at a West Virginia municipality in which three intersections were regulated by two-way stop sign control during the winter months and then converted to four-way stop sign control during the summer. The experimental design was a before-and-after analysis with control intersections. Traffic volume, delay, speed, and observance data were collected, analyzed, and used to determine road user costs. Accident experience and potential legal issues were also investigated. At the three intersections studied, use of four-way stop control was found to cause unnecessary motorist delay and road user costs. A delay analysis found that the use of four-way stop control was 2.6 times less efficient than use of two-way control. Annual road user costs increased by $2,400 per intersection after installation of four-way stop control. Mean midblock vehicle speeds were not affected by the type of intersection control; however, 85th percentile speeds decreased by 2.3 mph after installation of four-way stop control. The driver observance study showed that the stop sign violation rate increased by 11 percent after installation of four-way control.

The degree of traffic control used at an at-grade intersection should reflect the volume and speed of traffic associated with the intersection. Intersections with high volumes, high speeds, or both demand a higher level of intersection control than those with low speed and low volumes. For a variety of reasons (e.g., lack of knowledge of warrants for traffic control devices, pressure from the general public or politicians, lack of data about traffic and speed conditions at a site, or a change in traffic conditions over time) the level of traffic control at an intersection may not be appropriate for the given volume and speed.

Many jurisdictions in the United States have installed four-way stop sign control at low-volume intersections in residential areas in an attempt to reduce speeds or to provide additional safety for children playing on or near the streets or both. According to the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*), stop signs should not be installed for speed control because this misuse of traffic control devices probably promotes a lack of respect for all traffic control devices and may decrease driver compliance with all such devices. Recent research (*2–4*) has borne out the compliance problem. Other adverse consequences include the following:

- While several studies (*4–6*) have demonstrated the relative ineffectiveness of stop signs for speed control, there is some evidence (*5*) that drivers may actually increase their midblock speeds between signs;
- Use of four-way stop signs in place of two-way stop signs may cause substantial increases in automobile energy consumption, vehicle operating costs (*7–10*), and traffic delay; and
- Use of unwarranted stop sign control raises legal questions.

Findings concerning accident experience at two- and four-way stop controlled intersections are less definitive (*8, 11, 12*).

During the literature review, no studies could be located that utilized field data at low-volume (ADT less than 400 vehicles per day) stop-controlled intersections in residential areas. This is probably because of the difficulty in obtaining adequate sample sizes at this low volume level. Additional field research was needed, therefore, to provide a complete comparison of the actual operational characteristics associated with low-volume two-way and four-way stop-controlled intersections in residential areas.

A unique opportunity to compare operational issues at intersections under both two-way and four-way stop sign control was found in Star City, a town with a population of about 1,500 that is located north of and adjacent to Morgantown in north-central West Virginia. Three low-volume intersections in a residential section of the community were controlled by two-way stop sign control during the winter months and converted to four-way stop sign control during the summer months. This has been standard practice in the community for a number of years because it reduces vehicle speeds during summer months when children are playing in or near the street and allows vehicles to ascend grades when road surfaces are snow-covered. Since the site conditions and traffic volumes at the intersections remained constant during the use of the two-way and four-way stop sign control, variations in data obtained from studies conducted at the intersections would be attributable to the specific type of control being used and would not be influenced by extraneous factors such as variations in intersection geometrics and/or variations in sight distances. This latter situation would exist if a comparison were made of two-way versus four-way stop control at adjacent, similar intersections.

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STUDY OBJECTIVES

To accomplish the overall goal of the project, several specific objectives were established:

- To review previous research that has evaluated two-way versus four-way stop sign–controlled intersections;
- To collect traffic volume, spot speed, delay, compliance, and accident data at selected two-way and four-way stop sign–controlled intersections;
- To estimate and compare delay and road user costs at the selected two-way and four-way stop sign–controlled intersections; and
- To evaluate accident experience and legal aspects associated with alternating between two-way and four-way stop sign control.

DATA COLLECTION AND ANALYSIS

Experimental Design and Site Selection

A before-and-after analysis with control intersections was chosen as the experimental plan because use of control sites allows the evaluator to reduce the influence of other variables on study results. Traffic data were collected and analyzed during the before condition. The intersection control was then changed, and traffic data were collected and analyzed during the after condition. Control data were also collected and compared at other nearby intersections during both the before and after condition to take into account possible changes in traffic trends that could have influenced results at the study intersections. Note that in all cases, data collection studies were conducted at the same location, on the same day of the week, and at the same time of day during the before and after study conditions to minimize introduction of bias into the results.

The study intersections, designated S1, S2, and S3, were right angle intersections of two-lane intersecting streets located in a moderate-income residential section of Star City. Posted speed limits throughout the area were 25 mph. The north-south street was the major roadway at each intersection; stop signs were located on the east and west approaches during the use of two-way stop control. Sight distances varied considerably. The topography of the area was generally level to rolling; all three intersections had grades of about 6 percent on the north-south roadways. To provide adequate control, the researchers stipulated that one two-way stop intersection and one four-way stop intersection be used as control intersections.

Traffic Data

All data on the use of two-way stop control were collected during a four-week "before" period. City officials then converted the two-way stop sign control at each of the study intersections to four-way stop control. A waiting period of 6 weeks was allowed to permit traffic to adjust to the new control conditions. Data collection was then resumed. All data on the use of four-way stop sign control were collected during a 4-week "after" period. The amount of data that could be collected during the before and after conditions was constrained by the following factors: (a) the intersections that were being studied were on very low-volume streets and (b) there was only a limited amount of time available between the start of the study and the changeover date from two-way to four-way stop sign control.

Traffic Volume

Portable pneumatic tube traffic counters were used to acquire average daily traffic (ADT) volumes at the study and control intersections during both the before and after conditions. These data are presented in Table 1. In addition, counts of vehicle turning movement were made at each intersection before and after the conversion.

TABLE 1 AVERAGE DAILY TRAFFIC VOLUMES AT THE STUDY INTERSECTIONS

Intersection	Major Street ADT	Minor Street ADT	Total	Side Street Traffic (%)
S1				
Before	337	130	467	28
After	344	117	461	25
S2				
Before	333	130	463	28
After	255	117	372	31
S3				
Before	413	153	566	27
After	406	171	577	30

The average daily traffic on north-south and east-west streets did not change significantly between before and after conditions. Five of six before and after ADT comparisons indicated traffic volume variations of less than 10 percent. The exception occurred on the north-south street at Intersection S2; this 23 percent traffic volume decrease may be attributable to motorists choosing alternative routes in an effort to minimize delay. Hourly traffic variations on the north-south and east-west streets at the study intersections were similar during before and after conditions. Vehicle turning movement volumes were very similar during before and after conditions at the study intersections.

Traffic Delay

Two types of raw traffic delay data were collected for use in this study: (a) average intersection traversal time and (b) stopped time delay. Intersection traversal time was defined as the time required for a vehicle to travel from the midblock point on one approach to the midblock point on the approach directly opposite the point at which the vehicle entered the intersection. Average intersection traversal time was obtained by summing each individual intersection traversal time and dividing the sum by the number of observations. Four separate average intersection traversal time studies (i.e., one for each direction of travel) were conducted for each study and control intersection during each traffic control condition.

Stopped time delay data were collected on each intersection approach during both before and after conditions. To collect these data, an observer was positioned near the intersection approach under study. This observer used a stopwatch to record the amount of time that each entering vehicle was traveling at a speed of 3 mph or less.

Intersection traversal time and stopped time delay data were collected in conjunction with average daily traffic approach volumes or spot speed data (or both) to determine (a) total intersection delay, (b) stopped time delay, and (c) speed change delay for each traffic control condition and direction of travel at each intersection. Total intersection delay was defined as the total delay experienced by vehicles traveling through an intersection in a particular direction of travel. Total intersection delay was determined by first calculating intersection traversal time on the basis of average midblock speed. The intersection traversal time based on speed was then subtracted from actual intersection traversal time to obtain a total intersection delay expressed in seconds per vehicle. Total intersection delay expressed in hours per day was then calculated by multiplying total intersection delay expressed in seconds per vehicle by the appropriate average daily traffic approach volume. Total intersection delay was assumed to represent delays associated with all turning movements on a particular approach, even though intersection traversal time had only considered vehicles that were traveling straight through. The validity of this assumption was substantiated to some degree by traffic volume characteristics: vehicles traveling straight through constituted at least 71 percent of the total approach volume on the north-south approaches and at least 52 percent of the total approach volume on four of six east-west approaches.

Average total intersection delay on the north-south approaches at the study intersections increased from 0.4 to 5.0 sec per vehicle during the after condition (Table 2). Average total intersection delay on the east-west approaches decreased from 5.1 sec per vehicle during the before condition to 4.5 sec per vehicle during the after condition. The increased north-south street delays were expected; nominal average total intersection delay of less than 0.4 sec per vehicle, which resulted from vehicles exhibiting caution on entering the intersection, would inevitably increase after installation of stop signs on the north-south approaches. The decreased east-west street delays could have been caused by an increased sense of security experienced by motorists entering from the east and west approaches. If a motorist on the east or west approach of an intersection knew that drivers on the north and south approaches had to stop, the motorist might not be as concerned about north-south street traffic and might enter the intersection without exercising normal caution.

The before and after daily total intersection delays were also compared so that the differences in north-south and east-west street traffic volumes would be considered in the delay analysis. The before and after comparisons presented in Table 3 indicate that daily total intersection delay at the study intersections increased by approximately 12 min on each north and south approach during the after condition. Daily total intersection delay decreased by less than 1 min on each east and west approach.

TABLE 3 DAILY TOTAL INTERSECTION DELAY AT THE STUDY INTERSECTIONS

	North-South Street		East-West Street		Total	
Intersection	Two-Way	Four-Way	Two-Way	Four-Way	Two-Way	Four-Way
S1	4.4	27.5	11.8	12.2	16.1	39.8
S2	1.3	16.8	13.0	7.7	14.2	24.5
S3	0.0	32.1	11.6	12.5	11.6	44.6
Total, all	5.7	76.4	36.4	30.8	42.1	108.9
Average per approach	1.0	12.7	6.1	5.1	3.5	9.1

NOTE: Delays are given in minutes.

The total intersection delay analysis had already considered the overall effect of north-south and east-west street traffic volume differences. Stopped time (the average time that vehicles were stopped or practically stopped) and speed change (the average time required for vehicles to decelerate from average vehicle speed to a minimum speed or stop plus the time required to accelerate back to average vehicle speed) delays were therefore analyzed on a seconds per vehicle basis to obtain a more detailed and complete understanding of vehicle operational characteristics during the two-way and four-way stop control conditions. Stopped time delay was determined directly from stopped time delay data. Speed change delay was calculated by subtracting stopped time delay from total intersection delay.

In general, stopped time delay at the study intersections varied from 0.9 to 3.3 sec per vehicle during before and after conditions. During the two-way stop condition, average stopped time delay on the east and west approaches was 2.1 sec per vehicle. After installation of stop signs on the north-south

TABLE 2 TOTAL INTERSECTION DELAY AT THE STUDY INTERSECTIONS

Intersection	North Approach	South Approach	North-South Average	East Approach	West Approach	East-West Average
S1						
Two-way (before)	1.0	0.8	0.9	4.1	5.2	4.7
Four-way (after)	5.7	6.1	5.9	4.9	4.9	4.9
S2						
Two-way (before)	0.0	0.5	0.3	5.1	6.4	5.8
Four-way (after)	4.6	4.6	4.6	4.2	4.3	4.3
S3						
Two-way (before)	0.0	0.0	0.0	5.5	4.2	4.9
Four-way (after)	3.7	5.2	4.5	5.2	3.7	4.5
Overall average						
Two-way (before)	0.3	0.4	0.4	4.9	5.3	5.1
Four-way (after)	4.7	5.3	5.0	4.8	4.3	4.5

NOTE: Delays are given in seconds per vehicle.

approaches, average delay on the east-west approaches was reduced to 1.5 sec per vehicle. This reduction is overshadowed, however, by a 1.3 sec per vehicle increase in stopped time delay on the north-south approaches. Because the north-south streets were the major roadways at all study intersections, the 1.3 sec per vehicle increase in stopped time delay is far more important than the 0.6 sec per vehicle reduction.

Analysis of speed change delays at the study intersections, presented in Table 4, indicated that average speed change delay for the north-south directions of travel was only 0.4 sec per vehicle during the two-way stop control condition. An average speed change delay of 3.7 sec per vehicle was evident after installation of four-way stop control. Analysis of speed change delay for the east-west directions of travel showed that no significant trends occurred during before and after conditions.

Spot Speed

Spot speed data were collected for both directions of travel at the midblock point on all four approaches of each intersection during both the before and after conditions. In general, mean speeds on the north-south streets were consistently greater than those on the east-west streets. The average mean speed on the north-south streets, presented in Table 5, decreased from 23.0 to 21.9 mph after installation of four-way stop sign control. Average mean speed on the east-west streets decreased from 18.6 to 18.3 mph. These differences were not statistically significant. Thus mean speeds on the north-south and east-west streets can be said to be relatively unaffected by the use of two-way and four-way stop control.

The 85th percentile speeds on the north-south streets decreased by an average of 2.3 mph after installation of four-way stop sign control. The 85th percentile speeds on the north-south streets, presented in Table 5, were 2 mph in excess of the 25-mph speed limit during the before condition and identical to the 25-mph speed limit during the after condition. The 85th percentile speeds on the east-west streets remained constant at 21.7 mph during both before and after conditions.

In general, the limits of the 10-mph pace decreased and the percentage of vehicles traveling within the 10-mph pace increased

TABLE 4 SPEED CHANGE DELAY AT THE STUDY INTERSECTIONS

Intersection	North Approach	South Approach	North-South Average	East Approach	West Approach	East-West Average
S1						
Two-way (before)	1.0	0.8	0.9	2.7	3.2	3.0
Four-way (after)	4.6	4.9	4.8	3.6	3.3	3.5
S2						
Two-way (before)	0.0	0.5	0.3	3.3	4.8	4.1
Four-way (after)	3.3	2.7	3.0	2.4	3.2	2.8
S3						
Two-way (before)	0.0	0.0	0.0	2.4	0.9	1.7
Four-way (after)	2.8	3.9	3.4	3.6	2.0	2.8
Overall average						
Two-way (before)	0.3	0.4	0.4	2.8	3.0	2.9
Four-way (after)	3.6	3.8	3.7	3.2	2.8	3.0

NOTE: Delays are given in seconds per vehicle.

TABLE 5 OVERALL VEHICLE SPEED CHARACTERISTICS AT THE STUDY INTERSECTIONS

Intersection	Orientation	Stop Control In Use	Mean Speed X (mph)	Standard Deviation S (mph)	Median Speed (mph)	85th Percentile Speed (mph)	15th Percentile Speed (mph)	Limits of 10 mph Pace Lower (mph)	Limits of 10 mph Pace Upper (mph)	Percent Vehicles Within 10 mph Pace
S1	N-S	2-Way	24.0	4.4	24.0	28.8	19.1	19.3	29.3	74
		4-Way	23.0	3.6	22.4	25.6	18.1	17.5	27.5	87
	E-W	2-Way	19.5	3.2	19.5	22.4	15.6	14.0	24.0	92
		4-Way	18.9	3.4	18.7	22.4	15.0	13.5	23.5	94
S2	N-S	2-Way	21.6	4.1	21.6	25.8	16.6	16.0	26.0	78
		4-Way	20.6	3.5	20.4	23.7	16.7	15.0	25.0	88
	E-W	2-Way	18.1	2.8	17.6	21.2	15.2	12.5	22.5	94
		4-Way	18.1	3.1	17.9	21.2	14.6	13.5	23.5	96
S3	N-S	2-Way	23.3	4.6	23.2	26.5	18.4	18.3	28.3	75
		4-Way	22.2	4.2	22.1	24.7	18.0	17.3	27.3	82
	E-W	2-Way	18.1	3.4	17.1	21.4	15.0	13.3	23.3	94
		4-Way	18.0	3.0	17.6	21.5	14.9	12.3	22.3	95
Total All	N-S	2-Way	23.0	4.4	22.9	27.0	18.0	17.9	27.9	76
		4-Way	21.9	3.8	21.6	24.7	17.6	16.6	26.6	86
	E-W	2-Way	18.6	3.1	18.1	21.7	15.3	13.3	23.3	93
		4-Way	18.3	3.1	18.1	21.7	14.8	13.1	23.1	95

after installation of four-way stop control. Before and after upper limits on the north-south streets averaged 27.7 and 26.6 mph, respectively (Table 5). The percentage of vehicles traveling within the 10-mph pace on these streets increased by 10 during the after condition. The changes in the 10-mph pace on the east-west streets were insignificant.

Traffic Control Device Compliance

Traffic control device compliance studies were conducted at each of the study and control intersections during both the before and after conditions. The percentage of nonstopping drivers increased from 14.1 percent during the before condition to 25.1 percent during the after condition (Table 6). During the four-way stop sign control condition, 26.4 percent of the north-south street traffic did not stop and 23.8 percent of the east-west street traffic did not stop. The percentage of drivers performing a voluntary full stop and the percentage of drivers stopped by traffic remained essentially constant during before and after conditions. Approximately 15.7 percent made a voluntary full stop, and 3.5 percent were stopped by traffic. The percentage of drivers who practically stopped (0–3 mph) decreased from 65.7 to 55.8 during the after condition. Note that driver compliance percentages on the north-south and east-west streets were approximately equal during the four-way stop sign control condition.

Control Intersections

Analysis of traffic volume, delay, speed, and observation data from the control intersections indicated that before and after traffic characteristics (specifically, through volumes, turning movements, spot speed parameters, intersection traversal times, and driver compliance characteristics) at the control intersections did not change significantly. Because traffic characteristics were similar during before and after conditions, it was felt that data differences at the study intersections could be directly attributed to the type of stop control utilized at the intersections.

Accident Experience and Legal Cases

Accident data at each of the study intersections were obtained by reviewing the accident file for the town of Star City. Preliminary accident data evaluation indicated that only three accidents were recorded at the study intersections during the 5-year period from May 1979 to May 1984. None of the accidents was attributable to the use of a particular type of stop sign control; either the accidents were known to be caused by other events or the accident report forms did not provide needed information. Therefore accident data at the study intersections were deemed to be insufficient for the performance of a reliable accident analysis.

A search of legal cases was performed to identify cases that could be used to evaluate the legal aspects associated with using four-way instead of two-way stop signs at intersections. Special attention was given to locating cases that involved low-volume intersections at which (a) alternating two-way and four-way stop sign controls were used, (b) two-way stop sign control was replaced by four-way stop sign control, and (c) four-way stop control was replaced by two-way stop control. No relevant cases involving these issues were located, however. Apparently, any cases must have been decided in a trial court and were not appealed; consequently, they were never published.

ROAD USER COST ANALYSIS

The study compared before and after road user costs to determine the relative economy associated with the use of both two-way and four-way stop control. Costs considered for analysis were (a) daily motorist delay costs, (b) daily idling costs, and (c) daily speed change cycle costs. In all cases, procedures recommended by the AASHTO "Red Book" (*13*) were utilized. Cost factors were updated to current conditions by using the AASHTO-recommended procedures (*13*). Accident costs could not be calculated because there were insufficient accident data; environmental costs (associated with air and noise pollution) were determined to be negligible at the study intersections.

Daily motorist delay costs were determined for each direction of travel and stop control condition at each intersection. These costs represent the dollar value of time lost due to total intersection delay. Comparison of before and after daily motorist delay costs indicated that average daily motorist delay costs on the north-south streets increased from \$0.03 to \$0.32 during the after condition. Average daily motorist delay costs on the east-west streets decreased from \$0.15 to \$0.14 during the after period. Total daily motorist delay costs at the three study intersections increased from \$0.54 per day during the two-way stop condition to \$1.39 per day during the four-way stop condition. Daily vehicle idling costs were also calculated. Before and after daily idling costs were less than \$0.07 for each direction of travel.

Daily speed change cycle costs were calculated for each direction of travel and stop condition at each intersection. Daily speed change cycle costs for the north and south directions of travel at the study intersections were assumed to be zero during

TABLE 6 OVERALL DRIVER COMPLIANCE CHARACTERISTICS AT THE STUDY INTERSECTIONS

Driver Compliance Category	Percent of Drivers Within Each Driver Compliance Category					
	North-South Streets		East-West Streets		All Streets	
	2-Way	4-Way	2-Way	4-Way	2-Way	4-Way
Voluntary full stop	NA[a]	16.3	15.2	15.9	15.2	16.1
Stopped by traffic	NA	1.3	4.6	3.6	4.6	2.4
Practically stopped	NA	54.6	65.7	56.9	65.7	55.8
Nonstopping	NA	26.4	14.1	23.8	14.1	25.1

[a]Not applicable.

the two-way stop sign control condition because north-south street traffic was not required to stop. The average total daily speed change cycle cost on the north-south street at each study intersection was $6.34 per day after installation of four-way stop control. The average total daily speed change cycle cost on the east-west street at each study intersection remained essentially constant at approximately $2.38 per day during both before and after conditions. The installation of four-way stop sign control at the study intersections increased the total daily speed change cycle cost by $18.73 per day.

Daily motorist delay costs, daily idling costs, and daily speed change cycle costs were summed to obtain the total daily road user costs for each intersection and study condition. The daily speed change cycle cost was the most significant cost component in the road user cost analysis. During the two-way stop sign control condition, 91 percent of the total road user cost was attributable to speed change cycle costs, 7 percent was attributable to motorist delay costs, and the remainder was attributable to idling costs. Similarly, during the four-way stop sign control condition, 94 percent of the total road user cost was attributable to speed change cycle cost, 5 percent was attributable to motorist delay cost, and the remainder was attributable to idling costs.

The average total daily road user cost per study intersection increased by $6.58 per day after the conversion from two-way to four-way stop sign control (Table 7). The primary cause of this increase was the additional road user cost on the north-south street at each intersection. The average total daily road user cost on the north-south streets increased by $6.71 per day, while the average total daily road user cost on the east-west streets decreased by a negligible $0.13 per day. The installation of four-way stop sign control resulted in an average annual road user cost increase of $2,402.92 per study intersection, or an overall annual increase of $7,208.75 at the three study intersections. It was concluded that the use of two-way stop control was 3.5 times more efficient economically than the use of four-way stop control.

CONCLUSIONS AND RECOMMENDATIONS

Previous evaluations of two-way and four-way stop control used intersection delays, road user cost analysis, vehicle speeds, driver compliance to stop signs, accident analysis, or a combination of those factors as their basis. However, in a literature review, no studies were located that utilized field research along with all of these criteria to provide a complete comparison of the actual operational characteristics associated with low-volume two-way and four-way stop controlled intersections in residential areas.

The following specific conclusions derived from this study are applicable to intersections similar to the ones studied:

- Use of four-way stop sign control at low-volume residential street intersections causes unnecessary motorist delay and creates additional road user costs. In this case, use of two-way stop control was 3.5 times more efficient economically than the use of four-way stop control.
- Mean midblock speeds did not change significantly between the two-way and four-way stop conditions. However, use of four-way stop control resulted in a lower 85th percentile speed and a higher percentage of vehicles traveling within the 10-mph pace.
- The percentage of nonstopping vehicles was 11 percent higher during the four-way stop condition, indicating a general lack of respect for unwarranted four-way stop sign control.
- Accident data were insufficient to perform a reliable accident analysis.

It was concluded that in general, four-way stop sign control at low-volume residential street intersections should be changed to two-way stop sign control. The use of two-way stop sign control in place of four-way stop sign control minimizes delay and road user costs. Traffic engineering studies should be conducted, however, to take into account environmental and/or geometric conditions that may differ from those of the intersections described in this study.

Although accidents were not a problem at the intersections evaluated in this study, there are serious safety concerns associated with the practice of using alternative types of intersection control for different time periods during one year. These safety concerns focus on the time periods that follow the stop sign conversion. Accidents could result if drivers accustomed to proceeding through an intersection without being required to stop did not notice a recently installed stop sign. Similarly, accidents could result if drivers on cross streets proceeded into an intersection after removal of stop signs on a major street. Therefore it was concluded that the practice of using alternating types of intersection control for different periods of time during one year should be eliminated. Although the legal review revealed no relevant cases associated with alternating two-way and four-way stop control, good engineering judgment and sound risk management principles would indicate that four-way stop sign control should not be used at the study intersections.

TABLE 7 SUMMARY OF TOTAL ROAD USER COSTS AT THE STUDY INTERSECTIONS

	Daily Cost ($)								
	North-South Street		East-West Street		Total		Annual Cost Total ($)		Annual Increased Cost ($)
Intersection	Two-Way	Four-Way	Two-Way	Four-Way	Two-Way	Four-Way	Two-Way	Four-Way	
S1	0.06	6.34	3.02	2.06	3.08	8.94	1,124.20	3,263.10	2,138.90
S2	0.02	4.40	2.23	1.96	2.25	6.36	821.25	2,321.40	1,500.15
S3	0.00	9.47	2.67	2.98	2.67	12.45	974.55	4,544.25	3,569.70
Total, all	0.08	20.21	7.92	7.54	8.00	27.75	2,920.00	10,128.75	7,208.75
Average	0.03	6.47	2.64	2.51	2.67	9.25	973.33	3,376.25	2,402.92

Some additional research should be done to verify the results of this study. Additional study intersections in other geographic areas should be incorporated into future work so that the results can be deemed independent of local traffic trends and driver behaviors. A larger study area should also be used to obtain additional accident data so that a complete accident analysis can be performed.

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DISCUSSION

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Claims of statistical significance appear throughout the manuscript without sufficient clarification. Since a great deal of effort was expended in assuring a valid experimental design, it is logical that some statistical use was made of this design in the analysis. There is no mention, however, of the statistical method used in making these claims, nor of the level of significance. Also, there is no indication of sample size. How many vehicles are included in the tables? Although Table 5 is the most comprehensive by far, the numbers of vehicles are omitted. Since the authors recognize that the intersections in this study are low-volume, sample size could be critical to this study. As chair of the A3B11 Subcommittee on Statistical Methods in Accident Analysis, I felt compelled to make these comments.

AUTHORS' CLOSURE

We appreciate Pendleton's constructive comments on our paper. In responding to reviewers' comments on the original manuscript about the need to shorten the paper significantly and to orient it toward a user audience, we obviously neglected to include some necessary statistical information about our study. Pendleton deserves thanks for seeing to it that this information is presented.

Although it is not evident from the paper, we recognized that sample size was critical to a study of this type. One of the first steps in this work was to determine sample size requirements. For intersection traversal time and stopped time delay, minimum sample size requirements were obtained using the sample size requirements for travel time and delay studies contained in the *Manual of Traffic Engineering Studies* (4th ed., P. C. Box and J. C. Oppenlander, Institute of Transportation Engineers, Washington, D.C., 1976) for a confidence level of 95 percent.

For the spot speed and driver compliance data, we used the *Manual of Traffic Engineering Studies* sample size requirements for a confidence level of 90 percent. Because there was only a limited amount of time available between the start of the study and the traffic control changeover date, the desired level of confidence (95 percent) had to be reduced in the speed and compliance studies so that data requirements would be reasonable, given the time constraints imposed.

Sample size requirements were met or exceeded for all studies: sample sizes were in the range of 30 to 50 vehicles in all cases. In all cases, the t-test at the 95 percent level of significance was used.

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Traffic Control and Accidents at Rural High-Speed Intersections

KENNETH R. AGENT

In many instances, when rural high-speed highways are constructed, there are a number of at-grade intersections along the roadway. These rural high-speed intersections create accident potential because of the conflicting traffic movements. The objectives of this study were to (a) determine the types of traffic control measures used at rural high-speed intersections, (b) establish the type of accidents that occur at rural high-speed intersections, (c) discover the factors that contribute to these accidents, and (d) recommend traffic control measures that could most effectively decrease accident potential at such locations. The site characteristics, traffic control used, and accidents that occurred at 65 rural high-speed intersections were summarized. The differences that resulted when the right of way was controlled by stop signs instead of a traffic signal are discussed. The factors that contributed to the accidents and the characteristics of the accidents were analyzed. The data obtained at each intersection were summarized, and recommendations that could be used as a guide for implementing changes at other, similar intersections were made for the study locations. The accident analysis shows that providing the driver adequate warning of the intersection is of primary importance for this type of intersection. At signalized intersections, providing a proper change interval and maximizing the visibility of the signal heads are essential. The need to consider separate left-turn phasing also is shown.

In many instances, when rural high-speed highways, such as bypasses, are constructed, there are a number of at-grade intersections along the roadway. These rural high-speed intersections create accident potential because of the conflicting traffic movements. Various types of traffic control measures have been used. For example, one basic decision is whether the intersection should be controlled by stop signs (usually only the minor streets) or whether traffic signals should be used. Other traffic control measures, such as intersection control beacons, warning signs, channelization, and rumble strips, have been used.

There has been no systematic analysis of the results of using the various types of traffic control. There was a need for an analysis of the accidents that had occurred at several intersections of this type and a study to relate the accidents to the traffic control and other intersection characteristics to determine what types of traffic control may be used to reduce accident potential at such intersections.

The objectives of this study were to

- Determine the types of traffic control measures used at rural high-speed intersections,
- Establish the type of accidents that occur at rural high-speed intersections,
- Discover the factors that contribute to these accidents, and
- Recommend the traffic control measures that could most effectively decrease accident potential at such locations.

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PROCEDURE

A sample of rural high-speed at-grade intersections was selected from across Kentucky. The intersections were selected to provide a variety of traffic volume, roadway geometrics, and traffic control. In all, 65 study locations were selected. The list of locations was supplied by the Division of Traffic of the Kentucky Department of Highways. At a large number of the intersections, changes in traffic control had been implemented. In general, the intersections were high-volume locations. Several were on bypasses, and either a traffic signal or an intersection beacon was present at almost all the locations. The sample was not selected to represent the total sample of such intersections, which would include a high percentage of intersections that are unsignalized and without beacons.

A site visit was made to each intersection. The field data collected dealt with the intersection geometrics and the traffic control at the intersection and its approaches, including information such as intersection type, speed limit, right-of-way control, lighting, raised channelization, pavement markings, number of lanes, sight distance, signing, and traffic signal information. Data from the individual intersections were then coded into a computer file and summarized to show the typical characteristics of this type of intersection.

The dates of installation of traffic control, such as traffic signals and intersection control beacons, were determined. Accident data for several years were collected at each intersection, unless the intersection was new. Where appropriate, data were compared before and after the installation of a major traffic control device such as a traffic signal.

Accident rates were calculated for each intersection to determine the effect of any changes in traffic control as well as to determine an accident rate for each intersection. Intersections that had similar characteristics were combined to determine how factors such as the presence of a traffic signal influenced the accident rate.

Data from each accident report were coded into a computer file as well. This information was then summarized to obtain the characteristics of the accidents that occurred at each type of location. In addition to the information included on the police report, a "directional analysis" code was assigned to each accident to further describe the type of accident, and a "comments" or "accident description" code was assigned to add information concerning the contributing factors to the accidents. The accident description code was generally obtained after reviewing the commentary included on the accident report.

RESULTS

Site Characteristics and Traffic Control at Study Intersections

The information obtained from the site visit and the accident history analysis for each study intersection was tabulated. Of the 65 locations, 15 had three approaches and 50 had four approaches. The speed limit on the major roadway was 55 mph at 49 locations and 45 mph at 16 locations. The speed limit on the road classified as the minor roadway was 55 mph at 31 locations, 45 mph at 14 locations, and less than 45 mph at 20 locations.

A traffic signal was the right-of-way control at 47 locations, with stop sign control at 18 locations. Three of the 18 stop sign locations had four-way stop intersection control. All but two of the 18 stop sign locations had an intersection control beacon. The beacon was yellow for the through roadway and red for stop approaches. Some form of lighting was present at 18 locations.

There were 245 approaches at the 65 study intersections. The total number of lanes on the approaches varied from one to four. Typically, additional lanes were added at the intersection for turning. Many of the approaches (64 percent) had a separate left-turn lane. Approximately half of the approaches had some form of right-turn lane.

The grade and curvature on the majority of approaches was classified as straight and level. Only 7 percent of the approaches had a steep grade and only 6 percent had curves classified as sharp.

Almost all approaches (96 percent) had either a painted centerline or a lane line. Most approaches (78 percent) also had a painted edge line. Several approaches (44 percent) had snowplowable markers (either Stimsonite 96 or recessed markers) installed. Slightly over half of the approaches (56 percent) had either a mountable or nonmountable median. More approaches had a mountable median than a nonmountable one.

A small number of approaches (7 percent) had rumble strips installed. The rumble strips were installed at nine intersections, of which four were controlled by stop sign and five by a traffic signal. Of the 16 approaches with rumble strips, 11 were approaches to a traffic signal and five were approaches to a stop sign. Most approaches had a painted stop bar. Excluding through approaches at which a stop bar was not appropriate, 86 percent of the approaches had stop bars.

The sight distances for vehicles stopped on an approach to observe vehicles approaching on the cross roadways was summarized. That distance was estimated for traffic signal– and stop-sign–controlled approaches. Results indicate that sight distances were generally very good, especially for the minor approach to observe the major approach (where sight distance was estimated to be over 1,000 ft in 67 percent of the cases). These findings reflect previous observations that most approaches were generally straight and level. Sight distances were less than 500 ft at only 5 percent of minor roadway approaches to observe the major street approach, compared to 42 percent of major roadway approaches to observe the minor roadway approach.

The characteristics of the 47 signalized intersections were summarized. Of the major roadways, 60 percent had a separate left-turn phase; only 6 percent of the minor roadways had a separate left-turn phase. Protected-only phasing was used for all left-turn phasing. A green extension system (GES) had been installed at nearly all locations. The length of yellow on the major roadway was 4 sec or greater at all but one location, and there was a yellow time of 5 or more sec at 34 percent of the locations. On the minor roadway, the yellow time was under 4 sec at 34 percent of the locations, compared to five or more sec at 17 percent of the locations. A red clearance time was provided at 60 percent of the major roadway approaches, compared to 36 percent of the minor roadway approaches. The length of the red clearance time was generally (71 percent) in the range 1.0 to 1.5 sec with a typical time of 1 sec. A 12-in. lens was used for all major roadways and all but two minor roadway signal heads. All of the signal heads were mounted overhead. Backplates were used on 32 percent of the major roadway approaches and 11 percent of the minor roadway installations. A pedestrian signal was present at only two locations.

A comparison of the length of yellow time with the speed limit was conducted. There was a general increase in the length of yellow time as speed increased, but a yellow time of 4.0 sec was the most common length of yellow for all speed limits. The average yellow time increased from 3.6 sec for locations with a speed limit of 35 mph to 4.1 sec where the speed limit was 45 mph to 4.3 sec for a speed limit of 55 mph. According to the standard method used to calculate yellow time, a yellow time of 5.0 sec would be appropriate for a speed limit of 55 mph.

As part of the site inspection, the types of warning signs present on the approaches were noted. The presence of various warning signs was summarized by type of approach and speed limit. For approaches to a traffic signal, a "signal ahead" sign was present at 71 percent of the approaches, with a crossroad sign present at very few approaches. Only 9 percent of the approaches that had a speed limit of 55 mph did not have a warning sign, compared to 59 percent of the approaches that had a speed limit of less than 45 mph. Also, 32 of the 40 approaches to a stop sign (80 percent) had a "stop ahead" sign, and 19 of the 30 nonstop approaches (63 percent) at a stop sign–controlled intersection had a crossroad warning sign.

Descriptions of the signal ahead signs used were summarized. The most common signing used a single standard size sign. However, the second most common signing consisted of two 48-in. signs. At five approaches, a continuous flasher was placed on the signal ahead sign. At two approaches to one intersection, overhead "prepare to stop when flashing" signs were placed, with flashers that work when the red indication is displayed. This was the only active advanced warning device used at any of the study locations.

A summary of the types of stop signing used was also compiled. Several sign combinations were used and the most prevalent was one 48-in. sign. In addition to the usual ground-mounted location, some stop signs were placed overhead, and some in barrels placed on the pavement. Two intersections that had two-way stop control used a "cross traffic does not stop" plate in conjunction with the stop sign. One approach (an exit to a shopping center) did not have a stop sign. The most common stop ahead signing was a single standard size sign, although two standard size signs or one or two 48-in. signs were also used.

A summary of the crossroad warning signs used on nonstop approaches at stop sign–controlled intersections was made. Both one and two standard size or 48-in. signs were used, with the most common form being a single 48-in. sign with an advisory speed plate.

Accident Analysis by Type of Major Traffic Control

The current traffic control devices in place at each intersection were noted during the site visits. If an intersection beacon or traffic signal was present, the date of installation and the type of previous traffic control were determined. Dates of installation for other devices, such as signs or rumble strips, were not available. An accident analysis was conducted to compare the type of right-of-way control used. The three categories used were (a) stop sign with no intersection beacon, (b) stop sign with intersection beacon, and (c) traffic signal. Accident rates at the study locations were calculated as a function of right-of-way control.

The combined accident rates at intersections that had the designated type of right-of-way control are summarized in Table 1. The total number of locations exceeds the number of intersections included in the study because the right-of-way control had changed at some time during the study period at most of the intersections, resulting in data for more than one type of right-of-way control. The combined accident rate was similar for each type of right-of-way control.

A summary of the change in accidents when the right-of-way control was changed is given in Table 2. Of the 11 locations at which an intersection beacon was added to stop sign control, there were decreases in accidents at seven of the locations, compared to an increase at four locations. A statistical analysis revealed that two locations had a significant increase and two a significant decrease in accidents. The overall accident rate decreased from 1.1 to 1.0 ACC/MV when an intersection beacon was added.

Of the 16 locations, an equal number of locations experienced decreases and increases in accidents when a stop sign (without intersection beacon) was replaced with a traffic signal. Four intersections experienced a statistically significant increase, compared to three with a statistically significant decrease. The overall accident rate actually increased from 1.3 to 1.8 ACC/MV (because of a large number of accidents at one intersection) when the traffic signal was added.

For the 20 locations at which a stop sign with an intersection beacon was replaced with a traffic signal, accidents decreased at 12 locations, increased at 7 locations, and remained the same at 1 location. Also, there was a statistically significant decrease in accidents at six locations, compared to a significant increase at three locations. The overall accident rate decreased from 1.4 to 1.1 ACC/MV when the traffic signal was added. This was the result of a reduction in the number of right-angled accidents in which the side street vehicle pulled into the path of the through vehicle.

Data in the previous tables show a slight benefit with the installation of an intersection beacon. An overall benefit was observed when a traffic signal was installed, although results were not consistent. The intersections that had traffic signals and a high accident rate typically had a problem with opposing left-turn accidents.

Accident Characteristics

A summary of characteristics of accidents at the study intersections is presented in Table 3. The characteristics are compared to those for all intersection accidents statewide. A summary by directional analysis at the study locations revealed that angle accidents were the most common, followed by rear end and opposing left-turn accidents. When all intersection accidents were considered, angle accidents were still the most common, followed by rear end accidents. The largest difference in type of accident was the much higher percentage of opposing left-turn accidents that occurred at the study locations. The comparison of accidents at the study intersections with statewide intersection accidents indicated that accidents at the study locations were (a) more severe, (b) more likely to occur during darkness at an unlighted location, (c) less likely to occur during snow or ice conditions, and (d) more likely to involve failure to

TABLE 1 ACCIDENT SUMMARY BY TYPE OF RIGHT-OF-WAY CONTROL

Right-of-Way Control	Number of Locations	Accidents	Number of Vehicles (MV)	Accidents per MV	MV per Year
Stop sign	27	338	309	1.1	5.6
Stop sign with beacon	37	541	448	1.2	4.8
Traffic signal	46	1,290	1,058	1.2	6.1

TABLE 2 CHANGE IN ACCIDENTS WHEN RIGHT-OF-WAY CONTROL CHANGED

Change in Right-of-Way Control		Number of Locations	Change in Accidents/Year			Statistically Significant Change	
Original Control	New Control		Increase	Decrease	No Change	Increase	Decrease
Stop sign	Stop sign with beacon	11	4	7	0	2	2
Stop sign	Traffic signal	16	7	7	2	4	3
Stop sign with beacon	Traffic signal	20	7	12	1	3	6

TABLE 3 CHARACTERISTICS OF ACCIDENTS AT RURAL HIGH-SPEED INTERSECTIONS AND COMPARISON TO ALL INTERSECTION ACCIDENTS

		Percent in Given Category	
Variable	Category	Accidents at Study Intersections	Statewide Intersection Accidents[a]
Directional analysis	Angle	46.6	53.9
	Rear end	21.1	23.1
	Sideswipe	7.5	9.8
	Single vehicle	4.0	6.5
	Opposing left-turn	20.5	3.7
	Bicycle	0.1	0.8
	Pedestrian	0.0	0.8
	Other	0.2	1.4
Accident severity	Fatal accident	1.3	0.2
	Injury accident	36.0	23.6
	Property damage only	62.7	76.2
Light condition	Daylight	76.8	78.7
	Dawn-dusk	3.6	3.5
	Darkness, unlighted	10.2	5.0
	Darkness, lighted	9.4	12.8
Road surface condition	Dry	75.7	70.8
	Wet	20.7	20.4
	Snow-ice	3.6	8.8
Contributing factors	Unsafe speed	5.1	4.1
	Failure to yield right-of-way	40.5	28.2
	Disregard of traffic control	11.9	8.1
	Alcohol	4.2	3.6
	Defective brakes	3.1	2.0
	Glare	1.3	0.9
	Limited view	3.0	4.3
	Improper or nonworking traffic control	0.8	0.4
	Slippery surface	7.8	11.3

[a]In 1985, 39,980 accidents occurred at intersections.

yield right-of-way, disregard of a traffic control, or defective brakes as a contributing factor.

A comparison was made of the types of vehicles involved in accidents at the study locations versus all statewide accidents and statewide intersection accidents. The percentages were similar but did show a higher percentage of combination trucks involved in accidents at the study locations.

Characteristics of accidents involving passenger cars only or single-unit or combination trucks were tabulated. A higher percentage of accidents involved trucks at intersections controlled by a traffic signal than at intersections controlled by a stop sign. Compared to accidents involving only passenger cars, accidents involving a combination truck were more often associated with (a) increased accident severity, (b) wet, snowy, or icy pavement, (c) darkness with no lighting, and (d) sideswipe, single vehicle, and angle collisions. These accidents were less frequently associated with opposing left-turn and rear end collisions.

A summary of the characteristics of the accidents by type of major traffic control (stop sign, stop sign with beacon, and traffic signal) is presented in Table 4. The angle accident was the most common type for all types of traffic control, but its percentage decreased dramatically for intersections controlled by a traffic signal. Conversely, the percentage of rear end and opposing left-turn accidents increased substantially for traffic signal locations. The opposing left-turn accidents occurred almost exclusively on approaches that did not have protected left-turn phasing. Accidents were slightly less severe at intersections that had traffic signals. More accidents occurred during darkness and on wet pavements at traffic signal locations.

A summary of comments further describing the accident (accident description code) is given in Table 5. These comments would usually be obtained from a statement by a driver who had been involved in the accident or a comment from the investigating police officer. To help form a better understanding of the cause of the accident, the accident description narrative given on the police report was read and any relevant comments were coded and summarized. Although these comments by the police officer, driver, or both were not documented by a detailed accident reconstruction, it was felt that these remarks provided valuable insights into the causes of the accidents. The consistent types of comments found at certain locations added to their credibility. In a large number of accidents, no specific explanation was given for the action of the driver who was at fault. The summary in Table 5 places the descriptions of common accident types into various categories when possible. Some comments, such as defective brakes, would apply to more than one of the general description categories, so these types of comments were placed into the miscellaneous category. A common accident at locations not controlled by a traffic signal involved the side-street vehicle pulling into the path of a through vehicle. The most common explanation given was that the side-street driver, after stopping, did not observe the approaching through vehicle (although sight distance was very

TABLE 4 CHARACTERISTICS OF ACCIDENTS BY TYPE OF MAJOR TRAFFIC CONTROL

Variable	Category	Stop Sign	Stop Sign with Beacon	Traffic Signal
Directional Analysis	Angle	70.7	68.2	31.3
	Rear end	8.6	12.9	27.8
	Opposing left-turn	9.5	7.8	28.7
	Sideswipe	6.2	6.5	8.3
	Single vehicle	5.0	4.6	3.4
	Bicycle	0.0	0.0	0.2
	Other	0.0	0.0	0.3
Accident severity	Fatal accident	1.5	2.6	0.9
	Injury accident	37.2	39.6	34.1
	Property damage only	61.3	57.8	65.0
Light condition	Daylight	76.6	81.6	74.9
	Dawn-dusk	4.7	4.8	2.7
	Darkness, lighted	8.9	2.8	13.7
	Darkness, unlighted	9.8	10.8	8.7
Road surface condition	Dry	78.7	79.8	73.3
	Wet	18.3	16.5	23.0
	Snow-Ice	3.0	3.7	3.7

good in the large majority of accidents). The second most common occurrence was that the side-street vehicle failed to stop. Other statements given by the drivers of side-street vehicles included the following: thought the intersection was a four-way stop, thought the through vehicle was turning, or saw the through vehicle but misjudged the time available.

The most frequent comment when a driver disregarded a traffic signal was that he did not have enough time to stop when the signal changed to red. Other common observations noted on the police report were that both drivers thought their signal indication was green, the signal was not working properly or had been set to the flash mode, or one vehicle entered the intersection on yellow. In a few instances, the driver failed to observe the signal. It also was noted in a few cases that the signal had just been installed and was not expected by the driver.

For opposing left-turn accidents, the most common driver comments were that the driver did not see the opposing vehicle or the driver's vision was obscured (in many instances, by a vehicle waiting to turn left in the opposite direction). Other comments were that the time available to make the turn was misjudged or that the driver thought the green ball was a left-turn arrow.

The most common rear end accidents involved a vehicle stopped or stopping at a traffic signal or a vehicle sliding on a wet or icy pavement. Other common circumstances in rear end accidents involved a vehicle stopped to turn or in the process of turning, or a vehicle stopping abruptly at the onset of a yellow indication.

The most common type of sideswipe accident involved changing lanes. Other accidents of this type resulted when a vehicle turned from the wrong lane, a turning vehicle hit a stopped vehicle on the intersecting roadway, or a vehicle passed a turning vehicle.

The comments or "accident description" codes are summarized by type of traffic control in Tables 6 through 8. The problems of opposing left-turn accidents and accidents involving a driver who disregarded the signal indications are shown at signalized intersections, as are the large number of rear end accidents. The larger number of comments stating that the driver did not have adequate time to stop, both drivers thought they had a green indication, and one driver entered the intersection on a yellow indication point out the need for an adequate change interval.

The comments presented in Tables 7 and 8 show that the major problem at stop sign–controlled intersections involves a side-street vehicle stopping and then pulling into the path of a through vehicle. The most common explanation, as stated before, was that the driver of the side-street vehicle did not observe the through vehicle, even though sight distance was very good in most instances. In approximately 10 percent of those accidents it was noted that the side-street vehicle did not stop at the stop sign. It should be noted that the percentage of vehicles disregarding the stop sign was slightly higher at locations that had an intersection beacon than at locations without a beacon.

Recommendations at Study Locations

After the site visit information and accident history had been used as input, recommendations were made for operational improvements at the study intersections. Because these locations were selected to give a sample of rural high-speed intersections, the recommendations for operational improvements at these locations could be used as a guide for other similar intersections. Some sort of recommendation was made for all but five of the intersections. The recommendations were made on the basis of the accident history or as operational improvements according to a standard method of application for a traffic control device. An example of a recommendation on the basis of accident history was the addition of left-turn phasing, which was recommended for cases in which there was a large number of opposing left-turn accidents. Guidelines for an excessive number of such accidents have been established. An example of an operational improvement was the modification of the change interval to conform to that given in the standard procedure.

TABLE 5 SUMMARY OF COMMENTS DESCRIBING ACCIDENT

General Description	Comment	Number of Accidents
Side-street vehicle pulled into path of through vehicle at stop approach	Did not see through vehicle	182
	Side-street vehicle failed to stop	92
	Through vehicle lost control: single-vehicle accident	37
	Thought intersection was four-way stop	18
	Thought through vehicle was turning	18
	Saw through vehicle but misjudged time available	18
	Vision obscured	5
	No explanation given, or miscellaneous	273
Opposing left turn	Did not see opposing vehicle	66
	Vision obscured	40
	Saw opposing vehicle but misjudged time available	17
	Thought green ball was left-turn arrow	15
	Opposing vehicle disregarded red signal (separate left-turn phase)	12
	Opposing vehicle traveling at unsafe speed	10
	No explanation given or miscellaneous	243
Disregarded traffic signal	Not enough time to stop when signal changed to red	71
	Both drivers thought indication was green	44
	Signal not working properly	44
	Vehicle entered on yellow	41
	Failed to observe signal	19
	Slid into intersection due to wet or icy road	17
	New signal installation	12
	Emergency vehicle disregarded signal	8
	Alcohol involvement	7
	No explanation given or miscellaneous	125
Rear-end accident	Vehicle stopped or stopping at signal	81
	Slid on wet or icy pavement	66
	Vehicle stopped to turn or turning	41
	Vehicle stopped abruptly for yellow indication	38
	Side-street vehicle stopped when observed through vehicle	24
	Vehicle backing	22
	Starting to accelerate at signal	19
	Vehicle stalled	10
	No explanation given or miscellaneous	125
Sideswipe accident	Changing lanes	47
	Turned from wrong lane	28
	Turning vehicle hit stopped vehicle	21
	Passing turning vehicle	19
	Vehicles turning into same lane	8
	Pulling from side road	5
Miscellaneous comments	Defective brakes	54
	Single vehicle	46
	Right turn on red	16
	Sun obscured vision	16
	Fog or rain limited sight distance	9
	U-turn	7
	Road construction	7
	Bicycle involved	2

TABLE 6 SUMMARY OF COMMENTS CONCERNING ACCIDENTS AT TRAFFIC SIGNAL LOCATIONS

Comment	Number of Accidents
Turned left into path of opposing vehicle, no explanation	196
One vehicle disregarded traffic signal, no explanation	147
Rear end, no explanation	91
Rear end, vehicle stopped or stopping at signal	81
Disregarded traffic signal; not enough time to stop when signal turned red	71
Opposing left turn; did not see opposing vehicle	56
Rear end; wet or icy pavement	57
Defective brakes	44
Signal on flash or not working properly	44
Disregarded traffic signal; driver said intersection was entered on yellow	41
Rear end; first vehicle stopped for yellow	38
Sideswipe; changed lanes	31
Opposing left turn; vision obscured	31
Rear end; vehicle stopped to turn left	25
Rear end; vehicle starting to accelerate	19
Disregarded traffic signal; driver did not see signal	19
Sideswipe; turned from wrong lane	17
Right turn on red	16
Opposing left turn; thought green light was left turn phase	15
Opposing left turn; driver thought there was time to turn	14
Rear end; backing	13

TABLE 7 SUMMARY OF COMMENTS CONCERNING ACCIDENTS AT STOP SIGN LOCATIONS WITH BEACON

Comment	Number of Accidents
Side-street vehicle pulled into path of through vehicle	
No explanation	147
Did not observe through vehicle	118
Failed to stop	62
Opposing left turn; no explanation	26
Rear end; no explanation	22
Single vehicle lost control avoiding side-street vehicle	21
Rear end; first vehicle stopped when observed through vehicle	17
Side-street vehicle pulled into path of through vehicle; driver thought there was a four-way stop	15

TABLE 8 SUMMARY OF COMMENTS CONCERNING ACCIDENTS AT STOP SIGN LOCATIONS WITHOUT BEACON

Comment	Number of Accidents
Side-street vehicle pulled into path of through vehicle	
No explanation	122
Did not observe through vehicle	63
Failed to stop	30
Opposing left turn; no explanation	21
Rear end; no explanation	12
Single vehicle lost control avoiding side-street vehicle	9

A summary of the recommended operational improvements at the study locations is presented in Table 9. The recommendations are tabulated separately for intersections in which right-of-way is controlled by a traffic signal and for those controlled by a stop sign.

At intersections controlled by a traffic signal, the most common recommendation involved the change interval, with some modification recommended in either the yellow warning or red clearance interval at all such locations. The objective was to use a standard procedure to determine the change interval. Also, since disregard of the traffic signal was a problem, a recommendation was made that red clearance intervals should be used at all of this type of intersection. The recommendations generally involved increasing the length of the change interval. Another recommendation made at more than half of this type of intersection was the addition of backplates to the signal heads to increase their visibility. The addition of separate left-turn phasing was also recommended at several locations. As stated previously, the accident summary showed a large number of opposing left-turn accidents at this type of intersection, supplying the basis for this recommendation. Installing additional signs or modifying the warning signs also was recommended for several intersections as a means of providing additional warning to the drivers. Some type of recommendation was made for all of the traffic signal intersections. As noted previously, these were the result of either the accident history or the standard operational procedure.

At intersections controlled by stop signs, the major recommendations involved installing additional signs or modifying the warning signs to provide additional notice. Other recommendations included adding stop bars to inform motorists of the proper location to stop on the side street to have the maximum available sight distance and installing either rumble strips, transverse stripes, or post delineators on the stop approach to warn drivers that they would be required to stop. A recommendation was also made that a beacon be installed in one of the two stop sign–controlled intersections that lacked such a warning signal.

CONCLUSIONS

This study summarizes the intersection characteristics and types of traffic control at a number of rural high-speed intersections. The types of accidents that have occurred and the factors

TABLE 9 RECOMMENDED OPERATIONAL IMPROVEMENTS AT STUDY LOCATIONS

Type of Right-of-Way Control	Recommendation	Number of Intersections
Traffic signal (total: 47 intersections)	Modify change interval	47
	Add backplates	28
	Install or consider left-turn phasing	12
	Install or modify warning sign	10
	Place stopbar	4
	Install GES	3
	Add left-turn lane	1
	Install rumble strips or transverse stripes	1
Stop sign (total: 18 intersections)	Install or modify warning sign	10
	Place stopbar	4
	Install rumble strips, transverse stripes, or post delineators	4
	Consider intersection beacon	1
	None	5

NOTE: More than one recommendation may have been made for any intersection.

contributing to those accidents were also analyzed. These findings were used in recommending operational improvements at the study intersections. These recommendations may be reviewed for possible implementation. Also, because these locations were selected to provide a sample of rural high-speed intersections, the analyses and resulting improvements recommended for the study intersections may be used as guides for implementing changes at other, similar intersections. The types of improvements recommended can be related to the conditions at a specific intersection to determine what type of traffic control would result in the safest intersection.

The accident analyses show that providing the driver adequate warning of the intersection is of primary importance for this type of location. On the through street, it is important to provide warning of the presence of a crossroad because even with adequate sight distance, many accidents occurred in which the driver of the side-street vehicle did not observe the through vehicle and consequently pulled into its path. Stop bars should be placed on the stop approaches to encourage the drivers to stop at a location that would maximize their sight distance of vehicles on the through roadway. The number of side-street vehicles that did not stop at the stop sign illustrates the need for adequate warning and stop signs on the stop approach. Rumble strips, transverse stripes, or post delineators could be used in addition to signing at locations for which there is a particular problem with vehicles disregarding the stop sign. It was found that installing an intersection beacon would not eliminate the problem of drivers who disregarded the stop sign. Providing adequate sight distance is critical.

Of equal importance at signalized intersections is provision of a proper change interval and maximization of the visibility of the signal heads. A red clearance interval should always be provided for both roadways. A vehicle detection and extension timing scheme also should be included for the major roadway. Backplates should always be used to increase the visibility of the overhead signal heads.

These conclusions were based on the reasons given for accidents involving a vehicle disregarding a traffic signal. The most common explanation was that there was not enough time to stop when the signal changed to red. Other common reasons were that both drivers thought they had a green indication or that one driver entered the intersection on a yellow indication.

The need to consider separate left-turn phasing also is shown. The use of active advance warning signs should be considered at problem locations at which a large number of avoidable accidents have occurred.

DISCUSSION

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Statistical claims are made throughout this paper without support. For example, the claim is made that "A statistical analysis revealed that two locations had a significant increase . . . ". What statistical method was used? What was the level of significance? Given that several statistical methods exist for making this claim and the validity of these methods is based on assumptions, it is imperative that such a statement be followed by some description of the statistical method. The level of significance is obviously essential as well. Similar statements claiming "statistically significant" results appear throughout the text. As chair for the A3B11 Subcommittee on Statistical Methods in Accident Analysis, I felt compelled to comment on this all too common deficiency.

AUTHOR'S CLOSURE

The results of the only statistical analysis mentioned in the paper are summarized in Table 2 and the results described in the text. However, as correctly noted by the discussant, the statistical approach and level of significance were omitted. This information should have been included. The technique used was based on a Poisson distribution and a 95 percent confidence level.

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Field Studies of Temporary Pavement Markings at Overlay Project Work Zones on Two-Lane, Two-Way Rural Highways

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In response to FHWA's proposed rule requiring that all states use 4-ft pavement markings on 40-ft centers as temporary markings in highway work zones, NCHRP awarded a research contract to the Texas Transportation Institute to conduct field studies to compare the safety and operational effectiveness of 1-ft, 2-ft, and 4-ft temporary broken line pavement markings in work zones. The following scope and test conditions were specified by NCHRP: (a) surfacing operations on two-lane, two-way facilities; (b) field sites involving pavement overlays (not seal coats); (c) data collection during hours of darkness; (d) dry roadway conditions; (e) sites with both tangent and curve sections; (f) centerline stripe only (no edgelines); (g) use of a 40-ft pavement marking cycle; and (h) field tests in real or staged work zones that are open to traffic. Field studies were conducted at night at seven pavement overlay project sites on two-lane, two-way rural highways in Arkansas, Colorado, Oklahoma, and Texas. Traffic stream measures of effectiveness included vehicle speeds, lateral distance from the centerline, lane straddling, and erratic maneuvers. In-vehicle studies using paid driver subjects were conducted to supplement the traffic stream evaluation. The 1-ft and 2-ft striping patterns on 40-ft centers performed as well as the 4-ft pattern for centerline striping at night for the conditions studied: pavement overlay projects on rural two-lane, two-way highways with 2.0 degree horizontal curvature, level to rolling terrain, and average speeds between 50 and 62 mph. Although the driver subjects at six sites rated the 1-ft pattern to be the least effective on the average, there was no statistical difference in mean ratings or rankings among the three patterns.

The cost of temporary traffic control is significant for many construction, maintenance, and utility projects. With the prospects of continued inflation, limited resources, and high interest rates, it is imperative that all aspects of temporary traffic control be evaluated for economy in application and benefits to the public.

FHWA has issued guidelines and proposed changes to the *Manual on Uniform Traffic Control Devices* (MUTCD) regarding temporary markings for construction and maintenance areas (*1*). Markings that are less than the full standard marking pattern (10-ft stripe on 40-ft centers) would be permitted for broken lines, but the proposed changes would require a minimum pattern of 4-ft stripes on 40-ft centers (36-ft gaps), which is more than double what many states now specify. There has been concern that if the 4-ft markings were adopted as the national standard, they would significantly increase project costs. Table 1 is a summary of data abstracted from a survey conducted by the Traffic Engineering Section of the Arizona Department of Transportation in 1986. The number of states using each of 15 different temporary pavement striping patterns is presented. NCHRP awarded a contract to the Texas Transportation Institute (TTI) to determine whether the proposed 4-ft markings on 40-ft centers would actually result in significant safety and operational improvements in comparison to current practice (*2*).

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The specific objective of the research was to conduct field studies comparing the safety and operational effectiveness of 1-ft, 2-ft, and 4-ft temporary broken line pavement markings in work zones. To ensure that the findings would be applicable to situations in which this type of marking is most typically used, the following scope and test conditions were identified by NCHRP:

- Surfacing operations on two-lane, two-way facilities;
- Field sites involving pavement overlays (not seal coats);
- Data collection during hours of darkness;
- Dry roadway conditions;
- Sites with both tangent and curve sections;
- Centerline stripe only (no edgelines);
- Use of a 40-ft pavement marking cycle; and
- Field tests in real or staged work zones that are open to traffic.

STATE OF THE ART

A review of the literature revealed a variety of research projects on work zone traffic control. However, little information was available on the relative effectiveness of temporary pavement marking patterns in work zones.

Godthelp and Riemersma used a theoretical analysis to estimate the effectiveness of particular delineation systems as a reference in perceiving course and speed (*3*). Although this work is very general, it does provide insight into the interactions of the driver and the driving environment. The authors

TABLE 1 SUMMARY OF TEMPORARY PAVEMENT MARKING PATTERN PRACTICE, 1986

Length of Stripe (ft)	Length of Gap (ft)	Striping Interval (ft)	Number of States
10	30	40	11[a]
8	32	40	1
5	95	100	1
4	36	40	8
3	37	40	1
3	77	80	1
2	18	20	1
2	38	40	7
2	48	50	6
2	78	80	1
2	98	100	1
1	24	25	2
1	39	40	6
1	74	75	1
1	79	80	1
States using separate markings for curves			7
States using temporary edgelines			26

NOTE: Survey conducted by the Arizona Department of Transportation, Traffic Engineering Section (2).
[a]Five of the 11 states allow stripes less than 4 ft long under specified conditions.

point out the obvious fact that work zones represent discontinuities for drivers in terms of driving speed and roadway characteristics and consequently place special demands on the traffic control devices used in these areas. Godthelp and Riemersma also conducted laboratory experiments to preview the guidance effectiveness of delineation devices (*4*). Their findings suggested that placement of delineators at a level lower than the driver's eye height improved delineation efficiency and that chevron panels were particularly effective if other devices tended to be somewhat haphazardly placed.

Raised pavement markers (RPMs) for construction zone delineation were examined in Arkansas by Spencer (*5*). This study reported that RPMs provided excellent wet weather and nighttime reflectivity and appeared to be an effective means of maintaining safe traffic flow in work zones. Niessner (*6*) reviewed the practices of nine state highway agencies concerning the use of RPMs for temporary delineation in work zones. A wide variety of projects were included. The nine state highway agencies reported that the RPMs provided excellent nighttime temporary delineation, particularly on wet roads. In addition, the delineation was low cost and required little or no maintenance. In two projects reported by Niessner, an accident reduction occurred. Officials in the majority of the states said that they would continue to use the RPMs in construction zones after the study had been concluded.

The Texas Transportation Institute (TTI) investigated candidate temporary pavement marking treatments for use at work zones (Table 2) by determining the effects of each on various measures of driving performance (*7*). The studies were conducted on a 6-mi test track at TTI's proving ground facility. Ten candidate temporary pavement marking treatments were evaluated during daylight hours, and seven of the ten candidates were also evaluated at night. The candidate treatments included patterns with stripes, RPMs, and combinations of both. Treatment 1 (4-ft stripes with 36-ft gaps) was considered to be the control condition in the studies.

TABLE 2 TEMPORARY PAVEMENT MARKING TREATMENTS EVALUATED BY TTI IN PROVING GROUNDS SETTING

Treatment	Description
1[a]	4-ft stripes (4 in. wide) with 36-ft gaps (control condition)
2[a]	2-ft stripes (4 in. wide) with 38-ft gaps
3[a]	8-ft stripes (4 in. wide) with 32-ft gaps
4[a]	2-ft stripes (4 in. wide) with 18-ft gaps
5[a]	Four nonreflective RPMs at 3⅓-ft intervals with 30-ft gaps *and* reflective marker centered in alternate gaps at 80-ft intervals
6[a]	Three nonreflective and one reflective RPMs at 3⅓-ft intervals with 30-ft gaps
7	2-ft stripes (4 in. wide) with 48-ft gaps
8	Treatment 2 *plus* RPMs at 80-ft intervals
9[a]	Two nonreflective RPMs at 4-ft intervals with 36-ft gaps *plus* one reflective RPM centered in each 36-ft gap
10	1-ft stripes (4 in. wide) with 19-ft gaps

[a]Treatments evaluated both day and night.

The major findings from the daytime proving ground studies were as follows:

- The vehicle speed and distance data failed to provide any basis for selection among the 10 treatment conditions. Because of the large variability within subjects and the small magnitude of change in the measures of effectiveness (MOEs), the analysis of the objective data failed to reveal any practical significant difference in treatments.
- Two treatments with short (2-ft) stripes and long gaps (48- and 38-ft intervals) were associated with missed curves and with a few wide deviations to the right of the centerline.
- The subjective ratings tended to support the data just mentioned. Drivers indicated that it was difficult to follow curves with short stripes or long gaps and preferred the 8-ft stripe with 32-ft gap pattern and the RPMs.

The major findings and conclusions of the nighttime studies were as follows:

- Speed and distance performance data for the nighttime studies were not sufficiently different to provide a basis for ranking the treatments. Speed profiles for night driving were comparable to those for the daytime studies.
- Erratic maneuver data also revealed no significant differences with respect to treatments.
- Drivers rated the 8-ft stripes with 32-ft gaps as the best and the 2-ft stripes with 38-ft gaps as the poorest of the four striping patterns tested. The three RPM treatments tested were all judged by drivers to be highly effective.
- Drivers rated the baseline treatment (4-ft stripes with 36-ft gaps) to be inferior to the three RPM treatments tested.
- In general, the nighttime studies supported the findings of the daytime studies in ratings of effectiveness. However, neither study found that the performance data provided any basis for ranking the seven treatments.

The TTI researchers emphasized that studies performed on proving grounds are no substitute for real-life field studies. Proving ground studies can help identify and eliminate candidate treatments that are considerably ineffective relative to the others. However, because subject drivers tend to do their best when tested in a proving ground setting, the test is not generally sensitive enough to discern small differences between candidate treatments. Field studies must be conducted to measure these differences.

FIELD STUDIES

Field Study Plan

A brief description of the study plan is given in the following sections. More complete details are provided elsewhere (2).

Study Sites

Field studies were conducted at seven pavement overlay construction projects on two-lane, two-way rural highways. The allocation of study sites was four in Texas and one site each in Arkansas, Colorado, and Oklahoma. The order of studies was as follows: work at sites 4, 1, 3, and 2 in Texas, followed by work at the sites in Oklahoma, Arkansas, and Colorado.

The characteristics of the sites are summarized in Table 3. All sites had 12-ft lanes with paved shoulders. The only visible markings were centerlines made of yellow reflective tape. Annual average daily traffic rates (AADTs) ranged between 2,530 and 6,700 vehicles. Three of the sites were located in highway sections with relatively level terrain, and four sites were in sections with rolling terrain. All of the sites included a horizontal curve and a tangent section. The degree of curvature was 2.0 degrees at six sites and 3.0 degrees at one site. Some of the sites included sections that would be marked again as no-passing zones after the pavement overlay construction work was completed.

Operational Measurements

Traffic stream measurements included vehicle speed, lateral distance from the centerline (measured from the centerline to the outer edge of the left front tire), lane encroachment (straddling centerline), and erratic maneuvers (e.g., abrupt swerving, excessive slowing, stopping, etc.). Vehicle speed, lateral distance, and lane encroachment data were collected by using an automated data collection system developed by TTI. Tapeswitches attached to the pavement surface were wired to computers housed in vehicles that were parked off the roadway as far from the operating lanes as possible.

A schematic of the tapeswitch placements and data collection configuration for a typical field study is shown in Figure 1. The specifics concerning the installation were as follows:

- One Z-type tapeswitch configuration was installed at a base station located upstream of the test section to record times, speeds, lateral distances, and encroachments of vehicles on a roadway section containing the highway agency's existing temporary centerline pavement marking pattern.
- Three Z-type tapeswitch configurations were located in the curve section at the 1/4, 1/2, and 3/4 distance points from the beginning of the curve.
- Three Z-type tapeswitch configurations, spaced about 400 ft apart, were located in a tangent section.
- One double tapeswitch configuration was located in the opposing lane near the curve tapeswitches and one near the tangent tapeswitches. The double tapeswitches recorded the times and speeds of opposing vehicles.

In addition to data recorded with the automated system, field personnel located near the two computer systems observed erratic maneuvers within the horizontal curve and tangent sections.

In-Vehicle Driver Response

In-vehicle studies were conducted to supplement the traffic stream evaluation. Each of 27 paid driver subjects (four at each of six sites and three at one site), recruited from the local areas, was accompanied by a TTI study administrator as he or she drove through one of the seven test sites. Each subject drove through a study site on each of three nights while traffic stream data were being collected for the 1-ft, 2-ft, and 4-ft striping patterns. The administrator recorded driver comments and erratic maneuvers and administered a post-drive-through survey each night to obtain additional information. Details of the survey forms and instructions can be found elsewhere (2). Age and gender distributions of the driver subjects are presented in Table 4 in relation to a national distribution of drivers (8). The first number shown is the proportion needed to match the national demographic in age and gender.

A speed/distance recorder, used at the four Texas sites, provided information necessary for developing driver speed profiles. Electronic problems in the test vehicle prevented TTI researchers from recording similar data in Oklahoma, Arkansas, and Colorado.

Experimental Design

Each marking pattern was tested at each site on consecutive week nights except when inclement weather or equipment

TABLE 3 STUDY SITE CHARACTERISTICS

Site	State	Route	Location	AADT	Study Direction	Section Length (ft)	Curve Length (ft)	Curve Direction	Degree of Curve	Lane/ Shoulder Width (ft)	Terrain
1	Texas	US 190	1 mi north of Milano	5,200	Northbound	6,700	1,022	RH	2.0	12/10	Level
2	Texas	SH 36	1 mi west of Brenham	9,600	Southbound	3,700	1,600	LH	2.0	12/10	Rolling
3	Texas	SH 276	1.5 mi east of Rockwall	6,000	Eastbound	2,530	1,831	RH	2.0	12/10	Level
4	Texas	US 96	Silsbee Bypass	5,000	Southbound	3,880	1,850	RH	2.0	12/10	Level
5	Oklahoma	US 64	Eastern edge of Sallisaw	3,000	Eastbound	3,640	1,060	RH	2.0	12/5	Rolling
6	Arkansas	US 71	4.5 mi north of Wickes	3,750	Northbound	3,200	700	LH	3.0	12/4	Rolling
7	Colorado	US 160	27 mi east of Durango	2,750	Eastbound	3,000	1,260	RH	2.0	12/5	Rolling

NOTE: All sites were at overlay projects on two-lane, two-way rural highways. The centerline stripes were the only markings on the highway sections.

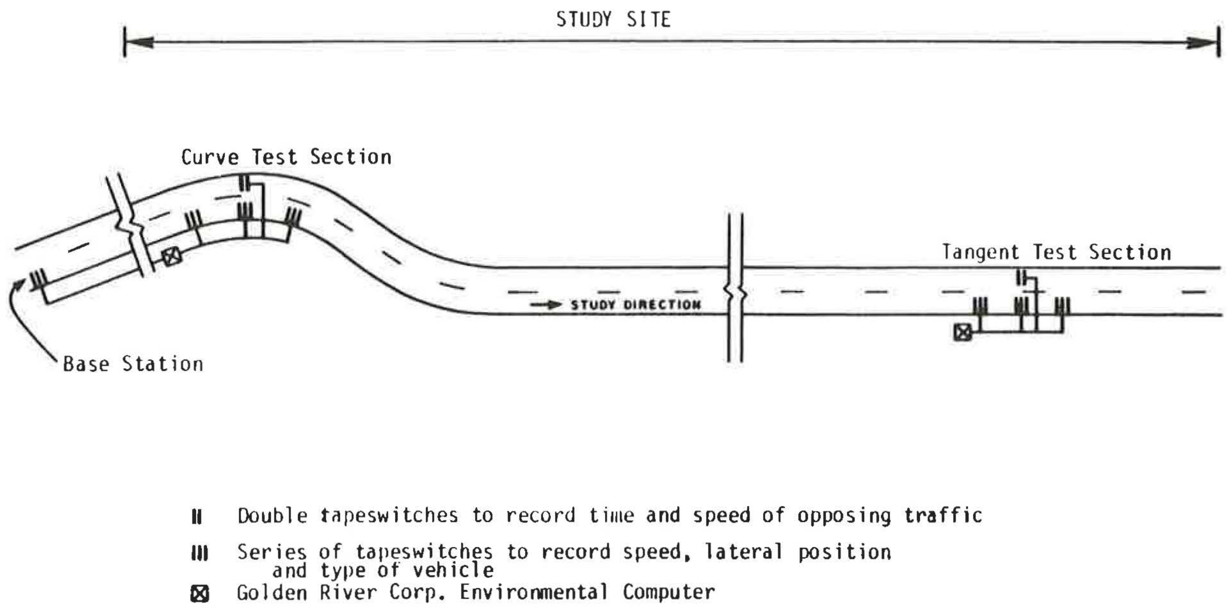

FIGURE 1 Data collection configuration.

TABLE 4 AGE AND GENDER DISTRIBUTION OF SUBJECT DRIVERS

	Age[a]					
	≥ 24	25–34	35–44	45–54	55 ≥	Total
Males	3/4	3/3	3/2	2/2	3/3	14/14
Females	2/3	3/1	3/3	2/3	3/3	13/13
Total	5/7	6/4	6/5	4/5	6/6	27/27

[a]Number of subjects needed/number of subjects tested.

problems prevented testing. The pavement tape used to mark the test centerline was manually removed each day and replaced with a new striping pattern in time for the nighttime studies. Removal of the stripes each day did not leave any visible markings on the pavement, regardless of the order in which the patterns were studied.

The order of striping patterns is shown in Table 5. The order of treatments was counterbalanced across sites according to a Latin square design. Note that each treatment was scheduled at least twice in each order position. Because there were seven sites rather than six, one treatment appeared three times in each order position. In regard to field data, two exceptions to the original counterbalanced design were required. Weather problems required an adjustment in the order during the studies at Site 7 (Colorado). The high potential of having to abandon the

TABLE 5 ORDER OF STRIPING PATTERN TESTS

	Pattern, ft		
Site	First Night	Second Night	Third Night
1	2	1	4
2	4	2	1
3	1	4	2
4	4	2[a]	1
5	1	4	2
6	2	1	4
7	4	1	2

[a]No data available.

studies at Site 7 because of prolonged inclement weather led to a decision to study the 1-ft stripe (instead of the 2-ft stripe) immediately after studying the 4-ft stripe. Loss of data for the 1-ft stripe was considered to be more critical than the loss of data for the 2-ft stripe. The rationale was that comparisons could be made between the two extreme test striping patterns. If no differences were found between the 1-ft and 4-ft patterns (as had been the case in the Texas studies), then it could be concluded that there would be no differences between the 2-ft and 4-ft patterns. The weather, however, did clear long enough to collect data for the 2-ft stripe after data were collected for the 1-ft stripe.

At Site 4, the initial study site, field data on the second night were lost due to inclement weather and equipment problems. The subject questionnaire was administered under all treatments, and the lower ratings at Site 4 on the second night may be expected to partially reflect the inclement weather.

Analysis Approach

Practical Speed and Lateral Distance Differences

Because of the large sample size expected from the field studies, it was anticipated that statistical significance would be detected in even small differences in average speeds and lateral distances between the 1-ft, 2-ft, and 4-ft treatments. The concern of the research team was to ensure that the results would be interpreted not only statistically but also from a practical standpoint. For example, during analysis of the differences between two of the temporary pavement marking patterns, a difference in average speeds of 1 mph might be found to be statistically significant because of the large sample size. However, from a practical standpoint, a 1-mph speed difference would be rather meaningless. It therefore became necessary to identify a speed differential that would be considered acceptable in a practical sense. On the basis of the many years of research and operational experience of the research team and discussions with several other traffic safety and operations experts, a speed difference of 4 mph or greater was chosen as practically significant. Similarly, the team chose differences in lateral distance of 1 ft or greater occurring at four of the six curve and tangent sensor stations as practically significant.

Tracing Vehicles: A More Powerful Analysis Design

Another important feature of the analysis was the analysis experimental design. The ability to trace individual vehicles from the base sensor station through each of the other six sensor stations (three in the curve and three in the tangent) allowed the use of a matched or paired comparison statistical design that significantly increased the power of detecting significant differences among the 1-ft, 2-ft, and 4-ft striping patterns. This increased power translated into a reduced sample size requirement for detection of differences with the same precision as an unmatched design (vehicles are not traced through the sensor stations). For example, a sample size of 63 matched (traced) vehicles would be equivalent to 125 unmatched vehicles (about 2 times as many) when detecting average speed differences of 4 mph at the 0.05 level of significance and 80 percent power.

FIELD STUDY RESULTS

This section of the paper discusses the combined results from the seven field study sites. Details for each study site are presented elsewhere (2).

Sample Size

Vehicle sample sizes by site and pavement marking pattern are presented in Table 6. The sample sizes are listed in four groups: (a) the total number of vehicles observed during the studies, (b) the number of traced vehicles, (c) the total number of vehicles with headways of 4 sec or longer, and (d) the number of traced vehicles with headways of 4 sec or longer. A total of 3,697 vehicles were sampled at the seven overlay study sites. Of these vehicles, 2,883 were traced through at least the base and the three curve stations (2,814 were traced through all seven stations), 2,803 had headways of 4 sec or longer, and 2,518 vehicles with headways of 4 sec or longer were traced through at least the base and the three curve stations (2,443 vehicles with headways of 4 sec or longer were traced through all seven stations).

Traced Vehicles with Headways > 4 Sec

For each study site, Tables 7 and 8 summarized the average speeds and the average lateral distances from the centerline at the base and the three curve and three tangent sensor stations. The curve stations, CURVE-1, CURVE-2, and CURVE-3, were located at the $1/4$, $1/2$, and $3/4$ distance points from the beginning of the horizontal curve. The tangent stations, TAN-1, TAN-2, and TAN-3, were located at 400-ft spacings, with the exception of Site 7 (Colorado). The spacing at Site 7 was reduced to 250 ft because the available tangent section was short.

TABLE 6 SAMPLE SIZE

Stripe	Site 1 (TX)	Site 2 (TX)	Site 3 (TX)	Site 4 (TX)	Site 5 (OK)	Site 6 (AR)	Site 7 (CO)	TOTAL
TOTAL NUMBER OF VEHICLES OBSERVED								
1-ft	125	184	349	43	118	146	138	1,103
2-ft	170	123	659	**	169	150	100	1,371
4-ft	148	192	434	38	125	149	137	1,223
								3,697
NUMBER OF TRACED VEHICLES								
1-ft	110	79	313	43	82	125	82	834
2-ft	137	97	611	**	116	95	73	1,129
4-ft	89	72	415	38	89	111	106*	920
								2,883
NUMBER OF VEHICLES WITH HEADWAYS ≥ 4 SECONDS								
1-ft	112	150	294	43	105	126	91	921
2-ft	143	99	524	**	129	95	76	1,066
4-ft	89	68	361	38	109	104	47	816
								2,803
NUMBER OF TRACED VEHICLES WITH HEADWAYS ≥ 4 SECONDS								
1-ft	106	62	271	43	80	114	73	749
2-ft	130	83	488	**	104	84	70	959
4-ft	84	58	344	38	84	97	(105)*	810
								2,518

*There were a maximum of 106 (105) vehicles traced through the curve only; 37 (30) vehicles were traced through all 7 stations.

**No data available.

TABLE 7 AVERAGE SPEEDS IN MPH FOR TRACED VEHICLES WITH HEADWAYS ≥ 4 SEC

N	STRIPE	BASE	CURVE-1	CURVE-2	CURVE-3	TAN-1	TAN-2	TAN-3
SITE 1 - TX								
106	1-ft	61	58	58	60	59	59	**
130	2-ft	60	58	59	61	59	60	**
84	4-ft	62	58	58	60	59	59	**
SITE 2 - TX***								
62	1-ft	56	56	56	**	56	57	56
83	2-ft	56	56	57	55	56	55	53
58	4-ft	57	57	58	57	57	57	55
SITE 3 - TX								
271	1-ft	57	54	53	53	**	**	**
488	2-ft	57	54	54	53	**	**	**
344	4-ft	58	55	56	54	**	**	**
SITE 4 - TX								
43	1-ft	**	56	55	56	**	**	**
0	2-ft	**	**	**	**	**	**	**
38	4-ft	**	59	57	58	**	**	**
SITE 5 - OK								
80	1-ft	58	55	55	53	55	56	56
104	2-ft	56	54	54	53	55	55	55
84	4-ft	56	53	52	52	53	54	54
SITE 6 - AR								
114	1-ft	57	58	56	54	55	56	54
84	2-ft	56	56	55	53	54	56	54
97	4-ft	57	57	56	55	54	55	54
SITE 7 - CO***								
73	1-ft	53	51	53	53	51	51	51
70	2-ft	54	**	53	54	52	52	52
105	4-ft	54	53	54	55	54	54	55

**No data available.

***Tangent data collection stations preceded the curve stations.

The statistical analysis revealed that there were no significant differences in average speed among the 1-ft, 2-ft, and 4-ft striping patterns, with the exception that statistically significant differences were found at the Site 1 CURVE-2 sensor location and at the Site 3 CURVE-2 and CURVE-3 sensor locations. However, the average speed differences at these three sensor stations were 2.5 mph or less and were not considered to be practically significant.

The analysis of variance procedure assumes that the variability among treatment groups is homogeneous. This assumption was tested using the Sheffé F-test for comparing population variances. Basically, as a rule of thumb, if sample standard deviations are within a factor of 2 (i.e., if the minimum sample standard deviation doubled does not exceed the maximum sample standard deviation), then the variance for all groups can be considered statistically equal. A review of the data revealed that the standard deviations at only two stations at one site (Site 2) were greater by a factor of 2. Therefore it was felt that the assumption of homogeneous variance is valid for these data.

Analysis of the lateral distance data also revealed that there were no statistical or practical differences in lateral distance from the centerline among the three striping patterns. The average lateral distance differences between the 1-ft and 2-ft striping patterns and the 4-ft striping pattern were only 0.4 ft or less.

Analysis of the vehicle encroachment (straddling) data did not reveal any patterns either. Centerline encroachment at the sensor stations was extremely infrequent and sporadic. The field observers noted that the few cases of vehicle encroachment that did occur were due to passing maneuvers and other factors unrelated to the centerline striping pattern. Erratic maneuvers caused by the striping patterns were also essentially nonexistent.

As additions to the data just mentioned, speed profiles of the subject drivers at the four Texas sites were developed to determine whether the speed patterns could help distinguish differences between the three pavement marking patterns. The speed profile sample size at each site (3 or 4 subjects) was not

TABLE 8 AVERAGE LATERAL DISTANCES IN FT FROM CENTERLINE FOR TRACED VEHICLES WITH HEADWAYS ≥ 4 SEC

N	STRIPE	BASE	CURVE-1	CURVE-2	CURVE-3	TAN-1	TAN-2	TAN-3
SITE 1 - TX								
106	1-ft	4.0	4.6	4.2	4.9	3.3	3.6	**
130	2-ft	4.0	4.6	4.4	5.2	3.5	3.6	**
84	4-ft	4.3	4.9	4.7	5.3	3.7	4.0	**
SITE 2 - TX***								
62	1-ft	4.8	3.2	3.1	**	3.3	2.7	2.6
83	2-ft	4.2	3.2	3.0	2.9	3.7	2.8	2.8
58	4-ft	3.6	3.1	2.9	2.8	3.0	2.9	2.6
SITE 3 - TX								
271	1-ft	4.4	4.7	5.0	4.8	**	**	**
488	2-ft	3.8	4.8	4.7	3.7	**	**	**
344	4-ft	4.1	4.9	5.2	4.0	**	**	**
SITE 4 - TX								
43	1-ft	**	5.5	4.7	6.3	**	**	**
0	2-ft	**	**	**	**	**	**	**
38	4-ft	**	5.8	4.9	6.1	**	**	**
SITE 5 - OK								
80	1-ft	2.6	3.5	3.3	4.8	3.6	3.2	4.4
104	2-ft	2.7	3.6	3.4	5.1	3.3	3.0	4.1
84	4-ft	2.6	3.6	3.5	5.1	3.6	3.0	3.7
SITE 6 - AR								
114	1-ft	2.1	2.3	2.3	2.0	1.9	2.2	2.0
84	2-ft	2.0	2.4	2.0	1.9	2.0	2.0	1.7
97	4-ft	1.8	2.2	2.1	1.9	2.1	2.5	1.9
SITE 7 - CO***								
73	1-ft	4.1	3.0	4.0	3.2	2.5	2.4	3.2
70	2-ft	2.7	3.2	**	3.0	2.3	2.2	2.7
105	4-ft	3.0	3.9	2.8	3.4	2.4	2.0	2.6

*Measured from the centerline to the outer edge of the left front tire.

**No data available.

***Tangent data collection stations preceded the curve stations.

large enough for the performance of statistical analyses. However, visual inspection of the speed profiles revealed no consistent speed patterns that indicated any differences between the three striping patterns. The speed profiles showed considerable variability and seemed to be indicators of individual driving habits rather than the results of differences among the three striping patterns.

In summary, speed, lateral distance, encroachment, erratic maneuver, and speed profile data for the sample of vehicles with headways of 4 sec or more indicated that there were no differences in driver performance between the 1-ft, 2-ft, and 4-ft striping patterns.

Subject Evaluations

Table 9 presents the ratings by the subject drivers of the three pavement marking treatments across the seven study sites. Each driver was asked to rate the markings on a scale as follows: 4- extremely effective, 3- effective, 2- satisfactory, 1- not very effective, and 0- unsuitable and possibly dangerous. With one exception, there were four driver/subjects per site, so the maximum total rating per striping pattern per site was 16. If all four subjects judged the treatment pattern effective, the total rating was 12. If all rated it satisfactory, the total rating was 8.

Site 1 had only three subjects. To include these data (shown in Table 9 within parentheses) with the other data, it was necessary to extrapolate the ratings as if there had been four subjects. The same procedure was followed with the ranking data.

The results showed that few drivers used the 4 rating and no one used the 0 rating. The average rating across all studies was a 10.8, slightly below the 12.0 (effective) rating. At two sites (2 and 7), mean ratings were 12.7 across treatments, whereas three sites had mean ratings of 9.3 to 9.7. There appeared to be a slight trend toward a relationship between order of effectiveness and length of the stripe. At only one site was the 1-ft stripe

TABLE 9 SUMMARY OF RATINGS OF PATTERNS AT SEVEN SITES

Treatment	Site 1[a] (Tex.)	Site 2 (Tex.)	Site 3 (Tex.)	Site 4 (Tex.)	Site 5 (Okla.)	Site 6 (Ark.)	Site 7 (Colo.)	Total	Mean
1 ft	9.3 (7)[b]	12	7	12[c]	7	8	13	68.3	9.8
2 ft	12 (9)[b, c]	13[c]	11[c]	8	11[c]	9	11	75.0	10.7
4 ft	10.4 (8)[b]	13[c]	11[c]	10	10	12[c]	14[c]	80.4	11.5
Mean	10.8	12.7	9.7	10.0	9.3	9.7	12.7	223.7	10.8

CODE: 16 = extremely effective, 12 = effective, 8 = satisfactory, 4 = not very satisfactory. Max = 16, min = 0.
[a]Site 1 had only three subjects. Extrapolations were made on the basis of four subjects for comparisons of ratings.
[b]Original rating based on three subjects.
[c]Best rating.

judged to be most effective. At two sites there was a strong preference for the 4-ft stripe, but overall, subjects lacked a strong preference between the 2- and 4-ft striping patterns. The variability across studies led to no significant difference between ratings.

The data in Table 10 summarize the ranking data across studies. A ranking of most effective was assigned a 1, next most effective a 2, and least effective a 3. Hence the best possible ranking at a site was a 4, and the poorest possible rank a 12. The mean ranking for each marking pattern across studies varied only slightly from the mean of 8.0. Again, the 1-ft stripe was poorest (9.2), but it was not significantly different from the 2-ft and 4-ft stripes.

After the studies at Sites 1 and 4 (Texas), the subject questionnaire was modified for the next five sites in an attempt to assess whether the drivers were even aware that there were differences in the three striping patterns. Drivers were instructed after the third and final night to rank the stripes in length, in spacing between them, and in brightness. They were not told in advance to look for these particular features, but it was important to know whether the drivers were basing the effectiveness ratings and rankings on some design feature rather than on extraneous factors unique to the site, such as the weather or the traffic.

Table 11 shows that drivers at all five sites ranked the 4-ft stripe as being longer than the 1-ft stripe. However, the drivers at Sites 2 and 5 had difficulty in discriminating the 2-ft and 4-ft stripes, and drivers at Sites 6 and 7 could not distinguish differences in the 1-ft and 2-ft lengths. In general, there was a trend toward being able to distinguish differences in length, even though the drivers were not asked to do so in advance.

Table 12 presents the estimates of spacing between stripes. Drivers at all but one of the sites reported that the 39-ft spacing (1-ft stripe) was greater than either the 38-ft spacing (2-ft stripe) or 36-ft spacing (4-ft striping). Strangely, they could not discriminate between the 38- and 36-ft spacing even though there was a 2-ft difference, while the 1-ft difference was detected.

Table 13 shows the estimates of brightness. Subjects at two sites were convinced that the 2-ft stripe was brightest and the 1-ft was the dimmest, but at the other three sites there was virtually no difference. Overall, drivers could not discriminate among brightness levels.

Drivers' Comments

Drivers' comments were highly variable and often dwelled on situational factors unrelated to the pavement marking patterns.

TABLE 10 SUMMARY OF RANKINGS OF PATTERNS AT SEVEN SITES

Treatment	Site 1[a] (Tex.)	Site 2 (Tex.)	Site 3 (Tex.)	Site 4 (Tex.)	Site 5 (Okla.)	Site 6 (Ark.)	Site 7 (Colo.)	Total	Mean
1 ft	10.7 (8)[b]	7[c]	12	6.5[c]	11	7[c]	10	64.2	9.2
2 ft	7.3 (5.5)[b]	7[c]	6.5	7	6[c]	9	7[c]	49.8	7.1
4 ft	6 (4.5)[b, c]	10	5.5[c]	7.5	7	8	7[c]	51.0	7.3
Mean	8.0	8.0	8.0	7.0	8.0	8.0	8.0	165.0	7.9

CODE: 4 = best, 8 = second best, 12 = worst. Max = 4, min = 12.
[a]Site 1 had only three subjects. Extrapolations were made on the basis of four subjects for comparisons of ratings.
[b]Original rating based on three subjects.
[c]Best rating.

TABLE 11 STRIPE LENGTH ESTIMATES AT FIVE SITES

Treatment	Site 2 (Tex.)	Site 3 (Tex.)	Site 5 (Okla.)	Site 6 (Ark.)	Site 7 (Colo.)	Total	Mean
1 ft	5	5	5	7	7	29	5.8
2 ft	10[a]	7	10[a, b]	7	7	41	8.2
4 ft	9	12[a]	10[a, b]	11[a]	11[a]	53	10.6

CODE: 3 = longest (12 max), 2 = midlength (8 mid), 1 = shortest (4 min). Note that questions of length, spacing, and brightness were not asked in first two studies. Mean rank across treatments is 8 by procedure.
[a]Longest stripe.
[b]One tie for longest.

TABLE 12 SPACING LENGTH ESTIMATES AT FIVE SITES

Treatment	Site 2 (Tex.)	Site 3 (Tex.)	Site 5 (Okla.)	Site 6 (Ark.)	Site 7 (Colo.)	Total	Mean
1 ft (39-ft space)	10[a]	12[a]	12[a]	8	9[a]	51	10.2
2 ft (38-ft space)	8	5	5[b]	8	8	34	6.8
4 ft (36-ft space)	6	7	6[b]	8	7	34	6.8

CODE: 3 = longest space (12 max), 2 = midlength space (8 mid), 1 = shortest space (4 min).
[a]Longest space.
[b]One tie for least spacing.

TABLE 13 STRIPE BRIGHTNESS ESTIMATES AT FIVE SITES

Treatment	Site 2 (Tex.)	Site 3 (Tex.)	Site 5 (Okla.)	Site 6 (Ark.)	Site 7 (Colo.)	Total	Mean
1 ft	9[a]	5	6	9[a]	8	37	7.4
2 ft	8	11[a]	10[a]	7	8	44	8.8
4 ft	7	8	8	8	8	43	8.6

CODE: 3 = brightest (12 max), 2 = midbrightness (8 mid), 1 = dimmest (4 min).
[a]Brightest.

However, when comments on length, spacing, brightness, or effectiveness were volunteered, these comments were generally reflected in the drivers' rankings, ratings, and estimates.

To summarize, the 1-ft, 2-ft, and 4-ft patterns were all rated satisfactory to effective. There was no statistical difference in ratings, but the trend was toward judging the 1-ft stripe as less effective (only at one site was the 1-ft stripe judged most effective). There was no difference in rankings, but again, the trend was toward the 1-ft stripe being ranked as slightly poorer. At four of seven sites, it was ranked as much poorer than the other two lengths.

Drivers were able to distinguish the lengths of the 4-ft and 1-ft stripes but had difficulty distinguishing between the 1- and 2-ft lengths and between the 2- and 4-ft lengths. They could tell that the 39-ft spacing, associated with the 1-ft stripe, was the longest, but they could not tell the difference between the 38-ft and 36-ft spacings. Brightness estimates were virtually random. Had the drivers been instructed in advance to concentrate on these features or if the patterns had been viewed successively on the same night, performance might have been better. However, to have done so might have biased the drivers toward basing their effectiveness and ranking judgments on these features.

After studies at three field sites in Texas, the subject questionnaire was modified to obtain direct statements from the subjects about the adequacy of the 1-ft, 2-ft, and 4-ft striping patterns. The following question was added and asked each night after the drive through: "Does this marking pattern provide adequate path delineation?"

The responses to this question are summarized in Table 14. From the table, it can be seen that 13 of the 16 subjects interviewed stated that the 1-ft striping pattern did provide adequate delineation, 15 stated that the 2-ft striping pattern was adequate, and all 16 believed that the 4-ft striping pattern was adequate. In general, drivers slightly preferred the longer stripes, but there is no compelling evidence that the 2- or 4-ft stripes are superior to the 1-ft stripe.

TABLE 14 SUBJECTS STATING THAT STRIPING PATTERN PROVIDED ADEQUATE DELINEATION–FOUR SITES

Treatment	Site 1 (Tex.)	Site 5 (Okla.)	Site 6 (Ark.)	Site 7 (Colo.)	Total
1 ft	3	3	3	4	13
2 ft	4	4	3	4	15
4 ft	4	4	4	4	16

FINDINGS AND SUGGESTED RESEARCH

On the basis of driver performance and driver subjective evaluations, the 1-ft and 2-ft on 40-ft centers striping patterns performed as well as the 4-ft pattern for centerline striping at night at seven pavement overlay projects on rural two-lane, two-way highways with 2.0 degree horizontal curvatures, level to rolling terrain, and average speeds between 50 and 62 mph. Studies were conducted in four states.

The findings should not be generalized to situations not tested. Nighttime viewing in an ambient background of near darkness will enhance the contrast of the bright reflective yellow stripes. Moreover, the horizontal curves were 2.0 degrees, with the exception of one curve that was 3.0 degrees. It is possible that the performance of the three tested striping patterns may not be equal on horizontal curves with greater curvature or at urban or suburban construction zones where the ambient lighting is different than the conditions studied. Also, the three striping patterns tested may not result in the same driver performance on mountainous highways and other types of highways with lower operating speeds.

The study did not attempt to optimize spacing or brightness to determine the most cost-effective striping pattern. Although the three striping patterns tested provided adequate delineation on rural two-lane, two-way highways, they may not necessarily represent the optimum patterns from a cost-effectiveness standpoint. It is possible that patterns with larger spacings may also provide adequate path delineation on rural two-lane, two-way highways.

The limitations of this research relative to scope of the field studies were discussed in the previous section. The discussion suggests the following:

- Future research should be directed at the effectiveness of the three striping patterns (1, 2, and 4 ft on 40-ft centers) at construction zones in situations with less brightness contrast (suburban and urban areas), horizontal curvatures greater than 2 degrees, mountainous terrain, and operating speeds lower than those tested in the current study. Ideally, studies should also be conducted when the pavement is wet and during rain.
- Research should also be directed at determining the optimum spacing of the 1-ft, 2-ft, and 4-ft stripes at construction zones on two-lane, two-way rural highways.

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DISCUSSION

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Within the scope and test conditions specified by NCHRP, the Texas Transportation Institute (TTI) chose a set of near-perfect conditions. On the pavement overlay projects on rural two-lane, two-way highways, the following conditions prevailed:

- Site locations were relatively level or rolling terrain.
- Although all sites included a horizontal curve and a tangent section, the degree of curvature at six sites was only 2.0 degrees; at the seventh site, it was 3.0 degrees.
- All sites had 12-ft lanes with 4-ft to 10-ft paved shoulders.
- Site speeds averaged between 50 and 62 miles per hour.
- The ambient background was "near darkness."
- The marking material selected for this field test was highly retroreflective pavement-marking tape.

These last two points have a major impact on skewing the research results. Drivers actually see pavement markings as a function of their contrast with the road surface (*1*). New pavement overlays such as those in the test sites are generally very black, providing excellent contrast to the yellow marking tape. The retroreflective properties of the newly applied marking tape itself provide an extremely bright optical target. On a clear, dry night (each one in this study), it is far easier for a driver to see such highly visible pavement markings than in most driving situations. The only visual distractions appear to have been limited traffic and the data collection system of "computers housed in vehicles parked off the roadway as far from the operating lanes as possible." Note that there was no discussion of potential change in driver behavior as a result of the parked vehicles.

One condition imposed by NCHRP is likely to have further skewed TTI's results: the absence of edgelines. Experience and an ample body of evidence indicate improved driver performance in the presence of edgelines (*2*). With no indication of lane boundary and limited visual information at the edge of pavement in this study, drivers' focus on the centerline was even more acute.

With a wide expanse of blacktop in a nearly dark environment and a brilliant ribbon of yellow to follow, as in this study, drivers should perform relatively consistently. It is not surprising that TTI's summary of traced vehicles indicates no differences in driver performance between the 1-ft, 2-ft, and 4-ft striping patterns with the measurement criteria of speed, lateral distance encroachment, erratic maneuver, and speed profile data. Similarly, it is reasonable to expect that given the strong visual target of highly retroreflective new pavement-marking tape contrasted against a newly surfaced road in a background of near darkness, some individuals in the subject evaluations could not differentiate between the 1-ft and 2-ft stripes, or even perhaps between 2-ft and 4-ft stripes. Each is perceived as a very bright spot in a black environment. Such spots may also appear elongated by the relatively high speeds. The TTI observation that drivers could differentiate the 39-ft spacing separating the 1-ft spots of bright light but could not discriminate between the 38-ft and 36-ft spacing separating the 2- and 4-ft bright spots supports this.

Perhaps the most surprising result of the field study was that even with these ideal conditions, *each method* of subject evaluation reported the poorest results with the 1-ft stripe and a preference for the 4-ft stripe. Yet far more important is the reported finding that *none* of the treatments were judged as extremely effective. The treatments were only rated 2 on a scale of 0 to 4. This is consistent with a prior TTI research study that reported that drivers rated 8-ft stripes with 32-ft gaps as the best striping treatment (*3*).

Given all this, it is imperative that the data in this field study are not interpreted as representative of a pavement marking pattern. They can at best be indicative of a newly placed, highly retroreflective pavement-marking tape on a resurfaced road where there is little or no visual "clutter."

The "typical" construction zone does not meet test conditions selected for TTI's study, and work zone safety is becoming a more critical issue. Analysis of U.S. traffic accidents reveals that work zone fatalities have increased from 490 in 1982 to 680 in 1985 (*4*). The Standing Committee on Highway Traffic Safety of the American Association of State Highway and Transportation Officials conducted a survey of work zone accidents on the Interstate and Primary System in 1985. Their summary reported (*4*) that

- There was an estimated $800 million economic cost associated with 400 fatal accidents, 15,000 injury accidents, and 31,000 property damage–only accidents.
- Work zone fatal accidents are concentrated in rural areas.
- Work zone accidents produce more injuries and fatalities than the national average for all accidents.
- Although more than two-thirds of all accidents occur in daylight, nighttime accidents are far more severe. Nighttime accidents account for more than half of the fatal accidents and more than their share of injury accidents.

Work zones are particularly hazardous because they present drivers with changes in the normal driving environment. Such changes place greater demand on drivers, possibly leading to confusion and accidents. Up to 90 percent of all the information used by drivers to guide and control their vehicles is obtained visually (*5*), and the pavement itself is a primary information source for drivers. In fact, if drivers are presented with conflicting information, they will generally choose to follow the pavement (*6*).

Pavement markings through work zones should provide a clear path for drivers' guidance. Such markings must be effective where needed most: at night, under adverse weather conditions, and when drivers may have other visual limitations, such as advancing age, fatigue, or alcohol consumption. The need for strong delineation patterns in work zones is gaining widespread acceptance, and our court system is providing impetus for action. In both Louisiana and New Mexico, the states were held liable for wrongful deaths where striping was not in place to warn and guide motorists through work zones (*7*). The state of North Carolina has taken the lead in providing increased information through construction work zones by using 8-in. markings, twice the standard marking line width (from a letter by J. M. Lynch to W. Cromartie, North Carolina DOT, Raleigh, August 8, 1985).

Safe driving requires both appropriate visual information and drivers who are able to receive and interpret that information. However, studies indicate that in most construction zone accidents, the driver receives neither visual stimulation nor sufficient warning (*8*). The fact that drivers often fail to meet the challenges of work zones is documented by studies indicating that the accident rate increases in work zones during construction, as compared to before construction (*8–10*). Drivers cannot effectively control their vehicles without sufficient visual information, and even this current TTI study indicates that a pavement marking pattern of short stripes with long gap ratios does not provide an effective level of communication. There is a significant body of evidence to indicate that driver performance is enhanced through stronger pavement marking patterns (*11*).

The negative consequences of this report could be far-reaching. Even though the report states "that the findings should not be generalized to situations not tested," response to this presentation at the annual Transportation Research Board meeting indicates that this is precisely what will happen. The potential detrimental impact to safety and mobility is heightened by TTI's own conclusions: With evidence only of *treatment* (with highly retroreflective marking tape under ideal conditions), TTI has claimed not only that the striping *patterns* of 1 ft, 2 ft, and 4 ft on 40-ft centers are adequate but that even larger spacings may help to optimize cost effectiveness.

As indicated in the statement of the problem, TTI uses a very narrow interpretation of the word "cost." Cost is not just money spent. More importantly, cost is measured in value received. If drivers cannot safely position their vehicles through a work zone to prevent harm to those individuals or objects in the area and to protect themselves and their passengers, a responsible jurisdiction should not open that area to traffic. Sound business considerations and concern for the public welfare dictate comprehensive resource management. Inadequate pavement marking patterns, especially in work zones where drivers need enhanced visual communication, are a prime example of false economy.

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AUTHORS' CLOSURE

We do not agree with the discussant's claim that the design of this research project led to questionable conclusions. Our response will show that the methodology and analysis were in fact sound and led to valid conclusions.

The experimental question was simply, Does the length of the temporary stripe (4, 2, or 1 ft) and associated 36-, 38-, and 39-ft gaps make a difference in how motorists drive when temporary pavement markings are used on overlay projects on two-lane, two-way rural highways? Field studies were conducted at seven pavement overlay project sites on two-lane, two-way rural highways in Arkansas, Colorado, Oklahoma, and Texas to determine whether motorists would drive better with 4-ft stripes. The data failed to indicate major differences. The in-car studies with paid drivers were included to provide a medium in which drivers could express their opinions; these were taped in real time while the subjects were driving, as well as being given during post-test evaluations by rankings and ratings. The drivers were selected to represent the driving population and, particularly, to include those over age 55.

EXPERIMENTAL DESIGN, STATISTICAL ANALYSIS, AND DESIGN INTERPRETATION

The discussant's initial allegation suggests that there were errors in experimental design, statistical analysis, and data interpretation. Certainly, we have reputations for strength in the areas of rigorous design and statistical analysis. Because of this, we went to extra lengths, which included the use of advanced data collection technology, to have a most powerful experimental design.

Most researchers conducting field studies on pavement markings have not been fortunate enough to be able to incorporate the most powerful statistical design for identifying significant differences in pavement marking treatments. This most powerful design, which we used, is a repeated measures design with control. In general, most other studies do not collect data in a way that enables a given vehicle to be traced throughout the pavement marking zone. Therefore there is generally an inflated estimate of speed variability, and the resulting test statistic for determining the pavement marking treatment effects is not sensitive to small differences. By tracing the vehicles through the study site, we were able to incorporate the covariance structure among vehicles into the design (and test statistic) to produce a statistical test that is most powerful in declaring statistical significance among very small differences. The fact remains that even by using the power statistical method in the study, we found no statistically significant differences among the 4-, 2-, and 1-ft stripes on 40-ft centers with respect to speed, lateral placement, lane encroachments, and erratic maneuvers. In summary, the statistical design and analysis that we used are beyond reproach.

TEST SITES

Our work was a valid comprehensive field study representative of a variety of two-lane, two-way highways within the scope and budget available. In an attempt to generalize the results as much as possible, the studies were conducted in four states, with test sites that represented as diverse geometrics and characteristics as could be found on overlay projects on two-lane, two-way rural highways. We specifically stated the study conditions and limitations, and we suggested future research to resolve the issue of *optimum* centerline striping patterns for other kinds of work zone applications.

CONTRAST BETWEEN PAVEMENT AND MARKINGS

The discussant made an issue of the "extremely bright optical targets" (the stripes on black asphalt), claiming that they were easier to see than markings in most driving situations. Overlays on two-lane, two-way rural highways are, by definition, fresh, dark backgrounds. We would have been remiss to have used sun-baked irregular or worn surfaces because they do *not* represent the real-world situation with *fresh overlay*. The fact that the temporary tape markings provide excellent contrast and visibility is a point in their favor.

In the real world, temporary centerline stripes on overlay projects are in place for a period of up to 2 weeks until the overlay work is completed. A striping crew then applies permanent striping. Also, because the temporary yellow reflective tape markings used as centerlines for these projects are in place for a maximum of approximately 2 weeks, the stripes are indeed brilliant. The temporary centerline markings at pavement overlay projects on two-lane, two-way rural highways generally *are* superior in terms of cleanliness and reflectivity to the markings on upstream and downstream highway sections because they are newer. This is precisely one of the reasons that driver performance was the same when the 2- and 1-ft striping patterns were used, compared to the 4-ft striping pattern on the overlay sections of the rural highways. If the markings were in place for significantly longer durations, as is the case for the other work zone applications, then it is possible that the findings would be different.

EDGELINES

The discussant criticizes NCHRP for requiring no edgelines in the field study. Obviously, if long, continuous edgelines had marked the pavement course, the drivers might well have used these markings for visual guidance rather than the centerline stripes that were the subject of the research that we reported, and thus it would not have been possible to evaluate the specific effects of the three candidate striping patterns.

A second point is that edgelines would not be representative of many overlay projects on two-lane, two-way rural highways, where often the only cue the driver has is the centerline stripes. So, had we done the research with edgelines, the findings would be inapplicable to the more common overlay situation immediately after the pavement is laid.

Great care was taken to ensure that the administrative personnel and measuring devices would not be seen before the test site and thereby would not bias the drivers. This point was implied by the statement that these personnel were in a vehicle far from operating lanes.

DATA INTERPRETATION AND SAFETY IMPLICATIONS

It is in the area of data interpretation and safety implications that the discussant's flawed analysis is most evident.

Application of Results

We clearly emphasize and state that *the results only apply to pavement overlay projects on two-lane, two-way rural projects* and certainly do not advocate translating these findings to other highway work zone situations. Early in her comments the discussant falsely gives the readers the impression that we recommend adoption of the 2- or 1-ft striping pattern in *all* work zones. She admits near the end of her discussion that our paper states "that the findings should not be generalized to situations not tested."

Drivers' Evaluations

The results of the driver evaluations showed that on the average, although the 1-ft striping pattern was rated and ranked slightly lower *numerically*, its ratings and rankings were not statistically significantly different from those of the 2- and 4-ft striping patterns. Drivers at seven different sites *could have* rated the 4-ft striping pattern consistently the highest, but they did not. They were aware that the 4-ft stripe was longer than the 1-ft stripe but had some difficulty discriminating between the 2-ft and 4-ft stripes. They had trouble discriminating brightness as well. All patterns were judged "satisfactory" to "excellent." The discussant interpreted these ratings as less than desirable and suggested that the drivers were trying to indicate they wanted still longer stripes. It is true that in earlier research by TTI in *controlled* proving ground studies (not field studies), drivers *rated* the 8-ft stripe as their first choice. However, driver *performance* with the 8-ft stripe was not better than with several shorter stripes. Furthermore, the 8-ft stripe was not one of the treatments investigated in the field studies, as noted above. Only if the drivers had *consistently* rated the 4-ft stripe as the best would a still longer stripe have been indicated in the present application. Further, seldom does a sample of drivers rate *anything* as "excellent." Driver variability enters the picture in rating, as does a tendency to include other environmental elements into the ratings (weather, previous highways driven, etc.).

Cost

The criticism that the study narrowly interprets the word "cost" when lives are at stake was an attempt to discredit the research as leading to a loss of lives. Only if someone grossly misinterpreted the objective of the study would this issue apply.

Presentation of Results

The discussant implies that because the research that we reported did not evaluate all possible combinations, it should not have been reported at the Annual Meeting of the Transportation Research Board. If the research community waited until all variables relative to a subject are evaluated before presentation and publication, knowledge would not be advanced very much.

ACCEPTANCE OF RESULTS

The discussant's review contradicts previous reviews, which are based on the knowledge and integrity of reputable researchers and highway officials. The study results were reviewed and accepted by these knowledgeable professionals: (a) an NCHRP Panel of Experts, (b) the TRB Committee on Traffic Control Devices, and (c) experts serving on the Construction and Maintenance (C&M) Technical Committee of the National Committee on Uniform Traffic Control Devices. On the basis of the results of the research and recommendations from a task force headed by one of us, the C&M Technical Committee unanimously approved a recommendation that a full complement of markings be used in all work zones, with the exceptions noted in the following recommendations:

> The National Committee requests that the FHWA reopen and revise the Rule-Making on Work Zone Pavement Markings which appropriately reflects the following recommendations:
>
> 1. For paving operations, short-term markings may be installed to a lesser dimensional standard than that specified for permanent markings.
>
>   Short-term pavement markings for paving operations are defined as temporary pavement marking lines placed on centerlines and lane lines, following the paving operations, which will be in place up to two weeks, at which time it is expected that permanent markings will be in place.
>
>   To the extent practicable, it is intended that temporary work zone markings and/or appropriate channelizing and delineating devices will approximate the guidance normally supplied by permanent markings.
>
> 2. Short-term pavement markings for lane lines and dashed centerlines may be less than four feet in length.
> 3. The National Committee recommends that the FHWA recognize that normal pavement markings for chip and sand seals on low-volume roadways, and roadways undergoing milling operations, may not be practical and therefore other delineation treatments shall be used.
> 4. When the installation of short-term pavement markings is impractical during pavement operations, channelizing or delineating devices with appropriate warning signs shall be used.
> 5. The National Committee endorses additional research to be conducted by FHWA to improve engineering practices to insure that safe and cost-effective temporary markings and other delineation treatments are adopted for use in highway work zones, particularly for short-term paving operations.

Publication of this paper sponsored by Committee on Traffic Control Devices.

Evaluation of Wide Edgelines on Two-Lane Rural Roads

BENJAMIN H. COTTRELL, JR

The effect of 8-in.-wide edgelines on the incidence of run-off-the-road (ROR) and related accidents was evaluated. The treatment locations consisted of three two-lane rural road sections totaling 60.7 miles. A before-and-after design with a comparison group and a check for comparability was used to analyze data. Five years of accident data, covering the 3 years before wide edgeline installation and the 2 years after installation, were used. It was concluded that there is no evidence to indicate that wide edgelines significantly affected the incidence of ROR and related accidents for any individual treatment location or for the locations combined. The related accidents include ROR accidents that involved driving under the influence of alcohol or drugs, ROR accidents on curves, ROR accidents during darkness, and opposite-direction accidents.

There are a high number of run-off-the-road (ROR), drunken driving, and night accidents in rural areas. In 1985, there were 19,385 ROR accidents in rural areas in Virginia (*1*). Of this total, 268 (1.4 percent) were fatal accidents, 9,434 (48.6 percent) were injury accidents, and 9,683 (50.0 percent) were property damage accidents. ROR accidents accounted for 29.1 percent of all rural accidents, 40.7 percent of the fatal accidents (the largest percentage for any type of accident), and 35.6 percent of the injury accidents in rural areas. Individuals driving under the influence of alcohol or drugs (DUI) were involved in 9,878 (14.8 percent) of all rural accidents. Accidents involving DUI accounted for 34.4 percent of fatal accidents, 20.1 percent of injury accidents, and 11.0 percent of property damage accidents in rural areas. There were 22,570 accidents during darkness, which constituted 33.9 percent of all accidents in rural areas.

Edgelines are used to delineate the right edge of the roadway to provide guidance to motorists. The standard edgeline width is 4 in. The edgeline is one element in a pavement marking system that provides warning and guidance information to the driver without diverting attention from the roadway (*2*). Reflectorized pavement markings are the most common form of delineation at night, when reduced visibility creates a greater need for guidance information. Edgelines 8-in. wide may reduce the probability of a driver running off the road and increase the probability that a driver will position his vehicle close to the centerline. However, since it is possible that wide edgelines will influence the lateral position of the vehicle in this way, the probability of centerline encroachment may increase as well.

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OBJECTIVES AND SCOPE

The objectives of this research were to evaluate the effect of wide edgelines on the incidence of ROR, DUI, and other related types of accidents, as well as on the lateral placement and speed of vehicles. The scope was limited to two-lane rural roads. Primary routes were selected because accident data are more detailed and more readily available for these than for secondary routes.

The subject of this paper is the incidence of accidents. The report that documented the evaluation of lateral placement and speed may be summarized as follows (*3*):

- There were no statistically significant differences between the 4- and 8-in.-wide edgelines in lateral placement, lateral placement variance, encroachments by automobiles and trucks, mean speed, and speed variance.
- The mean lateral placement was significantly lower for the 8-in.-wide edgeline. The difference was small, however, and of no practical significance.
- Lateral placement and speed were not practically affected by a change from a 4-in. to an 8-in.-wide edgeline.

STUDY DESIGN

Experimental Plan

After testing several procedures for evaluating highway safety improvements, a before-after design with a comparison group and a check for comparability was selected. A detailed description of this procedure is given by Griffin (*4*). The procedure he described is condensed and discussed later in this section. The before-after design with a comparison group and a check for comparability provides some relief from two fallacies. By using a comparison group, the influence of extraneous factors is at least partially controlled; therefore there is some relief from the post hoc ergo propter hoc (after the fact, therefore because of the fact) fallacy. By using multiple before and after readings (e.g., each year represents a reading), some relief is obtained from the regression to the mean fallacy (*4*). Consequently, this evaluation design is more rigorous and more valid than a simple before-after design and a before-after design with a comparison group.

The comparability is determined by the difference in the rate of change in the frequency of accidents at the treatment and comparison locations during the before and after periods (Figure 1). The rates of change in accident frequencies are expressed as natural logarithms. When the rates of change in accident frequencies of the treatment and comparison groups

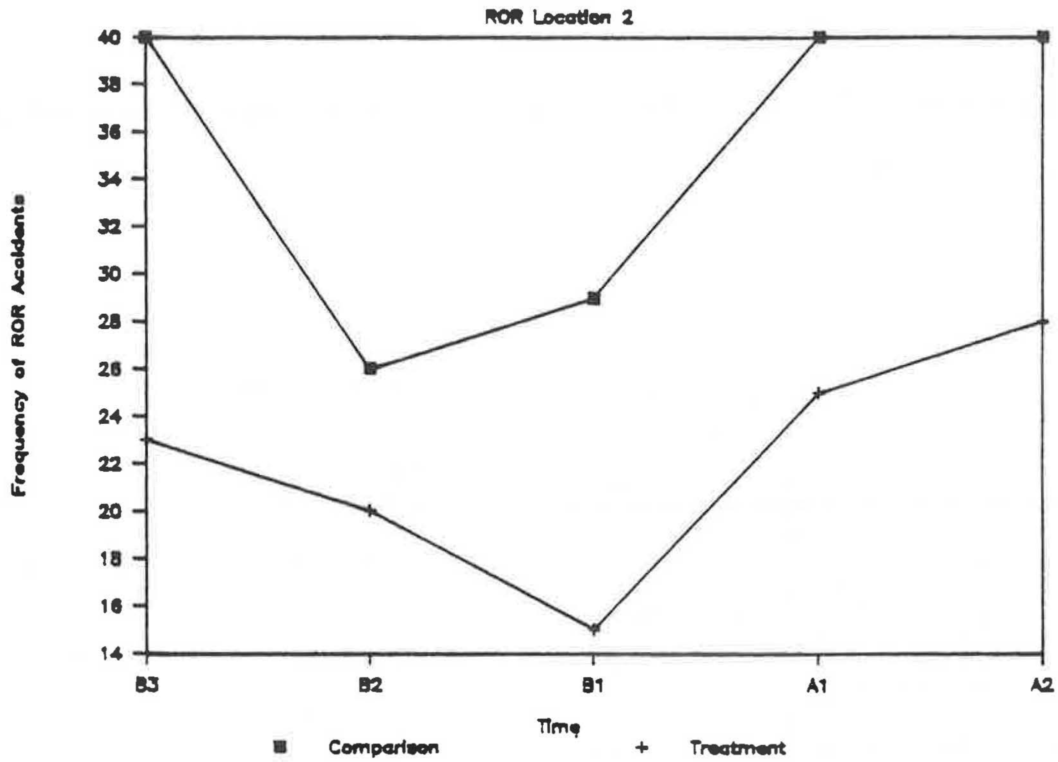

FIGURE 1 Frequency graph.

are equivalent, the slopes of the natural log (ln) frequency over time are the same, and therefore they are parallel (Figure 2). The procedure involves two steps:

Step 1: Check for Comparability. If the slopes on the treatment and comparison functions of ln frequency versus time deviate by more than chance expectation during the before and after periods, then the comparison group is not comparable to the treatment group, and further analysis is not appropriate. If the slopes do not deviate, there is no reason to doubt the comparability of the comparison group (*4*).

Step 2: Effect of the Treatment. In the second step, the treatment and comparison groups are collapsed across the before and after periods. If the slopes on the treatment and comparison functions do not deviate by more than chance expectation from before to after, then there is no evidence that the treatment imposed affected the incidence of accidents. If the slopes do deviate, then the treatment is said to have produced an effect. If the slope on the treatment is more negative (or less positive) than the slope in the comparison function, the treatment is beneficial. If the slope on the treatment function is less negative (or more positive) than the slope on the comparison function, the treatment is harmful (*4*).

Statistical Equations

The calculations used to answer the questions are based on the likelihood ratio chi-square (G^2) test. A $2 \times n$ contingency table, where n = total number of years of data, is developed. The overall goodness-of-fit test, G^2 total, is equal to the sum of G^2 Comparability and G^2 Treatment. In other words, the contingency table is partitioned into two parts:

- G^2 Comparability for the goodness of fit within the before and after periods for homogeneity of the treatment and comparison group, and
- G^2 Treatment for the goodness of fit from the before and after periods for the association of the treatment and comparison groups (*4, 5*).

The critical G^2 values that are compared with G^2 Comparability and G^2 Treatment are based on a 0.05 level of significance and are 7.81 and 3.84, respectively.

The formula for the likelihood ratio chi-square (G^2) test is (*4*):

$$G^2 = -2 \sum_i \sum_j X_{ij} \ln \frac{\hat{m}_{ij}}{X_{ij}} \tag{1}$$

where X_{ij} = observed accident frequency in cell ij row (i) and column (j);

$$\hat{m}_{ij} = \frac{X_{i+} X_{+j}}{X_{++}}$$

for G^2 Before when $i = 1, 2, 3$, and $j = 1, 2$; for G^2 After and G^2 Treatment when $i = 1, 2$ and $j = 1, 2$

$$X_{i+} = \sum_i X_{ij} \quad \text{(sum of row } i\text{)}$$

$$X_{+i} = \sum_j X_{ij} \quad \text{(sum of column } j\text{)}$$

$$X_{++} = \sum_i \sum_j X_{ij} \quad \text{(sum of the partitioned contingency table being tested)}$$

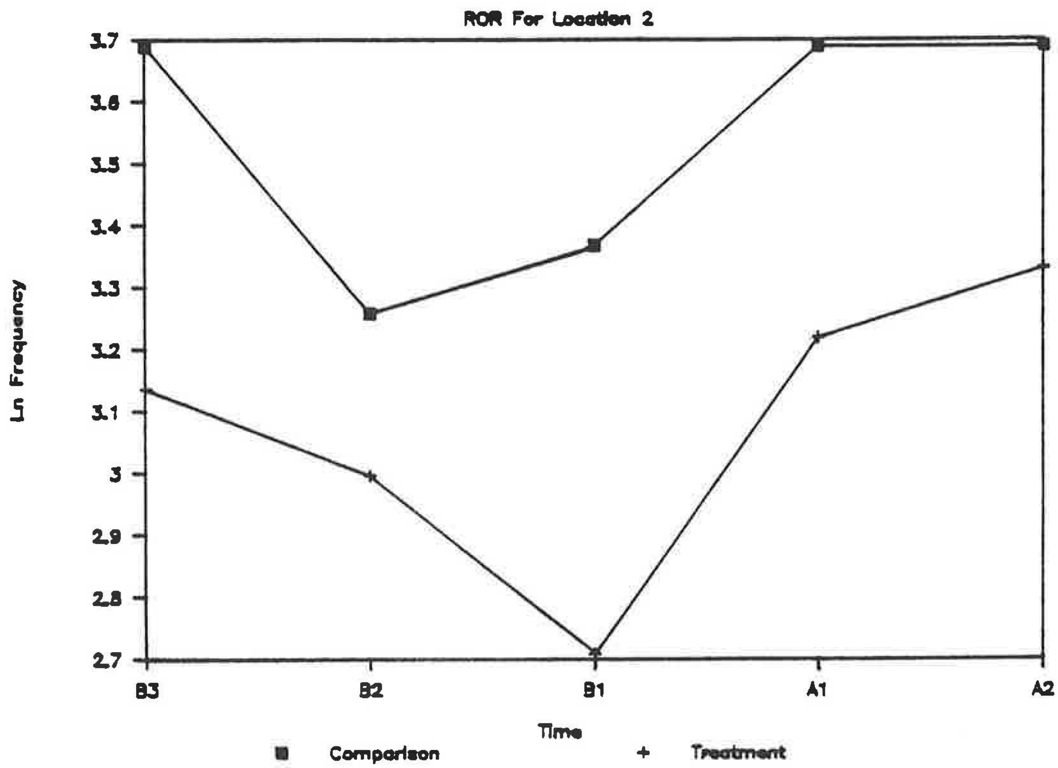

FIGURE 2 Ln frequency graph.

An alternative for calculating G^2 Treatment uses the same 2×2 table as in step 2, in which the treatment and comparison groups are collapsed across the before and after periods. The following equations are used:

	Comparison	*Treatment*
B3–B1	X_{11}	X_{12}
A1–A2	X_{21}	X_{22}

$$\tau = \frac{X_{11}\,X_{22}}{X_{12}\,X_{21}} \tag{2}$$

where τ is the cross-products ratio: $(\tau - 1) \times 100$ equals the apparent percentage change in accidents attributable to the treatment. Equation 3 is used to determine if the apparent treatment effect is significant:

$$Z = \frac{\ln \tau}{(1/X_{11} + 1/X_{12} + 1/X_{21} + 1/X_{22})^{1/2}} \tag{3}$$

For $\alpha = 0.05$ and a two-tailed test, the confidence interval lies between −1.96 and 1.96.

The advantage to using this alternative is that the apparent change in accidents attributable to the treatment is obtained. Both methods of calculating G^2 Treatment were used in the analysis.

A limitation should be noted in using this study design. To avoid dividing by 0, which results in an undefined G^2 value, each cell in the 2×5 contingency table must be greater than 0. Note that frequencies are used in contingency tables instead of rates. Moreover, exposure was a factor in the selection of the comparison groups.

Combining Treatment Sections

So that the effects of all three treatment sections can be examined together, the logarithms of the odds ratios are combined by using a technique commonly called Gart's procedure (5–7). Gart's procedure combines 2×2 contingency tables with the natural logarithm of the odds (or the maximum likelihood) ratio as the measure of association. The log odds ratio for each location is weighted on the basis of the accident frequency. Figure 3 displays the worksheet used for the procedure, along with the equations used. The chi-square statistic for testing the homogeneity of the odds ratio, X^2 homogeneity with 2 degrees of freedom, indicates the existence of insignificant differences among the three odd ratios. An acceptable X^2 homogeneity indicates no significant difference. The chi-square statistic for testing the significance of the mean log odds ratio, X^2 association with 1 degree of freedom, indicates the existence of insignificant differences between the comparison and treatment groups. The chi-square total is equal to the sum of X^2 homogeneity and X^2 association.

There are benefits to combining the three locations. By increasing the amount of data available for testing, the statistical power is increased. In other words, combining the locations improves the opportunity to identify a treatment effect if one is present.

Treatment Locations

Three sections of roadway—17.2-mi, 19.1-mi, and 24.4-mi long—served as the treatment locations. Wide (8-in.) edgelines were painted at these sites during spring and summer 1984. The wide edgelines were repainted approximately one year later. The actual edgeline width varied from 7.0 to 10.0 in. The study

Accident Type:

Study Number	Comparison Before a	Comparison After b	Treatment Before c	Treatment After d	Estimated Weights W_i	Relative Risk L_i	Product $W_i L_i$	$W_i (L_i)^2$
1								
2								
3								
Total					ΣW_i		$\Sigma W_i L_i$	$\Sigma W_i (L_i)^2$

$W_i = \left(\frac{1}{a} + \frac{1}{b} + \frac{1}{c} + \frac{1}{d}\right)^{-1}$ - estimated weight

$L_i = \ln \frac{a}{b} \cdot \frac{d}{c}$ - natural logarithm of the odds ratio

$X^2_{total} = \Sigma W_i (L_i)^2$

$X^2_{homogeneity} = \Sigma W_i (L_i)^2 - \frac{(\Sigma W_i L_i)^2}{\Sigma W_i}$

$X^2_{association} = X^2_{total} - X^2_{homogeneity}$

Critical X^2

df	α=0.05
2	5.99
1	3.84

FIGURE 3 Worksheet for combining the three locations.

sections were in four districts, so four different paint crews were used. On the basis of 12 sample site studies in the interim report for lateral placement and speed changes, the average edgeline width for each treatment location was:

- 17.2-mi section: 7.6 in.
- 19.1-mi section: 7.4 in.
- 24.4-mi section: 9.3 in.

Comparison Locations

Several measures were used as a guide in selecting locations for comparison with the three treatment locations. The primary objective was to identify locations that were similar to the treatment locations for the following characteristics: two-lane rural roads, overall roadway geometrics, average daily traffic, total accident frequencies, run-off-the-road accident frequencies, and alcohol- and drug-related accident frequencies. Also, no changes that would influence the frequency of accidents were planned for the road sections.

The key to the appropriateness of a comparison location is the check for comparability. If the results of the check were that the treatment location was not comparable with the comparison location, the alternative comparison location would be the treatment location with all other accident types. The use of all other accident types on the treatment location as a comparison location is generally acceptable. The alternative comparison locations eliminate extraneous factors such as exposure, roadway geometrics, alignment, and weather because the alternative comparison and treatment road sections are the same. Information on the treatment and comparison locations is presented in Table 1.

Measures of Effectiveness

This evaluation focuses on the effectiveness of the wide edgelines in reducing accidents, especially ROR, DUI, and other related types of accidents. ROR accidents were the primary type of accident evaluated. Also, ROR accidents involving four other factors in addition to DUI were selected for a detailed analysis. ROR accidents at curves were considered because horizontal alignment is a factor in ROR accidents. Because edgelines are important in delineating the roadway during darkness, ROR accidents during darkness were selected as a measure. ROR accidents in inclement weather were selected as a measure because inclement weather is an extraneous factor that

TABLE 1 DESCRIPTION OF THE TREATMENT AND COMPARISON LOCATIONS

Location	1985 ADT (vehicles)	1985 Daily Distance Traveled (vehicle-mi)	Roadway Width (ft)	Roadway Description
1 T: Route 20, Buckingham	2,275	43,340	20.0	Straight road
C: Route 40, Pittsylvania and Halifax	2,180	54,085	20.0	Straight road
2 T: Route 20, Albemarle	3,685	66,090	20.5	Winding road
C: Route 8, Floyd and Montgomery	3,670	71,385	22.0	Winding road
3 T: Route 501, Bedford and Rockbridge	2,580	48,710	19.4	Winding and straight roadway
C: Route 20, Albemarle and Orange	2,700	40,095	20.1	Winding and straight roadway

NOTE: T: = treatment location; C: = comparison location. The alternative comparison location was the treatment location with all other accidents.

may contribute to ROR accidents. Because of concern about drivers encroaching on the centerline because of wide edgelines, opposite-direction accidents were evaluated. In all, six measures of effectiveness were used.

Data

Accident data were obtained from the Virginia Department of Transportation's computerized traffic accident-reporting system. Three years of before-data with 4-in.-wide edgelines and 2 years of after-data with 8-in.-wide edgelines were used. The accident data were based on accident reports completed by the state or local police officer who responded to the accident. The presence of a curve, darkness, or inclement weather was determined by the police officer. Similarly, DUI was noted as a contributing factor on the basis of tests administered by the police officer or when DUI was suspected because of the situation, evidence, or testimony of witnesses.

ACCIDENT DATA ANALYSIS

The analysis results for each measure of performance will be described for each treatment section and for all sections combined. Although two levels of significance, 0.05 and 0.10, were examined, only 0.05 is displayed in the analysis tables. Unless otherwise stated, the conclusions on the effect of the treatment are the same for both levels.

Run-off-the-Road Accidents

The analysis data for ROR accidents are presented in Table 2. In the check for comparability, treatment Location 1 was not comparable to its original comparison location. Therefore the alternative comparison location of all non-ROR accidents on the treatment location was used and found to be comparable for all treatment locations. On the basis of $\alpha = 0.05$, there was no evidence that the wide edgelines significantly affected the incidence of ROR accidents for any of the three treatment locations individually or combined. However, for a level of significance of 0.10, Location 1 shows a significant decrease in ROR accidents. The apparent percentage reduction is 55 percent. The low accident frequency in the A1 period probably accounts for the significant decrease in accidents because the A2 period accident frequency is the highest of the 5-year period.

TABLE 2 ROR ACCIDENTS

(a) Analysis for Each Location[a]

	Accident Frequency					
	Location 1		Location 2		Location 3	
Year	C	T	C	T	C	T
B1	1	4	40	23	20	21
B2	10	10	26	20	18	16
B3	5	10	29	15	23	16
A1	13	6	40	25	14	12
A2	12	11	40	28	25	20

	G^2 Values for Locations				Critical X^2
Source	1	2	3	*df*	$\alpha = 0.05$
Comparability					
Before	2.02	0.93	0.84	2	5.99
After	1.15	0.10	0.02	1	3.84
Treatment	3.14	0.11	0.04	1	3.84
Total	6.31	1.14	0.90	4	
Apparent Change (%)	−55	9	−6		

(b) Combining Locations With Gart's Procedure[b]

Source	X^2	*df*	Critical X^2 $\alpha = 0.05$
Homogeneity	2.91	2	5.99
Association	0.31	1	3.84
Total	3.22	3	

NOTE: C = Comparison, T = Treatment. The comparison group is all other accidents at the treatment location. Apparent change values are given only if locations are comparable.

[a] Conclusion for each location: Comparability—Acceptable for each; Treatment—No significant effect for each.

[b] Conclusion for combined locations: Homogeneity—Acceptable; Association—No significant effect.

ROR Accidents Involving DUI

The analysis data are presented in Table 3. Because there were 0 values in the original comparison location for Treatment Location 1, the alternative comparison location of all other accidents on the treatment locations was used and found to be comparable for all treatment locations at $\alpha = 0.05$. There was no evidence that the wide edgelines significantly affected the incidence of accidents involving both ROR and DUI on all treatment locations. On the basis of X^2 homogeneity and X^2 association, the combined locations are acceptable and there is no indication of a significant effect.

ROR Accidents on Curves

All three treatment locations were comparable with the alternative comparison locations. There was no evidence that the wide edgelines significantly affected the incidence of ROR accidents on curves for the treatment locations (Table 4). When the locations are combined, the X^2 homogeneity was acceptable, and X^2 association indicated no significant effect.

ROR Accidents During Darkness

The analysis data are presented in Table 5. All three pairs of treatment and original comparison locations were comparable. Furthermore, for all three locations, there was no evidence to suggest that the wide edgelines significantly affected the incidence of ROR accidents during darkness. The apparent percentage increase for Location 2 of 122 percent is high but ineffective statistically.

X^2 homogeneity is acceptable, and X^2 association indicates that there is no significant effect for the combined locations. When the alternative comparison groups are used, the treatment and alternative comparison groups for Locations 1 and 2 are comparable. There are no significant effects for the two sites individually nor combined.

TABLE 3 ROR ACCIDENTS INVOLVING DUI

(a) Analysis for Each Location[a]

	Accident Frequency					
	Location 1		Location 2		Location 3	
Year	C	T	C	T	C	T
B1	2	3	60	3	34	7
B2	18	2	36	10	30	4
B3	11	4	43	1	38	1
A1	16	3	60	5	22	4
A2	19	4	64	4	42	3

	G^2 Values for Locations				Critical X^2
Source	1	2	3	*df*	$\alpha = 0.05$
Comparability					
Before	5.52	11.80	5.31	2	5.99
After	0.02	0.17	1.35	1	3.84
Treatment	0.44	0.55	0.02	1	3.84
Total	5.99	12.52	6.69	4	
Apparent Change (%)	−31	–	−7		

(b) Combining Locations With Gart's Procedure[b]

Source	X^2	*df*	Critical X^2 $\alpha = 0.05$
Homogeneity	0.25	2	5.99
Association	0.20	1	3.84
Total	0.45	3	

NOTE: C = Comparison, T = Treatment. The comparison group is all other accidents at the treatment location. Apparent change values are given only if locations are comparable.

[a]Conclusion for each location: Comparability—Acceptable for each; Treatment—No significant effect for each.

[b]Conclusion for combined locations: Homogeneity—Acceptable; Association—No significant effect.

TABLE 4 ROR ACCIDENTS ON CURVES

(a) Analysis for Each Location[a]

	Accident Frequency					
	Location 1		Location 2		Location 3	
Year	C	T	C	T	C	T
B1	4	1	46	17	25	16
B2	17	3	33	13	23	11
B3	11	4	36	8	29	10
A1	15	4	50	15	19	7
A2	19	4	49	19	29	16

	G^2 Values for Locations				Critical X^2
Source	1	2	3	*df*	$\alpha = 0.05$
Comparability					
Before	0.72	1.55	1.64	2	5.99
After	0.09	0.41	0.57	1	3.84
Treatment	0.01	0.02	0.00	1	3.84
Total	0.82	1.99	2.21	4	
Apparent Change (%)	−6	4	0		

(b) Combining Locations With Gart's Procedure[b]

Source	X^2	*df*	Critical X^2 $\alpha = 0.05$
Homogeneity	0.02	2	5.99
Association	0.01	1	3.84
Total	0.03	3	

NOTE: C = Comparison, T = Treatment. The comparison group is all other accidents at the treatment location. Apparent change values are given only if locations are comparable.

[a]Conclusion for each location: Comparability—Acceptable for each; Treatment—No significant effect for each.

[b]Conclusion for combined locations: Homogeneity—Acceptable; Association—No significant effect.

ROR and Weather

Because there were 0 values for each treatment and comparison location in the contingency table, it was not possible to analyze ROR and weather. The low frequency of ROR accidents in inclement weather demonstrates that weather is not a substantial influence in ROR accidents. Consequently, it was concluded that there was an insufficient number of ROR accidents in inclement weather to determine a statistical effect.

Opposite Direction

The analysis data are presented in Table 6. Because Treatment Location 1 had three 0 values in the contingency table, it was not possible to analyze this location. Because the original comparison location for Treatment Location 2 had a 0 in the table, the alternate comparison location of all nonopposite-direction accidents was used. Treatment Locations 2 and 3 were comparable with their alternative comparison locations. There was no evidence that wide edgelines affected the incidence of opposite-direction accidents. Similarly, the X^2 homogeneity and X^2 association were acceptable and showed no evidence of a significant effect.

Results from a Before-After Design With a Comparison Group

As noted previously, the before-after design with a comparison group and check for comparability has more statistical power and is more statistically valid then the traditional before-after design with a comparison group. These differences can be illustrated by reviewing the results of this study against the more familiar before-after design with a comparison group. The evaluation procedure used accident rates in accidents per million vehicle-mi and the Poisson distribution for testing (*8*). The B2 and B3 years were the before period. The results, presented in Table 7, are mixed, inconsistent, and inconclusive. Again, two advantages of the before-after design with a comparison group and check for comparability are that the comparability of the comparison group is tested and the test locations can be combined and evaluated.

Summary

On the basis of the analysis of the six measures of effectiveness, there is no evidence to indicate that wide edgelines significantly affected the incidence of ROR accidents and related accident types. This is also true when the level of significance was increased to 0.10 for a lower level of confidence.

TABLE 5 ROR ACCIDENTS DURING DARKNESS

(a) Analysis for Each Location[a]

	Accident Frequency					
	Location 1		Location 2		Location 3	
Year	C	T	C	T	C	T
B1	2	2	6	8	5	15
B2	1	5	4	12	9	8
B3	1	5	5	8	9	5
A1	3	5	4	13	13	8
A2	2	7	3	16	10	13

Source	G^2 Values for Locations 1	2	3	*df*	Critical X^2 $\alpha = 0.05$
Comparability					
Before	1.64	1.18	5.96	2	5.99
After	0.48	0.34	1.50	1	3.84
Treatment	0.08	2.37	0.49	1	3.84
Total	2.19	3.90	7.95	4	
Apparent Change (%)	−20	122	−25		

(b) Combining Locations With Gart's Procedure[b]

Source	X^2	*df*	Critical X^2 $\alpha = 0.05$
Homogeneity	2.28	2	5.99
Association	0.58	1	3.84
Total	2.86	3	

NOTE: C = Comparison, T = Treatment. The comparison group is all other accidents at the treatment location. Apparent change values are given only if locations are comparable.

[a]Conclusion for each location: Comparability—Acceptable for each; Treatment—No significant effect for each.

[b]Conclusion for combined locations: Homogeneity—Acceptable; Association—No significant effect.

TABLE 6 OPPOSITE DIRECTION ACCIDENTS

(a) Analysis for Each Location[a]

	Accident Frequency					
	Location 1		Location 2		Location 3	
Year	C	T	C	T	C	T
B1	5	0	56	7	36	5
B2	20	0	41	5	29	5
B3	15	0	37	7	34	5
A1	16	3	57	8	20	6
A2	22	1	65	3	41	4

Source	G^2 Values for Locations 1	2	3	*df*	Critical X^2 $\alpha = 0.05$
Comparability					
Before	–	0.67	0.11	2	5.99
After	–	2.82	2.64	1	3.84
Treatment	–	1.32	0.03	1	3.84
Total	–	4.80	2.77	4	
Apparent Change (%)	–	−36	8		

(b) Combining Locations With Gart's Procedure[b]

Source	X^2	*df*	Critical X^2 $\alpha = 0.05$
Homogeneity	0.79	2	5.99
Association	0.51	1	3.84
Total	1.30	3	

NOTE: C = Comparison, T = Treatment. The comparison group is all other accidents at the treatment location. Apparent change values are given only if locations are comparable.

[a]Conclusion for each location: Comparability—Acceptable for each; Treatment—No significant effect for each.

[b]Conclusion for combined locations: Homogeneity—Acceptable; Association—No significant effect.

TABLE 7 STUDY RESULTS FROM THE BEFORE-AFTER DESIGN WITH A COMPARISON GROUP AND POISSON TEST

Accident Type	Location 1, Route 20, Buckingham County	Location 2, Route 20, Albemarle County	Location 3, Route 501, Bedford and Rockbridge Counties
ROR	48.9% decrease at 90% CL	84.4% increase at 99% CL	23.3% decrease at 90% CL
ROR and DUI	Not performed because of division by 0	Not significant	66.7% increase at 95% CL
ROR and curve	66.7% increase at 95% CL	21.5% decrease at 90% CL	Not significant
ROR and darkness	52.7% decrease at 99% CL	90.8% increase at 99% CL	51.1% increase at 95% CL
Opposite direction	Not performed because of division by 0	53.0% increase at 99% CL	99.6% increase at 99% CL

NOTE: CL = confidence level.

Moreover, these findings concur with the results of an evaluation of wide edgelines in New Mexico where, by using a before-after design with a comparison group, 100 mi of wide (8-in.) edgelines were compared with 353 mi of a comparison group with the standard 4-inch edgelines (*9*).

CONCLUSION

The before-and-after design with a comparison group and a check for comparability was used, along with Gart's procedure (for combining the accident data from the three study locations and the respective comparison groups), to analyze the data. There was no evidence that wide edgelines significantly affected the ROR accident frequency or the frequency of related accident types for each study location nor for the combined locations at 0.05 level of confidence. The accident types included ROR accidents, ROR involving DUI, ROR on curves, ROR during darkness, ROR and weather, and opposite direction. The findings are based on 5 years of accident data, 3 years before wide edgeline installation and 2 years after installation.

ACKNOWLEDGMENTS

The assistance of Lindsay I. Griffin III in the application of the experimental plan is acknowledged and very much appreciated. Griffin's work described the before-after design with a comparison group and a check for comparability. The research reported here was financed with Highway Planning and Research funds administered by the Federal Highway Administration.

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DISCUSSION

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Because wide edgelines have been found to be cost-effective alternatives to standard edgelines if they further reduce crashes by no more than about 1 percent (*1*), considerable effort is under way in the United States to assess their possible safety effect. The Virginia study is one of the first of these current initiatives to be reported. The purpose of this discussion is to offer additional perspective on this study—and others like it that seek to document small safety benefits by using crash data—and to urge caution in the interpretation of its findings.

In analyzing crash data, the highway safety researcher is first interested in learning the effect of the treatment on crash frequency or crash rate. The two most significant types of crashes that are likely to be influenced by wide edgelines are run-off-road (ROR) and opposite-direction (OD) crashes. The hypothesis is that ROR crashes might be reduced as a result of the enhanced conspicuity of the wider edgelines but that OD crashes might be increased as drivers steer toward more central positions on the roadway. In Table 8, the frequencies of these combined crash types as observed in Virginia are tabulated, and by using a procedure common to before-after studies with control sections, the likely reduction in crashes in the 2-year after period are estimated. When the three treatment sections were considered together, there were about 17 fewer ROR and OD crashes in the 2-year period following application of 8-in. edgelines than would otherwise be expected. This translates to a 7 percent reduction on the basis of the total number of crashes observed in the after period and a 13.6 percent reduction on the basis of the observed number of ROR and OD crashes. Although these benefits appear small in magnitude, they greatly exceed the levels necessary for cost-effective application (*1*) and hence may be of considerable practical significance.

Unfortunately for safety researchers, crash frequency is a highly variable quantity, and simple analyses such as those just mentioned must be supplemented by more sophisticated techniques in an attempt to assure that the observed effect is not simply due to chance occurrence. These extended analyses attempt to minimize the risk of erroneous conclusion. One error that the safety researcher wants to avoid is the conclusion that a treatment is effective in reducing crashes when, in fact, it is not. This is termed an error of the first kind (or Type I error), and the probability of committing this error is called the level of significance. Statistical testing procedures can be designed to keep the risk of committing a Type I error to a small level. The Virginia study tested two levels of significance, 0.05 and 0.10, which are indicative of the range commonly employed by highway researchers. For neither of these two levels of significance was the crash effect of the wide edgelines in Virginia found to be statistically significant. While this certainly might mean that the differential effect of wide edgelines was nil, it also might mean that the sample size was such that large variability in the crash data was allowed to mask a small treatment effect.

In any event, the safety researcher also wants to minimize risk of committing another kind of error, a Type II error or error of the second kind. A Type II error results when a treatment is concluded to be ineffective when, in fact, it is effective. A large risk of committing a Type II error is expected when the treated

TABLE 8 SUMMARY OF EXPECTED EFFECTS OF 8-IN. EDGELINES ON RUN-OFF-ROAD AND OPPOSITE-DIRECTION CRASHES

	Crash Frequency					
	Observed				Expected	
	Before Period		After Period		After Period	
Location	All Other[a]	ROR + OD	All Other[a]	ROR + OD	ROR + OD	Reduction
1	16	24	21	21	31.50	10.50
2	76	77	69	64	69.91	5.91
3	46	68	29	42	42.87	0.87

NOTE: Total crashes observed in after period: 246
Total ROR + OD crashes observed in after period: 127
Total reduction in ROR + OD crashes expected: 17.28
Reduction as a percentage of ROR + OD crashes: 13.61
Reduction as a percentage of crashes of all types: 7.02

[a] At each location in the Virginia study, crash frequencies used for control were the frequencies of all other crash types at that same location. In this table, all crash types except ROR and OD are included in this category.

mileage and safety effect are small and when the variability in crash frequency is large. Unfortunately, although it appears that such conditions characterized the Virginia experiment, no assessment was made of the Type II risk, and apparently no attempt was made to maintain the risk at an acceptable level by selecting a sample of adequate size.

In an attempt to illustrate the sample size problem, a hypothetical analysis was undertaken of a before-after crash study with matched or paired treatment and control sites. This is illustrative of the kind of study conducted in Virginia, except that testing for comparability of the matched sites was unnecessary. In the absence of real data, the following assumptions were made:

- Extent of crash data: Two years before and two years after treatment,
- Length of each site: Five miles,
- Traffic volume: 2,000 vehicles per day,
- Crash rate: Five crashes per million vehicle miles,
- Mean crash frequency at each site: 36.5 crashes in 2 years (from above),
- Variance in crashes: 25 percent of the mean frequency or 9.1 crashes in 2 years,
- Correlation coefficients: 0.50 between crashes before and after time of treatment and 0.25 between before-after crash decrement at treatment site and that at the matched control site, and
- Size of treatment effect to be detected: 1 percent of the untreated mean crash frequency to reflect the approximate size necessary for cost effective treatment.

For the above experiment, the number (n) of site pairs required in the sample to maintain acceptable levels of risk is (2):

$$n = \frac{(z_{1-\alpha} + z_{1-\beta})^2}{d^2} \tag{4}$$

in which z is the normal variate, α is the level of significance, β is the probability of committing a Type II error, and d is given as follows:

$$d = \frac{(m_T - m_C)}{\sigma} \tag{5}$$

in which $(m_T - m_C)$ is the value of the average difference in crash frequency that is to be detected and σ is the standard deviation of the relative change in crash frequency of a site pair resulting from treatment. Given the above assumptions, the value of d can be shown to be 0.09904 and

$$n = \frac{(z_{1-\alpha} + z_{1-\beta})^2}{0.009809} \tag{6}$$

The typical crash investigation of a feature such as wide edgelines might involve 10 to 30 test pairs (100 to 300 mi of roadway in the context of the example given here). The level of significance is commonly 0.05. Given the above assumptions, and by using Equation 6, the probability of not detecting a real 1 percent crash effect (i.e., the probability of committing a Type II error) is very large, of the order of 0.87 to 0.91. If the level of significance is relaxed to 0.10 and if β is set to 0.40 (still a large risk), the required number of test pairs is about 240, corresponding to about 2,400 mi of highway. Seldom is crash data of the type required for such an analysis available for more than 2,000 mi of roadway, and even when it is, the risk of error remains large if the treatment effect is small.

Although it is important to avoid direct comparisons between the Virginia experiment and this hypothetical example, the example does illustrate that large mileages may be required for crash studies in which observation of small treatment effects is important. It further illustrates that interpretation of the Virginia findings is incomplete without an assessment of the risks of committing a Type II error. In summary, although the Virginia study has produced useful new data, it has not conclusively established the safety effects of wide edgelines. Because wide edgelines offer promise as a cost-effective accident countermeasure, a great deal could be lost if continued experimentation is prematurely abandoned.

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DISCUSSION

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Although this paper is commendable in both the application and use of the statistical before-after methodology, the description of the methodology, including mathematical notations, is taken directly from Griffin's work. Although Griffin is acknowledged, the author should ensure that Griffin receives adequate recognition for his work, which is literally duplicated in parts of this paper. As an alternative, the author might have wished to eliminate the portions of text that come from Griffin's article and simply refer the reader to that article for detail.

Given that the results of the traditional before-after design using the less powerful Poisson test were, in the author's words, "mixed, inconsistent, and inconclusive," Table 7 should have been omitted. Inclusion of these results will naturally lead to misuse and misinterpretation.

The conclusion of this paper is too strong and is not supported by the sparse and limited data presented. All that can be said is that at these locations, no significant safety effect due to edgelining could be found. To make inference to all edgelined sections on the basis of these three nonrandomly selected locations is not valid.

Finally, the author has made the all-too-common error of interpreting partitioned chi-squares when the overall chi-square is not significant. This is equivalent to the analysis of variance analog of interpreting model parameters when the overall F-statistic is not significant. Specifically, for Locations 2 and 3 of Table 2, since the overall (total) chi-square is not significant, it is incorrect to attach any meaning to the chi-square statistics for homogeneity or treatment.

AUTHOR'S CLOSURE

I would like to thank John A. Deacon and Olga J. Pendleton for commenting on this paper. Their comments and interest in my work are greatly appreciated. The paper has been improved by their contributions.

Deacon's point is that the results of the paper should be viewed with caution because the sample size was too small to detect a small treatment effect, such as a 1 percent decrease in accident frequency. Before this research was initiated, I carefully reviewed the reference cited by Deacon. This work describes several scenarios of a sample size range of values for α, β, and the size of the treatment effect to be detected. From this, it was apparent that 2,000 mi or more of roadway was necessary to obtain "acceptable" values of α, β, and the size of the treatment effect to be detected. Limited resources did not permit me to pursue such a large-scale effort.

It is noted that the value of d in Deacon's hypothetical analysis should be 0.12, yielding $\beta = 0.94$ for $n = 10$ to 30. A 1 percent treatment effect is small enough to be quite difficult to detect. If study data for ROR accidents are used, for $\alpha = 0.10$, $n = 3$, and size of treatment effect to be detected = 10 percent; β is equal to 0.70. Admittedly, this β value is high. Note that even for $n = 240$ with $\alpha = 0.05$, treatment effect to be detected = 0.01 and $\beta = 0.40$, as stated by Deacon, "the risk of error remains large if the treatment effect is small." Because of the sample size, no inferences are made.

Given the limitation on the sample size, a special effort was made to improve the statistical method of the research by using (a) a statistically rigorous and valid study design and (b) Gart's procedure to combine the three study locations, thereby increasing the power of the test. Moreover, study locations with fairly high ROR accident frequencies were selected to permit the study of a higher number of ROR accidents in lieu of additional miles of roadway.

In this research, the study locations were combined by using Gart's procedure, including weighting the locations on the basis of accident frequency. Higher weight is given to locations with higher accident frequency. When weighting is used to combine the data for the three study locations in Table 8, the following values are obtained:

	Recommended	*Deacon's*
Total reduction in ROR + OD crashes expected	5.02	17.28
Reduction as a percentage of ROR + OD crashes	3.95	13.61
Reduction as a percentage of all types	2.04	7.02

These values are substantially lower than those in Table 8 because the location with the largest reduction had the lowest accident frequency. The recommended values are more appropriate, better reflect the actual changes, and are more reliable.

Three changes were made in the paper in response to Pendleton's discussion. I have emphasized the contributions of Lindsay I. Griffin III through additional discussion in the paper and the acknowledgments. Griffin's review of the paper and his contributions to this closure are also greatly appreciated. The methodology was discussed because (a) most readers are probably unfamiliar with the methodology and (b) the methodology has not been cited in journals or reports widely distributed to transportation professionals.

The discussion on Table 7 was revised to emphasize that by including Table 7, the advantages of the before-after design with a comparison group and check for comparability are demonstrated.

The conclusion in the preprint was incorrectly stated because it was inconsistent with analysis findings and the limited sample size. The revised conclusion is that there was no evidence that wide edgelines significantly affected the ROR accident frequency or related accident types for each study location or for the combined locations at a 0.05 level of significance.

The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the sponsoring agencies.

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Reliability and Risk Assessment in the Prediction of Hazards at Rail-Highway Grade Crossings

ARDESHIR FAGHRI AND MICHAEL J. DEMETSKY

The principles of reliability and risk assessment were applied in a model for the evaluation of rail-highway grade crossings and the prioritization of improvements. The performance of this newly developed model was evaluated and compared with the performance of five other nationally recognized models—the DOT, Peabody-Dimmick, NCHRP 50, Coleman-Stewart, and New Hampshire—by using a data base maintained by the Virginia Department of Transportation. The results indicated that because of the probabilistic nature of the model, its performance was exceptional when compared with that of the other models. The developed model is seen as a valuable prediction tool, but more important, it demonstrates the potential for applications of reliability and risk assessment in transportation.

Industrial and government planners, managers, engineers, and researchers have long recognized the importance of risk and uncertainty considerations in engineering tasks. These considerations, however, have not been central to policy formulation until recently. This trend toward consideration of risk and uncertainty has been accompanied by a rapid proliferation of literature on the subject of risk, indicating that both the professional and general public are becoming aware of the need to consider uncertainty in engineering decisions.

A careful examination of many basic engineering problems shows the various roles of risk analysis at decision points. In general, risk and uncertainty analysis includes identifying, quantifying, and evaluating risk, understanding the perception of risk, and determining the level of risk that is acceptable within a particular social and technical context.

The focus of this paper is the problem of measuring hazardous indices for rail-highway crossings. This problem was selected because uncertainty is not explicitly considered in the derivation of methods that are currently being used in the United States. The analysis in this paper deals with the problem of identifying the risk. After identification, the risk is reflected as a quantifiable metric that is used as one consideration in a multiattribute design process that allocates improvement funds to selected crossing sites.

Various empirical formulas for calculating hazard indices for rail-highway grade crossings have been developed by various organizations and researchers. One type, the relative formula, provides a measure of the relative hazards or the accident expectations at various types of railway crossings. These indices may be used to rank a large number of crossings in order of priority for improvements. The crossing with the highest hazard index is regarded as potentially the most dangerous and hence the most in need of attention. Another type of formula is called an absolute formula because it forecasts the number of accidents that is likely to occur at a crossing or a number of crossings over a certain time period and the number of accidents that may be prevented by making improvements at these crossings.

In a recent study conducted through the Virginia Transportation Research Council (*1*), Faghri and Demetsky evaluated five nationally recognized models for predicting rail-highway crossing hazards: the Department of Transportation (DOT), Peabody-Dimmick (P-D), NCHRP 50, Coleman-Stewart (C-S), and New Hampshire (N.H.). The general formats of these models are as follows:

NCHRP 50 Method

$$EA/\text{year} = A_1 X\ B_1 X\ \text{trains/day}$$

New Hampshire Formula

$$\text{Hazard Index} = V\ T\ P_f$$

DOT Accident Prediction Formula

$$A = \frac{T_0}{T_0 + T}(a) + \frac{T}{T_0 + T}\ \frac{N}{T}$$

where

$$a = K \times EI \times MT \times DT \times HP \times MS \times HT \times HL$$

Coleman-Stewart Model

$$\log \bar{A} = C_0 + C_1 \log \bar{V} + C_2 \log \bar{T} + C_3 (\log \bar{T})^2$$

Peabody-Dimmick Formula

$$I = 1.28\ \frac{H^{0.170} \times T^{0.151}}{P^{0.171}} + K$$

where

Virginia Transportation Research Council, Department of Transportation, P.O. Box 3817 University Station, Charlottesville, Va. 22903.

EA = expected number of accidents;
A_1, B_1 = empirical adjustment factors;
V = average 24-hour traffic volume;
T = average 24-hour train volume;
P_f = protection factor;
A = final accident prediction, accidents per year at the crossing;
N = number of observed accidents;
T = number of years;
T_0 = formula weighing factor;
a = initial accident prediction, accidents per year at the crossing;
K = constant for initialization of factor values at 1.00;
EI = factor for exposure index based on product of highway and train traffic;
MT = factor for number of main tracks;
DT = factor for number of through trains per day during daylight;
HP = factor for highway paved;
MS = factor for maximum timetable speed;
HT = factor for highway type;
HL = factor for number of highway lanes;
$\bar{A}$ = average number of accidents per crossing year;
$\bar{V}$ = weighed average daily traffic volume for the N crossings;
$\bar{T}$ = weighed average train volume for the N crossings;
C_0, C_1, C_2, C_3 = empirical factors;
H = average number of vehicles in 24 hours;
T = number of trains per day;
P = protection type coefficient; and
K = additional adjusting parameter.

The DOT, Peabody-Dimmick, NCHRP 50, and Coleman-Stewart are absolute formulas. The New Hampshire model is a relative formula.

The results of this comparative study indicated that the DOT model was more accurate than the rest of the group in predicting rail-highway crossing hazards; thus it was recommended for use in Virginia. During the evaluation process, however, several problematic common features were observed among the five models. These included the following:

- The models were developed by using nationwide data.
- The parameters were determined through linear regression techniques (except the DOT model, which was developed by using nonlinear regression analysis).
- None of the absolute models are expected to predict the exact number of accidents that will occur at a crossing. At best, they can predict only the mean number of expected accidents at a crossing during an extended time period. However, the expected value is a better indicator of the number of accidents that will occur at a location than is a mere review of that location's history (2).

Given that the problem deals with a random variable, the occurrence of accidents at a crossing, it is odd that probabilistic approaches have not been developed. The foregoing observations motivated an investigation into the feasibility of approaching the problem from a probabilistic viewpoint. Accordingly, the mathematic principles of reliability and risk assessment were used to establish a hazard index for a crossing on the basis of the probability that an accident would occur at the crossing. Before the problem is formulated, however, a brief summary is presented of the concepts and fundamentals of reliability and risk assessment that apply.

FUNDAMENTAL CONCEPTS OF RELIABILITY AND RISK ASSESSMENT

Risk analysis, which is a subset of safety analysis, requires consideration of the probability of an accident's occurrence and its consequences (3). Reliability and risk analysis have had a wide variety of applications in nuclear engineering (3), chemical engineering (4), and civil engineering (5).

In this work, the probability per unit time that an undesirable event may occur is estimated by using the fundamentals of reliability theory and is expressed as the expected frequency with which the event might be initiated (3). To formulate the probability concepts of failure analysis, two types of systems are considered: those that operate on demand and those that operate continuously. Demand failures occur in a system during its intermittent, possibly repetitive, operation: either the system operates at the nth demand (event D_n) or it does not operate (event $\bar{D}_n$). The probability $P(W_{n-1})$ that the system works for each of $n - 1$ operations is the intersection of the probabilities of success for each operation:

$$P(W_{n-1}) = P(D_1 D_2 \ldots D_{n-1}) \tag{1}$$

The fact that the system works for $n - 1$ operations does not mean that it will operate at the nth demand. That is, $P(D_n | W_{n-1})$ is the conditional probability that the system will operate at the nth demand, given that the system works for $n - 1$ demands. $P(\bar{D}_n | W_{n-1})$ is the corresponding conditional probability of failure. The probability that a system will not operate on the nth demand when it has worked for all previous demands is

$$P(\bar{D}_n W_{n-1}) = P(\bar{D}_n | W_{n-1}) P(W_{n-1}) \tag{2}$$

Equation 2 may also be written as

$$\begin{aligned} P(D_1 D_2 \ldots D_{n-1} \bar{D}_n) = {} & P(\bar{D}_n | D_1 D_2 \ldots D_{n-1}) P \\ & \times (D_{n-1} | D_1 D_2 \ldots D_{n-2}) \ldots P \\ & \times (D_2 | D_1) P(D_1) \end{aligned} \tag{3}$$

Ideally, for demand-type failures there should be available a complete tabulation of all the probabilities in Equation 3 for every intermittently operating component in a system. Because of limitations in the experimental data available, it is usually necessary to assume that the demand events are identical and independent. Any failure is then assumed to be random so that

$P(\bar{D}_n|W_{n-1}) = P(\bar{D})$ and $P(D_n|W_{n-1}) = P(D)$. In such a case,

$$P(D_1 D_2 \ldots D_{n-1}) = [P(D)]^{n-1} = [1 - P(\bar{D})]^{n-1} \quad (4)$$

and

$$P(D_1 D_2 \ldots D_{n-1}\ \bar{D}_n) = P(D_1 D_2 \ldots D_{n-1})\, P(\bar{D}_n) = P(\bar{D})\,[1 - P(\bar{D})]^{n-1} \quad (5)$$

In this case, only the demand failure probability $P(\bar{D})$ needs to be tabulated.

For systems that are in continuous operation and that do not undergo repair, the analog to Equation 2 is given as

$$f(t)\, dt = \lambda\, (t)\, dt\, [1 - F(t)] \quad (6)$$

where

$f(t)\, dt$ = probability of failure in dt about t;
$\lambda\,(t)\, dt$ = probability of failure in dt about t, given that it survived to time t; and
$1 - F(t)$ = probability that the device did not fail prior to time t.

Another way of saying the same thing is

$$f(t) = \lambda\, (t)\, [1 - F(t)] \quad (7)$$

where $f(t)$ is the failure probability density, that is, the probability of failure in dt about t per unit time. The term $\lambda(t)$ is the conditional failure rate and is often called the hazard rate; the units of $\lambda(t)$ are inverse time.

Reliability, $R(t)$, is defined as the probability that a specified fault event has not occurred in a system for a given period of time and under specified operating conditions. In other words, reliability is the probability that a system performs a specified function or mission under given conditions for a prescribed time. Reliability is the complementary probability of $F(t)$, that is,

$$R(t) = 1 - F(t) \quad (8)$$

In other words, $F(t)$ is the unreliability, the probability that the device or system will fail at some time between 0 and t, and $R(t)$ is the probability that it will not fail during that time period.

A summary of equations relating $\lambda(t)$, $R(t)$, $F(t)$, and $f(t)$ is presented in Table 1. Derivations of these formulas may be obtained elsewhere (*3*).

To formulate the failures of components mathematically, several probability distributions that describe such failures are used. For systems whose operations are intermittent, discrete probability distributions are used, and systems whose operations are continuous can be described by continuous probability distributions. Some of the most common probability distributions that are applied in reliability engineering problems are presented in Table 2.

To summarize,

- Two conditional failure probabilities are used in reliability: the failure/demand and the failure/unit time (or hazard rate).
- The hazard rate $\lambda(t)$ contains all the information needed to study failures of a system. If $\lambda(t)$ is not known with certainty, statistical estimation procedures must be used to estimate the value of λ (*3*).

The fundamental relationships defined in Equations 1–8 and the selection of an appropriate probability distribution now provide the means for applications of reliability and risk assessment in rail-highway hazards prediction.

APPLICATION

The ideal hazard prediction technique for rail-highway grade crossings is an equation that accurately predicts the frequency of accident occurrence by taking into account all variables that have some influence on the event. From a practical point of view, such an equation is too large and the data requirements too extensive to be of any value. Also, accidents are influenced by such factors as driver skill and perception, certain environmental conditions, and other factors that are at many times impossible or too costly to accurately quantify in any consistent way. Finally, accidents occur from essentially random causes; consequently, any predictive equation is bound to explain less than 100 percent of accident behavior, even in the very long run.

Accordingly, such an equation should not be expected to predict the exact number of accidents that will occur at a given time period. At best, it can predict the expected number of

TABLE 1 SUMMARY OF RELIABILITY EQUATIONS (*3*)

Word Description	Symbol	=	First Relationship	=	Second Relationship	=	Third Relationship
Hazard rate	$\lambda(t)$		$-(1/R)\, dR/dt$		$f(t)/[1 - F(t)]$		$f(t)/R(t)$
Reliability	$R(t)$		$\int_t^\infty f(\tau)\, d\tau$		$1 - F(t)$		$\exp\left[-\int_0^t \lambda(\tau)\, d\tau\right]$
Cumulative failure probability	$F(t)$		$\int_0^t f(\tau)\, d\tau$		$1 - R(t)$		$1 - \exp\left[-\int_0^t \lambda(\tau)\, d\tau\right]$
Failure probability density	$f(t)$		$dF(t)/dt$		$-dR(t)/dt$		$\lambda(t)R(t)$

TABLE 2 PROBABILITY DISTRIBUTIONS USED IN RELIABILITY ANALYSIS

Name	Function
Discrete Distributions	
Binomial	$P(r) = (n!)\ [r!(n - r)!]^{-1}\ [P(D)]^r\ [P(D)]^{n-r}$ where n is the number of demands or trials that an experiment consists of and r is a random variable, defined to be the number of demands for which the system fails.
Poisson	$P(r) = (\exp -\mu\ \mu^r)\ (r!)^{-1}$ where μ is the most probable number of occurrences of an event.
Continuous Distributions	
Erlangian	$f(t) = [\lambda(\lambda t)^{r-1} \exp -\lambda t]\ [(r - 1)!]^{-1} \quad \lambda > 0,\ r \geq 1$ where λ is the hazard rate.
Exponential	$f(t) = \lambda \exp - \lambda t$
Gamma	$f(t) = [\lambda(\lambda t)^{r-1} \exp -\lambda t]\ \Gamma(r)]^{-1} \quad \lambda > 0,\ r > 0$ where $\Gamma(r)$ is the gamma function.
Lognormal	$f(t) = (\sqrt{2\pi}\alpha t)^{-1} \exp \{-[\ln (t/\beta)]^2\ (2\alpha^2)^{-1}\} \quad \alpha,\ \beta > 0$ where α is the shape parameter (dimensionless) and β is the scale parameter or "characteristic life" (in units of time).
Weibull	$f(t) = \alpha/\beta\ [(t - \tau)/\beta]^{\alpha-1} \exp\{-[(t - \tau)/\beta]^{\alpha}\} \quad \alpha > 0,\ \beta > 0,\ 0 \leq \tau \leq t \leq \infty$ where τ the time delay parameter.

accidents at a crossing during a given time period. Any change that occurs in the variables of the equation alters the mean number of expected accidents. Thus the forecasted expected value is considered by statisticians to be a better indicator of the number of accidents that will occur at a location than that location's history.

The probability of an accident at a rail-highway crossing has been formulated as follows (2):

$$\lambda = P = R(K + S) \tag{9}$$

where

$\lambda = P$ = probability of the event of an accident,
K = probability of a vehicle arriving at a grade crossing occupied by a train,
S = the probability of a train arriving at a grade crossing occupied by a vehicle, and
R = the risk that a driver will be unaware of his surroundings and hence will not (or perhaps will be unable to) take the evasive action necessary to avoid a pending collision.

$R = 1$ implies total risk (unswerving drivers who completely ignore onrushing trains or are completely unaware of an obstacle in their path), and $R = 0$ implies perfect information and complete awareness, hence no risk.

"Risk" defined in the foregoing way includes both cases in which a train occupies the crossing and cases in which a train is approaching the crossing:

$$P = rK + RS \tag{10}$$

in which r and R are the corresponding risks for the two situations. Furthermore, P would also be expected to be a function of warning devices. This would change Equation 10 to

$$P = C(rK + RS) \tag{11}$$

in which C is a coefficient that depends on the type of protection at the crossing.

Early accident statistics indicate that accidents that could be predicted by the function CrK account for about 35 percent of the accidents involving trains. However, further analysis indicates that unless the crossing is used by extremely slow-moving trains at night, the value of r drops so low when a train is occupying the crossing prior to the motorists' final opportunity to stop that it is almost negligible (2). For mathematical expediency, this allows the return to an assumption of a common formula for all cases:

$$P = CR'S' \tag{12}$$

where R' is the risk of operation perception and S' is the probability of a vehicle arriving at a grade crossing occupied by another vehicle.

This approach was necessary because the Virginia data base contained data for both types of accidents (i.e., the accidents with trains occupying the crossing and accidents with vehicles occupying the crossing) and does not differentiate between them. Also, this modified formula provides a level of mathematics suitable for developing a usable model.

Now, because S' is the probability of a train arriving in a given second of time and a vehicle arriving in a given 2 to 3 sec,

$$S' = ab \tag{13}$$

where a is the probability of a train arriving in a given second and b is the probability of a vehicle arriving in a given 2 to 3 sec. Although the logic of a 2- to 3-sec arrival interval seems to be good, the statistics do not entirely support it (2). For example 2.5 times as many accidents occur in the 1-sec interval (moving train hits a moving car) as occur in the 2- to 3-sec interval (moving train appears on the crossing after the driver has gone beyond his final opportunity to stop). During those 2 to 3 sec the driver still has alternatives of evasive action, even though he cannot stop. He can run off the road or he can hit an object other than the train. He can also accelerate and possibly cross the tracks before the train arrives. For the purposes of the accident model, a highway risk time of 1 sec is used.

The flow of traffic on a facility is a function of the time of the day, which makes it desirable to estimate hourly traffic flow rates. However, there is a high degree of randomness within any hour. If it is given that V_h is the volume of traffic in the hth hour but randomness is assumed within that hour, the probability that no vehicle crosses a predetermined point on a roadway in a randomly chosen second of time is $\exp -V_h/T_h$ (assuming Poisson arrivals), where T_h is the number of seconds in an hour. Therefore the probability of at least one random arrival in a chosen second is $1 - \exp -V_h/T_h$. Because of the low volume of trains, the approximation of Z_t/T_t (in which Z_t is the number of trains in the time period) is valid for almost any distribution that may be used. The information available for this study was the number of trains per day and the average daily traffic. Thus

$$b = 1 - (\exp -V/24 \times 3{,}600) \tag{14}$$

and

$$a = Z/24 \times 3{,}600 \tag{15}$$

DISCUSSION OF VARIABLES

Protection Type (C)

Previous research in the form of before and after studies has developed relative hazard relationships for the various protection types. If crossbuck protection is set equal to one, the relative hazard is as follows:

Protection	*Hazard*
Crossbucks	1.00
STOP signs	0.65
Wigwags	0.34
Flashing lights	0.30
Gates	0.17

Risk Factor (R')

R' was defined as the risk that a driver will be unaware of his surroundings when a train is approaching and therefore will not take the evasive action necessary to avoid collision. R' can also be expected to be a function of the physical features at the crossing. Features such as angle of crossing, highway speed, train speed, sight distance, visibility, number of lanes, and others can alter the risk. $R' = 1$ implies total risk, that is, unswerving drivers who completely ignore on-rushing trains or are completely oblivious to an obstacle in their path. $R' = 0$ implies perfect information and complete awareness, hence no risk. All models in the literature use regression analysis techniques to find the correlation between the number of accidents and site variables. In this study, the risk factor for each crossing was determined by using all the variables that were used in the DOT model, which were then normalized to be used as probabilities in the final formulation. These variables are factor for exposure index based on product of highway and train traffic, factor for number of main tracks, factor for number of through trains per day during daylight, factor for highway type, and factor for number of highway lanes. The variables from the DOT model were used because, as will be shown later, this model had the highest predictive power. However, if there are other relevant factors (such as school bus traffic and sight distance) in an agency's data base, they may also be included in R'. The more relevant variables are included in the value of R', the more accurate the final results will be.

Final Formulation

Once all the variables have been defined, the probability of occurrence of an accident per second per crossing can be stated as

$$P = CR'ab \tag{16}$$

This probability per unit time (P) can be looked on as the hazard rate (defined earlier) for each crossing. If each crossing is considered as a separate system and random failures are assumed for each system [i.e., those failures for which the hazard rate $\lambda(t)$ is a constant], the Poisson discrete distribution can be used to derive the final form of this equation. The probability of exactly r failures occurring in time t is given by

$$P(r;\, t) = \exp -\lambda t\, (\lambda t)^r/r! \tag{17}$$

and the cumulative probability of X or fewer failures is

$$P(X < x;\, t) = \sum_{r=0}^{x} \exp -\lambda t\, (\lambda t)^r/r! \tag{18}$$

Equation 18 permits calculation of the failure probability density $f(t)$ for the rth failure in dt about t. What is required, of course, is for the system to have undergone $(r - 1)$ prior failures so that it is ready to fail for the rth time with a conditional probability λ [i.e., $P(r - 1 | r) = \lambda$, because λ is constant]. Thus the Erlangian distribution (time-dependent form of the Poisson discrete distribution) follows, as

$$f(t) = P(r - 1,\, t) = \lambda(\lambda t)^{r-1} \exp -\lambda t/(r - 1)!$$
$$\lambda > 0,\ r \geq 1 \tag{19}$$

The Erlangian distribution is valid for an integer number of failures r. The most important special case is for $r = 1$, in which case the exponential distribution is obtained as

$$f(t) = \lambda \exp -\lambda t \tag{20}$$

The cumulative failure probability for the exponential distribution is

$$F(t) = 1 - \exp -\lambda t \tag{21}$$

and the reliability is

$$R(t) = \exp -\lambda t \tag{22}$$

Substituting the value of λ in Equation 22 for each crossing gives

$$R(t) = \exp -(CR'ab)\, t \tag{23}$$

or

$$R(t) = \exp [CR'(1 - \exp -V/24 \times 3{,}600) \times (Z/24 \times 3{,}600)]t \tag{24}$$

By using Equation 24, the reliability of each crossing can be determined over a certain period of time.

This model was applied to the 1,536 rural public grade crossings that define the data base maintained by the state of Virginia, and the results were saved on a microcomputer hard disk for comparison with the other models. The methodology for comparing the models is discussed in the following section.

METHODOLOGY

The technique used for the comparison of representative models in this study was the power factor (PF) test. This test, which compares models for their hazard prediction capability, was first described by Mengert (6) and is defined as follows. The 10 percent power factor is the percentage of accidents that occur at the 10 percent most hazardous crossings (as determined by the given hazard index) divided by 10 percent. The same sort of definition holds for the 5 percent power factor, and so on. Thus, if PF (5 percent) = 3.0, then 5 percent of the crossings account for 15 percent (3 × 5 percent = 15 percent) of the accidents (when the 5 percent considered is the 5 percent most hazardous, according to the hazard index in question).

The PF can be seen as a primary measure of the usefulness of a hazard index for relative rankings of crossings. As an example, suppose that 10 percent of a certain group of crossings is to be selected for improvement, and assume that the most hazardous crossings are to be selected for this purpose. Then, if a given hazard index is used, the 10 percent most hazardous crossings will be selected according to that hazard index. The number of accidents that may be expected at these selected crossings in any period of time is proportional to the PF for the given hazard index. The greater the proportion of the total accidents that would occur at the crossings selected as the most hazardous, the more effective the hazard index, as evidenced by the PF. In fact, for some purposes, the payoff (or benefits) will be proportional to the number (or proportion) of accidents that would occur at the selected crossings because these accidents may be partially or totally prevented. Consequently, when the hazard index is to be used for selecting the 10 percent most hazardous crossings, the 10 percent PF seems to be the most direct measure of its effectiveness. The same would hold for the 20 percent power factor if 20 percent of the crossings were to be selected, and so forth.

RESULTS

To evaluate the performance of the new reliability-based model, the 1 percent, 2 percent, 3 percent, 6 percent, 10 percent, 20 percent, and 40 percent power factors of all the crossings in the data base were determined for each of the models. The results of the power factor test are shown in Tables 3

TABLE 3 POWER FACTORS OF EACH MODEL

Crossings (%)	Incremental Accidents	Cumulative Accidents	Accidents (%)	Power Factor
DOT Model				
1	5	5	3.10	3.10
2	6	11	6.83	3.42
3	3	14	8.69	2.90
6	11	25	15.52	2.58
10	11	36	22.36	2.24
20	30	66	40.99	2.05
40	42	108	67.08	1.68
NCHRP 50 Model				
1	4	4	2.48	2.48
2	6	10	6.21	3.10
3	3	13	8.07	2.69
6	14	27	16.77	2.79
10	11	38	23.60	2.36
20	27	65	40.37	2.01
40	33	98	60.86	1.52
New Hampshire Model				
1	5	5	3.10	3.10
2	5	10	6.21	3.10
3	0	10	6.21	2.07
6	9	19	11.80	1.96
10	20	39	24.22	2.42
20	25	64	39.75	1.98
40	33	97	60.25	1.51
Coleman-Stewart Model				
1	2	2	1.24	1.24
2	5	7	4.34	2.17
3	3	10	6.21	2.07
6	10	20	12.42	2.07
10	12	32	19.87	1.98
20	31	63	39.13	1.96
40	44	107	66.45	1.66
Peabody-Dimmick Model				
1	4	4	2.48	2.48
2	3	7	4.34	2.17
3	3	10	6.21	2.07
6	10	20	12.42	2.07
10	15	35	21.74	2.17
20	30	65	40.37	2.02
40	37	102	63.35	1.58
Reliability Model				
1	3	3	1.86	1.86
2	10	13	8.07	4.04
3	5	18	11.18	3.72
6	8	26	16.14	2.69
10	13	39	24.22	2.42
20	27	66	40.99	2.05
40	40	106	65.83	1.64

TABLE 4 RANKING OF THE MODELS IN THE POWER FACTOR TEST

Crossings (%)	Rank[a] 1	2	3	4	5	6
1	DOT	N.H.	NCHRP 50	P-D	Reliability	C-S
2	Reliability	DOT	N.H.	NCHRP 50	P-D	C-S
3	Reliability	DOT	NCHRP 50	N.H.	P-D	C-S
6	Reliability	NCHRP	DOT	P-D	C-S	N.H.
10	Reliability	N.H.	NCHRP 50	DOT	P-D	C-S
20	Reliability	DOT	P-D	NCHRP 50	N.H.	C-S
40	DOT	C-S	Reliability	P-D	NCHRP 50	N.H.

[a]Rank 1 has the highest power factor, Rank 5 the lowest.

and 4. Table 3 presents the power factors of each model separately for the previously mentioned percentages of hazards, and Table 4 presents the results of using the power factors to rank the models according to their hazard prediction capability.

The two tables indicate the stability and the exceptional performance of the reliability model. The probability distribution that was selected in this study to describe the reliability of crossings turned out to be a more realistic hazard predictor for the crossings than other models because of the random nature of the accidents that take place at the crossings.

CONCLUSION

Through application of the probabilistic concepts of reliability and risk assessment, a reliability-based model was developed for determining the reliability of rail-highway grade crossings in the state of Virginia. This model can be used as a prediction tool for evaluating and prioritizing rail-highway grade crossings for any period of time. The main improvement of the model over other available techniques is its probabilistic nature. The results of the comparison of this model and five other nationally recognized models show the stability and superior performance of this model as a relative hazard predictor.

The potential applications of reliability and risk assessment in a variety of transportation-related problems are evident from this paper. Through careful formulation, many dangerous and hazardous situations in transportation and traffic can be described by using this theory. Model sensitivity to the issue of whether a train occupies a grade crossing or a vehicle occupies a grade crossing can only be clearly resolved when future data bases differentiate this condition for observed accidents. The current solution to the question of whether a train or a vehicle occupies a grade crossing was expedited by the fact that the data base used did not differentiate between the two types of situations. This necessitated the use of a practical mathematical formulation. A more complex model that will differentiate between the vehicles that might occupy the crossing should be addressed in further research, and the trade-offs between accuracy and computational efficiency should be evaluated.

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Motorist Understanding of Railroad-Highway Grade Crossing Traffic Control Devices and Associated Traffic Laws

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The findings of a survey conducted in Tennessee to evaluate motorist comprehension of railroad grade crossing traffic control devices and associated traffic regulations are documented. The questionnaire survey was administered to 176 drivers and to 35 city police officers. The survey gathered input on driver recognition and understanding of common grade crossing traffic control devices, including signs, pavement markings, flashing light signals, gates, and train whistles, as well as driver perceptions of train capabilities and operating requirements. The survey results reveal that there are deficiencies in driver comprehension of several of the common crossing traffic control devices. Specifically, many drivers are uncertain or are misinformed about the applications of the crossbuck and advance railroad warning signs and about driver responsibilities at passive crossings and crossings with flashing light signals. Many drivers believe that a train operator can and should assume part of the responsibility for crossing safety by slowing or stopping the train. The survey also revealed that most drivers perceive a need to improve crossing safety. They recommend that gates, flashing lights, or both be installed at more crossings, driver education be increased, and more grade separations be constructed. Police officers, although they performed better than the general driving public on the survey, also demonstrated a lack of comprehension of some grade crossing traffic control devices and safety issues.

There are approximately 205,000 public railroad-highway grade crossings in the United States and an additional 150,000 private crossings. These crossings represent a unique and potentially hazardous driving situation, accounting for about 500 fatalities and over 2,500 injuries each year. To help motorists cope with the hazards, a number of traffic control and warning devices (and associated traffic regulations) have evolved and are recommended for use. These devices include the crossbuck sign, advance railroad warning sign, flashing light signals, automatic gates, bells, advance crossing pavement markings, and train horns. A basic presumption about all of these devices is that motorists understand their intended meanings and applications; otherwise, their usefulness as warning and regulatory devices is questionable.

Numerous studies (*1*) have addressed the operational and safety performance of railroad-highway grade crossing devices. Generally, these studies have revealed deficiencies in motorist response to many of the traffic control devices now in use. However, there has been only limited research into motorist understanding and comprehension of these devices. The studies that have been conducted have suggested that there may be serious shortfalls in motorist comprehension of grade crossing devices and associated traffic regulations and that these deficiencies may account for some of the performance problems observed at crossings.

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In one study, Sanders et al. (*2*) investigated driver knowledge and attitudes concerning grade crossing traffic control and related drivers' knowledge and attitude to their observed behavior. The study concluded that motorists' ability to make correct decisions at grade crossings is related to their knowledge of and attitudes toward the crossing traffic control. Drivers who were observed performing more safely at crossings had seen and correctly interpreted the traffic control device present.

The Sanders study also found deficiencies in motorist's comprehension of some of the traffic control devices commonly used at crossings. For example, the study found that 15 percent of the drivers in the sample believed that all crossings have active warning devices. More than 40 percent of the drivers believed that the elapsed time between flashing signal activation and train arrival was greater than one minute.

Womack et al. conducted a study in Texas that addressed driver comprehension of the railroad advance warning sign (*3*). The study found that 42 percent of the sampled drivers were unaware that this warning sign was circular, and 60 percent were unaware that it was yellow. More importantly, 64 percent believed that the sign was placed at a crossing rather than in advance of the crossing, and 70 percent said they would not necessarily expect to see a crossbuck sign following the advance sign. In addition, 17 percent said that they would "stop and look for trains" upon seeing a railroad advance warning sign.

All states, including Tennessee, have adopted laws concerning driver duties and actions at grade crossings. Most of these laws have the same or very similar wording as contained in the Uniform Vehicle Code (UVC) (*4*). In adopting the wording of the UVC, states have removed the concept of "stop, look, and listen" applying to all motorists approaching all crossings and have instead outlined, in specific terms, driver's obligations in operating their motor vehicles when certain conditions exist at a crossing.

A driver can proceed through a crossing with activated flashing light signals after stopping, but only when it is safe to do so. However, a person is not permitted to drive any vehicle through, around, or under any crossing gate or barrier while the gate arm is down or being opened or closed.

With regard to passive crossings, a driver is required to stop if a train is an immediate hazard or is in hazardous proximity to the crossing. However, if a driver cannot see or hear the train, there is no obligation set forth in the UVC or most state laws for drivers to stop or even slow down. In other words, all motor vehicles are not required to always stop or slow at all railroad-highway grade crossings. In fact, there are no requirements whatsoever to slow a motor vehicle down on an approach to a railroad-highway grade crossing or to take precautions other than those that would be required when traveling through a normal highway intersection.

The training that drivers receive, whether in a high school driver education course or in the state driving training manuals, will normally follow the state's legislative requirements for drivers at crossings. However, some public service programs tend to provide information that is contradictory to the law in the states and that therefore may be contributing to confusion on the part of motorists regarding highway-railroad grade crossings, whether the drivers had formal training or not.

A driver survey similar to a survey conducted by Tidwell and Humphreys (5) was conducted in Tennessee. In addition to assessing motorist comprehension of standard and innovative crossing traffic control devices, the survey gathered input on driver awareness of the grade crossing safety problem, the level of driver education relative to crossing traffic control and regulation, and driver suggestions for traffic control improvements.

SURVEY DESCRIPTION

Survey Instrument

Data for the research were gathered by using a questionnaire with 16 multiple-choice and 1 short-answer questions. These questions were designed to evaluate driver knowledge, recognition, and comprehension in the areas of adequacy of instruction and training on grade crossing devices and traffic regulations, two signs commonly used at crossings (the advance railroad crossing sign and crossbuck sign), understanding of and experience with flashing light signal installations, crossing gates, advance railroad crossing pavement markings, understanding of the passive traffic control strategy, train operation and train operator's responsibilities at a crossing, and suggestions on needed remedies or improvements for crossing safety.

Sampling Plan

The survey was conducted in three Tennessee cities: Nashville, Chattanooga, and Knoxville. An effort was made to obtain an unbiased and representative sample of the state's driver population by randomly surveying motorists as they renewed their driver's licenses. Subjects were recruited on a volunteer basis, that is, they were not paid for their services.

The majority of the questionnaires were administered by Tennessee Department of Safety (TDS) personnel to visitors at driver licensing centers in each of the three survey cities. The Department of Safety personnel distributed survey forms to motorists waiting to renew or, in some cases, to obtain their Tennessee driver's licenses. The TDS personnel, who had been trained to administer the survey, instructed the participants on completing the forms and collected completed questionnaires as the motorists departed. All participants were warned not to collaborate in their responses.

The survey was also given to a limited number of staff members at the University of Tennessee in Knoxville. No faculty members were surveyed.

Sample Size and Characteristics

A total of 176 motorists were sampled from the general driver population. This number was dictated by the time and funding constraints of the study; however, this sample size was adequate to accomplish the objectives of the research and assure reliability of the results. Participants were asked to complete a driver information form that gathered data on each individual subject's age, sex, education level, driver license status (i.e., in state, out of state, both, or none), and annual driving mileage.

The sample included drivers from a variety of socioeducational classes and therefore covers the entire driver population range. However, it should be noted that for some unexplained reason, the sample underrepresented older drivers. This fact does not invalidate the survey results, but it should be recognized that the survey may not accurately represent the comprehension level and conceptions of the older driver population.

Data Reduction and Analysis

The questionnaire and driver data were analyzed by using the Statistical Analysis System (SAS) battery of computer programs. As part of the data evaluation, several comparisons and contrasts were made. The validity and significance of these comparisons and contrasts were tested by using appropriate statistical tests, including chi-square tests, tests of proportions, and the Wilcoxon Sign Rank Test.

One question required subjects to give a short answer or narrative response. Subjects' responses to this question were reduced manually. Responses were paraphrased and grouped as appropriate for the sake of clarity and uniformity.

Police Survey

In reducing the survey data, it was noted that the sample by chance included two police officers and that the officers' responses appeared to be very different from the responses as a whole. This finding raised questions about the general comprehension level and conceptions of the law enforcement community concerning grade crossing traffic control and associated traffic regulations and prompted a second, smaller survey of 35 Tennessee police officers. The purpose of this second survey was to evaluate how police officers as a group perceive the intended meaning and application of the various grade crossing traffic control devices and regulations. The survey also permitted a comparison of the comprehension level and conceptions of the law enforcement community versus those of the general driver population.

The police survey was conducted in Knoxville, and the survey sample was composed entirely of Knoxville City police officers. The same questionnaire and driver information forms were used for the police survey. The police subjects were predominantly males between the ages of 25 and 44 with some college education. In addition, because of their occupation, the police officers all held Tennessee driver's licenses and were high-mileage drivers.

SURVEY RESULTS

Instruction and Training

In the survey, drivers were asked where they had received instructions or training, if any, on crossing safety. As presented in Table 1, 72.3 percent of the survey participants said that they received instructions on crossing safety from a driver handbook, presumably the *Tennessee Driver's Handbook* in most cases. This large percentage is not surprising because the *Tennessee Driver's Handbook* in fact presents one page of general information and instructions on crossing traffic control devices and traffic regulations.

TABLE 1 SOURCES OF INSTRUCTION AND TRAINING ON CROSSING SAFETY

Source	Percent of Drivers[a] (N = 166)
Driver handbook	72.3
Driver education course	34.8
TV, radio, or newspaper safety campaign	12.7
None (no prior instruction)	11.4

[a]The percentages do not sum to 100 percent because individual drivers could list several sources of instruction or training.

Only about one-third of the survey participants (34.8 percent) said that they received instructions, training, or both on crossing safety during a driver education course. This relatively low percentage suggests that many current courses are not devoting enough attention to crossing safety or that many licensed drivers simply have not had a formal driver education course. This finding is consistent with the fact that the young drivers (18 and below) performed poorly on most of the comprehension questions on the survey in comparison to older, more experienced drivers. The implication is that young drivers are not getting the training on crossing safety that they need and that knowledge on crossing safety is gained through experience that comes after drivers are licensed.

Table 1 also shows that 12.7 percent of the survey participants recalled receiving information on crossing safety through media safety campaigns, for example, Operation Lifesaver. The percentage is both encouraging and discouraging. On the positive side, crossing safety campaigns do appear to be reaching some motorists. However, the relatively low percentage of survey responses suggests a need to expand or improve these campaigns.

It is also significant to note that 11.4 percent of the drivers in the survey (19 of 166 drivers) could not recall ever receiving any instructions or training on crossing safety. This number, combined with the relatively poor showing of driver education courses and safety campaigns, indicates a general deficiency in crossing safety instruction and training for the driving public.

Grade Crossing Signing

The survey evaluated motorists' comprehension of two signs commonly used at or near grade crossings: the railroad crossing (crossbuck) sign and the railroad advance warning sign (*6*). With respect to the crossbuck sign, 76.3 percent of the survey participants correctly identified this sign as the one placed at the crossing. However, 19.0 percent of the drivers incorrectly identified the railroad advance warning sign as the one placed at a crossing. The implication is that some motorists do not associate the crossbuck sign with the actual point of hazard. Also, there is some confusion on the part of motorists between the crossbuck and advance railroad warning signs as to their meaning.

For the railroad advance warning sign, the survey addressed two questions: (a) do drivers recognize this sign as an advance crossing warning sign? and (b) what do drivers believe the sign means? Table 2 summarizes the drivers' responses to the first question on sign recognition. From Table 2, 63.6 percent of the survey participants (110 of 173 drivers) identified the railroad advance warning sign as the one placed before a crossing to give advance warning of the crossing location. This percentage is relatively low and may suggest that many drivers are not as familiar with the advanced warning sign and its application as they should be.

A significant precentage of drivers (16.6 percent) incorrectly identified a diamond shape sign with the message "RAILROAD CROSSING" as the appropriate advance warning sign for a crossing. This choice, although incorrect, is consistent with other types of warning signs and therefore its selection is not surprising. What is surprising is that 13.3 percent of the survey participants incorrectly identified the crossbuck sign as the sign that is used several hundred feet in advance of a crossing. This result again suggests that some drivers do not understand the full intent of the crossbuck sign and that there is motorist confusion between the crossbuck and railroad advance warning signs.

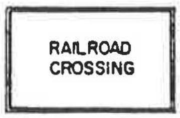
In addition to the recognition issue, the survey evaluated drivers' understanding of the intended (specific) meaning of the railroad advance warning sign. As presented in Table 3, only 8.8 percent of the subjects (15 of 171 drivers) gave the correct response, that is, there is a crossing ahead. Most survey participants (82.5 percent) said the sign meant to slow down to 20 mph because a crossing was ahead. This response is incorrect and undesirable from the standpoint of safety and roadway capacity. That is, if some motorists slow down to 20 mph at an unoccupied passive crossing on a high-speed roadway while others do not, the potential for rear-end accidents is high and traffic flow is interrupted.

Table 3 also shows that 3.5 percent of the subjects (six drivers) believed that the railroad advance warning sign indicates that there are signals ahead at the crossing, and 3.5 percent believed that the sign meant that a stop was required. Both of these are incorrect and totally undesirable with regard to crossing safety.

Passive Crossings

Survey participants were asked what they should do when approaching a crossing that does not have flashing light signals.

TABLE 2 RESPONSES TO THE SURVEY QUESTION "WHICH OF THE FOLLOWING IS USUALLY LOCATED SEVERAL HUNDRED FEET IN ADVANCE OF A RAILROAD CROSSING?"

Responses	Percentage of Subjects N = 173
	0.6
	16.6
	13.3
	63.6[a]
None of them	1.7
Don't know	4.0

[a]Correct response

TABLE 3 RESPONSES TO THE SURVEY QUESTION "WHAT DOES THIS SIGN [RAILROAD ADVANCE WARNING SIGN] MEAN?"

Responses	Percent of Subjects (*N* = 171)
Slow down to 20 mph due to crossing ahead	82.5
There is a crossing ahead	8.8[a]
There are signals ahead at the crossing	3.5
You will have to stop at the crossing	3.5
Don't know	1.7
Total	100.0

[a]Most correct response.

As presented in Table 4, only 24.3 percent of the subjects (36 of 148 drivers) gave the correct response, i.e., be ready to stop if you see or hear a train. Most subjects (69.6 percent) said that at passive crossings, one should stop, look, and listen for a train. These motorists perhaps were remembering the old grade crossing safety slogan, which did instruct motorists to stop at crossings. However, state traffic laws (*4*) do not require motorists to stop or slow down at a passive crossing unless a train is in hazardous proximity to the crossing, and few motorists do in fact stop at unoccupied passive crossings. Thus the incorrect responses indicate that many drivers are uncertain or misinformed about their responsibilities and required actions at passive crossings. If so, there is a need for better driver training and education.

TABLE 4 RESPONSES TO THE SURVEY QUESTION "WHAT SHOULD YOU DO WHEN APPROACHING A CROSSING THAT DOES NOT HAVE A RAILROAD SIGNAL?"

Responses	Percent of Subjects (*N* = 148)
Not applicable, because all crossings have railroads signals	6.1
Be ready to stop if you see or hear a train	24.3[a]
Speed up and cross the tracks quickly to avoid an accident	–
Stop, look, and listen at the crossing for a train	69.6
Don't know	–
Total	100.0

[a]Most correct response.

Also, 6.1 percent of the subjects (nine drivers) said that the question was not applicable because all crossings had flashing lights (Table 4). This response suggests that those motorists are completely naive to the passive traffic control strategy, that their traffic control expectancies at crossings are incorrect, or both. In any case, crossing safety would be jeopardized.

Flashing Light Signals

State laws (*4*) require motorists to stop at crossings with flashing light signals when the signals are activated; however, after stopping, motorists may proceed across the tracks if a train is not at the crossing or so near as to create a hazard. As part of the survey, motorist comprehension of flashing light signals at crossings was evaluated. Specifically, survey participants were asked what they should do upon seeing a railroad signal flashing.

As may be seen Table 5, 22.5 percent of the subjects (39 of 173 drivers) said that they should stop and then may proceed over the tracks if a train is not near. Most drivers (74.0 percent) said that they should stop and wait until the flashing lights go off before crossing the tracks. These two response groups together account for 96.5 percent of the drivers, and this high percentage indicates that most drivers understand they must stop in response to flashing light signals. However, most drivers are confused about their responsibilities and required actions after they stop.

The survey results indicate that at least some drivers believe that they must remain stopped at a crossing even when a train is near, whereas other drivers believe they should cross the tracks.

TABLE 5 RESPONSES TO THE SURVEY QUESTION "WHAT SHOULD YOU DO WHEN YOU SEE THIS RAILROAD SIGNAL FLASHING?"

Responses	Percent of Subjects (N = 173)
Take any action you think appropriate, because the signal is only advisory	2.3
Stop your vehicle only if you are driving a truck	—
Stop your vehicle and wait until the flashing lights go off, then proceed over the crossing	74.0
Stop your vehicle and proceed over the crossing if a train is not near	22.5[a]
Don't know	1.2
Total	100.0

[a]Most correct response.

TABLE 6 RESPONSES TO THE SURVEY QUESTION "WHAT SHOULD YOU DO WHEN YOU SEE GATES ARE DOWN AT A CROSSING?"

Responses	Percent of Subjects (N = 173)
Stop and remain stopped until the gate arms are raised	94.2[a]
Stop and then proceed around the gates if no train is coming	5.2
Slow down and then proceed around the gates if no train is coming	0.6
Any of the above	—
Don't know	—
Total	100.0

[a]Most correct response.

This may result in safety and operational problems, for example, at crossings where trains frequently stop in advance of the crossing with the signal lights flashing. At these crossings, some motorists believe they must remain stopped since the lights are flashing. Other motorists see no need to remain stopped, and they may make drastic maneuvers to get around stopped vehicles in front of them.

Four drivers (2.3 percent) said that flashing lights were advisory and therefore no stop would be required (Table 5). This response is totally undesirable and indicates a serious comprehension or attitude deficiency on the part of the respondents.

In the survey, drivers were also asked if flashing light signals appear at all crossings. This question was prompted by the research of Sanders et al. (2), who found that 15 percent of drivers thought that all crossings had some type of active traffic control. Like the Sanders study, the present survey revealed that some drivers apparently had misconceptions about the use of flashing signals and other active devices. In the present survey, 21.7 percent of the participants (37 of 171 drivers) believed that flashing light signals appear at all crossings, and another 1.2 percent (2 drivers) said they did not know if they did. These numbers are very alarming and suggest that some drivers have false expectancies about crossing traffic control, or they do not fully comprehend the passive traffic control strategy.

The previous research by Sanders et al. (2) also prompted an evaluation of drivers' perceptions of the elapsed time between signal activation and train arrival. In the survey, 22.5 percent of the drivers said that the elapsed time was always more than 1 minute. This percentage corresponds closely to the findings of the Sanders study, and it suggests that elapsed times between signal activation and train arrival tend to be very long, at least in the minds of drivers.

Crossing Gates

From Table 6, it can be seen that 94.2 percent of the participants (162 of 172 drivers) said that traffic should stop and remain stopped when the gates are lowered at a crossing. This response is consistent with state traffic laws (*4*), which do require all traffic to stop at a crossing when the gates are down and remain stopped until the gates are raised. The high percentage of "correct" responses indicates that most drivers do in fact understand the legal intent of gate arms.

However, Table 6 also reveals that 5.2 percent of the survey participants (nine drivers) said that traffic should drive around lowered gates if no train is coming, and one driver (0.6 percent) said it is not even necessary to come to a complete stop before going around the gates. It is not known if these drivers were aware of the law or whether they felt the law should be disobeyed under the circumstances. In either case, there apparently is a segment of the driver population that believes that it is all right to violate gate arms, and on the basis of field observations (*7*), these drivers and probably many more "follow-the-leader" drivers do in fact violate lowered gate arms.

Advance Railroad Pavement Markings

Drivers were asked which one of a group of pavement marking patterns was used in advance of some railroad crossings to warn approaching motorists. Over 70 percent of the survey participants (106 of 148 drivers) correctly identified the standard railroad crossing pavement markings (*6*) from the available choices. However, 15 drivers (10.1 percent) said that none of the given patterns were used, and 18 drivers (12.2 percent) answered that they did not know. The remaining 6.0 percent identified an incorrect pattern. These percentages indicate that many drivers are still unfamiliar with the "standard" markings.

Improvements or Remedies

Survey participants were given the opportunity to suggest improvements which they thought would enhance crossing safety. As can be seen from Table 7, 17.6 percent of the participants suggested that gates be installed at more crossings, and 7.4 suggested that flashing lights be installed.

Most of the motorists who suggested one of these improvements said that the improvements should be installed at all crossings. Four percent of the survey participants (seven drivers) suggested more grade separations.

Train Operations

The responsibility for negotiating a crossing safely rests primarily (if not entirely) on the driver; however, there is some question as to how much of this responsibility is recognized

TABLE 7 DRIVER SUGGESTIONS TO IMPROVE CROSSING SAFETY

Suggested Improvements	Number of Subjects	Percent of Total Subjects[a] (N = 176)
Add gates/gates with flashing lights	31	17.6
Add flashing lights	13	7.4
Install grade separations	7	4.0
Reduce roughness of crossing	4	2.3
Reduce gate/signal malfunctions	4	2.3
Improve signs/markings	4	2.3
Reduce train speeds	3	1.7
Install full-width gates	3	1.7
Eliminate crossings	4	1.7
Improve driver education/training	3	1.7
Improve sight distance down track	2	1.1
Install speed bumps	2	1.1
Make crossing bells louder	2	1.1
Require traffic to slow/stop at all crossings	2	1.1

[a]Percentage does not total 100 percent because many subjects did not suggest any improvements, whereas others suggested one or more.

and accepted by drivers. In an attempt to answer this question, the survey explored drivers' perceptions of train capabilities, as well as drivers' perceptions about the duties of the train operator.

Drivers were first asked to compare the stopping distance of a train to that of a large truck. Surprisingly, over 7 percent of the survey participants (12 of 169 drivers) said that a train can stop in the same or less distance than a truck. Another 11.2 percent were uncertain which vehicle could stop quicker, i.e., a train or large truck. Combining these two groups, over 18 percent of the survey sample (31 of 169 motorists) did not know that the stopping distance of a train was much greater than that of a truck or car. This relatively high percentage suggests that some drivers believe that a train could stop or slow down significantly if necessary to avoid a collision.

Participants were also asked what they thought a train operator should do if he or she saw cars crossing the tracks in advance of the train. The responses to this question suggest that some drivers, if not many, fail to recognize and accept total responsibility for their safety at grade crossings. For example, 27.2 percent of the survey participants said that the train operator should slow the train, while 17.7 percent said that the train operator should stop the train. Another 17.7 percent of the drivers said the operator should flash the train's headlight. All of these responses suggest that drivers place some responsibility for crossing safety on the train operator, or at least that they would like to. As further support of this, less than one fourth of the survey participants (21.8 percent) said that there was nothing a train operator could or should do if cars cross in front of the train.

Also in regard to train operations, survey participants were asked when (at which crossings) the train operator sounds the train's whistle. Over three fourths of the drivers (78.2 percent) said that they thought the whistle was sounded in advance of every crossing, 7.5 percent thought it was sounded only for hazardous crossings, and 11.6 percent did not know when it was sounded. The most significant finding is that a large majority of drivers expect a whistle at all crossings, yet research (*1*) has determined that train whistles often cannot be heard inside modern closed motor vehicles. Whether or not motorists also expect to hear the whistle that is sounded is not known.

Driver Variable Effects

As part of the research, the effects of survey location and participant's sex, age, education, license status, and driving experience were evaluated. On the basis of the evaluation, some general trends were identified, while other possible effects were discounted. The statistical reliability of these effects were established or discounted as the case may be by using chi-square tests and tests of proportions.

Survey Location

No significant differences were found in the data from the driver licensing centers in Nashville, Chattanooga, and Knoxville. The data from the university sample, however, differed from the data gathered from the three driver licensing centers. Generally speaking, the university personnel demonstrated a higher comprehension level. The higher correct response ratio is probably due to the generally higher level of education inherent to a college campus.

Sex

No significant differences were found in the responses of male versus female drivers.

Age

Very young drivers (18 years and below) and older drivers (above 54) tended to have more trouble understanding and recognizing traffic control devices. For example, these driver groups performed relatively poorly on the questions that dealt with recognition of the crossbuck and advance railroad warning signs and that asked the meaning of the advance railroad warning sign. The very young drivers also performed poorly on the experience-related questions. Most notably, 62.5 percent (versus 21.6 percent for all drivers) of the drivers 18 years of age and below believed that all crossings had flashing lights.

Education

Drivers with less than a high school education demonstrated a relatively poor comprehension of grade crossing traffic control devices. However, it should be noted that many of the drivers in this low education group were also young, newly licensed drivers. It is believed that the three factors of education, age, and driving experience together affect comprehension.

License Status

No differences were found between Tennessee and out-of-state drivers. Significant differences were again found, however, between newly licensed and experienced drivers, probably due to the combined effects of age, education level, and driving experience.

Driving Experience

Drivers who drove more than 20,000 miles per year tended to do better in all aspects of the survey. Drivers who drove fewer that 5,000 miles per year tended to do worse than drivers as a whole. It should be noted that most of the very low mileage participants were young, newly licensed drivers, and the combined effects of age, education, and driving experience probably account for their relatively poor performance.

Police Survey

The police officers, as a group, performed generally better on the survey than the general driver sample. That is, the officers in most cases demonstrated a somewhat better understanding and recognition of the traffic control devices and regulations applicable to railroad crossings. The officers' compared performance on the survey is illustrated in Table 8. The table compares the percentages of correct responses on 11 individual survey questions for the two groups, that is, the police officer sample versus the general driver sample. The 11 questions were selected as a basis for comparison because each of the questions had a "more correct response" and they all dealt with driver comprehension of crossing traffic control devices, regulations, or both. The police officers ranked higher than the general driver group on 9 of the 11 discriminating questions (Table 8). If is assumed that each question had an equal weighting on comprehension and if the Wilcoxon Signed Rank Test is used, the overall comprehension difference in the two groups is significant at the 99 percent confidence level. It can also be seen in Table 8 that the responses by the police officers on several of the individual questions were statistically significant on the basis of a one-tailed test of proportions assuming a 95-percent individual confidence interval is assumed.

It is not surprising that the police officers did better than the general public on the survey. First of all, the police officers indicated that they had more training and instruction on crossing safety compared to drivers as a whole. For example, 57.1 percent of the officers said that they had received training or instruction from at least two sources, while only 18.1 percent of the general drivers said that they had received training on crossing safety from multiple sources. It is assumed that the officer's additional training and exposure to traffic laws comes in connection with their general job training. Also, the police officers were predominantly males between the ages of 25 and 44, with some college education and extensive driving experience.

It should be noted that the police officers had a lower percentage of correct responses than the general public on only two questions (Table 8). One question asked what a driver should do upon seeing an activated flashing signal. A disproportionate percentage of the officers (85.7 percent) said that a driver should stop at the crossing and remain stopped until the flashing lights go off. The other question asked what a driver should do at a passive crossing. In response to this question, a very high percentage of the officers (76.5 percent) said that a driver should stop, look, and listen for a train. The officers' responses to both questions are surprising because they are inconsistent with state laws. Apparently the officers answered the questions from a very conservative, safety-conscious viewpoint.

Police Survey Summary

While the police officers responded more accurately in the survey than did the general drivers, the officers' comprehension level was still lower than desirable. As shown in Table 8, no single question was answered correctly by all of the police officers, and in only 6 of the 11 questions did more than 90 percent of the police officers respond correctly. In addition, on three of the questions, fewer than 25 percent of the police officers provided a correct response, and only 14.3 percent responded correctly to one of the questions. These results demonstrate that there is a substantial lack of understanding by the police officers of traffic control devices used at crossings, as well as of regulations regarding those devices.

TABLE 8 COMPARISON BETWEEN THE POLICE OFFICERS' AND GENERAL DRIVERS' COMPREHENSION LEVELS

Question	Percent of Correct Responses: Police	Percent of Correct Responses: General Drivers	Differences Significant at 95% Confidence Level[a]
Which sign is usually placed at the point where the railroad tracks cross the highway?	94.3	76.3	Yes
Which sign is usually located several hundred feet in advance of a railroad crossing?	71.4	63.6	No
What should you do when you see a railroad signal flashing?	14.3	22.5	No
Does a flashing signal appear at all crossings?	91.2	77.2	Yes
In general, how does the distance needed to stop a train compare with that needed to stop a large truck traveling at the same speed?	91.4	81.7	Yes
What does the Advance Railroad Warning sign mean?	91.4	82.5	Yes
What should you do when the gates are down at a crossing?	97.1	94.2	No
Which standard markings are painted on the pavement in advance of some railroad crossings?	88.2	71.6	Yes
What should you do when approaching a crossing that does not have a railroad signal?	20.6	24.3	No
When does a train operator sound the train's whistle?	94.1	78.2	Yes
What should a train engineer do if he sees cars crossing the tracks in advance of his train?	24.2	21.8	Yes

[a]Based on one-tailed test of proportions.

CONCLUSIONS AND RECOMMENDATIONS

Results from the survey indicate that there are substantial problems with the level of knowledge among drivers about traffic control devices used at crossings, as well as of the regulations that govern those traffic control devices. The percentage of the general drivers who gave correct responses to the questions on the survey was often low. The police officers as a group generally performed better than the general drivers; nevertheless, this group did not achieve the desired results. In certain categories, the percentage of police officers providing correct responses was very low and represented severe lack of knowledge.

Any driver who is confused or has a lack of understanding about how to respond to traffic control devices can cause significant safety problems leading to accidents with personal injuries and fatalities. This problem can be even more pronounced at highway-railroad grade crossings because, in general, the total responsibility for avoiding a collision with a train is placed on the driver of the motor vehicle. Thus if one motor vehicle operator performs unacceptably due to a lack of knowledge, serious consequences can result. Even a small fraction of the driving population performing in an unacceptable manner at crossings can cause the number of accidents occurring at crossings nationwide to increase. The total population does not have to be driving inappropriately.

Operating a motor vehicle inappropriately at a crossing due to a lack of knowledge cannot be construed as willful disregard for safety by a motorist. Often drivers involved in collisions with trains are assumed to have been careless, inattentive, or simply negligent in the operation of their motor vehicles. If this survey represents the general driving population, then one might well argue that at least some of the inappropriate and unsafe operation of motor vehicles at crossings can be due to a lack of understanding or knowledge of how to operate the motor vehicles.

It would be difficult, if not impossible, for all drivers to achieve a complete understanding of traffic control devices used at crossings and the regulations governing these devices. However, there is substantial room for improvement. In some programmatic areas there is a need to address more fully the area of motor vehicle operation at railroad-highway grade crossings. Specifically, increased attention should be given in the following areas:

- State highway and transportation departments should initiate a program with state departments of education to include appropriate training on railroad-highway grade crossings in the high school driver education curriculum. State departments of education normally have a strong influence on the program areas for high school students. State highway and transportation departments should develop a module of training that would be supported by the state departments of education for use in high schools in the driver education programs.
- State highway and transportation departments should work with the state agency responsible for developing driver licensing handbooks to include a sufficient amount of material on railroad-highway grade crossings, traffic control devices for crossings, and regulations pertaining to them. Many state highway and transportation departments already are very involved with developing their states' handbooks; these departments should assess whether the level of coverage of grade crossing issues is adequate.
- State highway and transportation departments should work with the state agency that has responsibility for driver licensing to include items regarding railroad-highway grade crossings on written examinations given to applicants. Although some states are using questions on railroad-highway grade crossings, there is a need for expanded activity in this area.
- Public service activities such as Operation Lifesaver should address more issues involving traffic control devices at crossings as well as the regulations regarding those. These public service announcements should be consistent with state laws governing the operation of a motor vehicle at a crossing.
- Operation Lifesaver and other educational programs should devote more effort to informing and educating the law enforcement community on the meaning and intent of grade crossing traffic laws and traffic control devices. Also, more attention should be given to proper enforcement of traffic laws and regulations at grade crossings, since uniform enforcement will promote driver understanding and obedience.
- Additional survey work should be conducted throughout the United States to determine whether the survey conducted in Tennessee is unique or is perhaps a reasonable representation of the population. If the general population of the United States has the same level of understanding as found in this survey, then there needs to be immediate attention taken to increase the level of knowledge of the driving public of traffic control devices used at crossings and the regulations that govern them.

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CALSIG: An Integration of Methodologies for the Design and Analysis of Signalized Intersections

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A proposed procedure for the design and analysis of signalized intersections is described. The procedure is an integration of existing methodologies, with each methodology incorporating a separate capacity and/or level-of-service analysis. The result is a procedure that is modular in form and possesses several levels of analysis. The proposed procedure can be discontinued immediately after each level of analysis. The procedure also incorporates design elements that can generate designs for any intersection parameter that is not established a priori. The procedure's design elements can also aid the user in identifying operational deficiencies and implementing design improvements.

In this paper, a proposed procedure for the design and analysis of signalized intersections is described. This procedure was developed as part of a research project conducted at the Institute of Transportation Studies, University of California, Berkeley. The research project, Calibration and Validation of the 1985 Highway Capacity Manual for California Conditions, was sponsored by the California Department of Transportation and the Federal Highway Administration.

The initial phase of this research consisted of a comprehensive review of the 1985 *Highway Capacity Manual* (HCM) (*1*). The objectives of this effort were to identify and document any areas of the 1985 manual that might not be completely applicable to traffic conditions observed in the state of California.

From this comprehensive review, it was concluded that additional enhancements could be incorporated into the HCM's existing analysis procedures for signalized intersections (*1*, Chapter 9). The reviewers believed that these enhancements might make the existing Chapter 9 procedures more attractive to transportation professionals in California and elsewhere. The enhancements that were added to the existing Chapter 9 procedures resulted in a modular analysis and design methodology, hereafter referred to as CALSIG.

The following section of this paper provides a general overview of the CALSIG procedure. In later sections, CALSIG's analytical framework and details are described, and the CALSIG procedure is discussed in greater detail. Following this explanatory material is an annotated sample problem. Conclusions are discussed in the final section of the paper.

The purpose of this paper is to acquaint the reader with the CALSIG procedure. CALSIG's structure and procedural application are described here in some detail. However, this paper is not intended to serve as a user's manual. Additional information may be required to apply the CALSIG methodology to all scenarios. The complete CALSIG user's manual (*2*) can be obtained from the Institute of Transportation Studies, University of California, Berkeley, Calif. 94720.

Institute of Transportation Studies, University of California, Berkeley, Calif. 94720.

PROCEDURAL OVERVIEW

Before the new procedure was developed, a careful evaluation was performed on all currently available design and analysis techniques for signalized intersections. This evaluation identified both positive and negative features within these existing procedures. CALSIG, the proposed procedure, was subsequently formulated to incorporate desirable features from existing methodologies. CALSIG is in fact an integration of currently available analysis procedures.

A separate analysis is associated with each integrated procedure. Thus CALSIG incorporates several levels of analysis. Each level requires an increasing degree of procedural detail.

Such a framework offers two significant advantages to the user:

- Because CALSIG contains increasing levels of analysis, a signalized intersection analysis may be performed with only the amount of detail appropriate for the specific application.
- Each of CALSIG's analysis levels yields a separate, but dependent, level-of-service (LOS) or operational prediction. By examining and comparing the results yielded by each level of analysis, the user can readily determine which, if any, signalized intersection parameters are contributing to predicted operational problems.

In addition, CALSIG incorporates both analysis and design procedures. CALSIG's design features consist of standards and guidelines typically used by California transportation professionals. A number of these guidelines are taken directly from the 1985 HCM's Chapter 9 appendices.

CALSIG's design features can aid the user in establishing parameters (e.g., approach geometrics, signal timing, etc.) when a signalized intersection is in the planning stages. CALSIG's design criteria can also be used effectively to assist the analyst in establishing operational improvements.

It should be noted that CALSIG is essentially a manual procedure for designing and analyzing signalized intersections. However, a substantial amount of time and effort is required to perform an entire CALSIG procedure manually. For this reason, a user-friendly computerized version of CALSIG was developed as part of the original research project. This software package dramatically reduces the time required to perform CALSIG computations.

ANALYTICAL STRUCTURE

This section describes the basic structure of the CALSIG procedure. Figure 1 is a procedural flow chart that schematically illustrates CALSIG's framework. It should be noted that CALSIG incorporates seven steps or modules and four analysis techniques. The seven modules are

1. Turning movement demand,
2. Approach geometrics,
3. Phase design (phase types and phase sequences),
4. Lane group saturation flow rates,
5. Minimum required green times,
6. Cycle length, and
7. Phase lengths.

CALSIG's four analysis techniques are

1. A *preliminary analysis* that is based on the critical movement analysis and predicts the performance of the overall intersection. The procedure is essentially identical to the 1985 HCM planning analysis.
2. An *intermediate analysis* that evaluates the critical flow ratio, V/S_{crit}, for each signal phase and estimates an aggregate intersection level of service. The procedure is essentially identical to the intersection capacity utilization technique developed by Crommelin (*3*).
3. A *comprehensive analysis* that utilizes the lane group V/c ratios as the primary measure of effectiveness (MOE). This analysis will predict the level of service for each lane group, each approach, and the intersection as a whole.
4. A *comprehensive analysis* that utilizes average stopped delay per vehicle as the primary MOE and is essentially the 1985 HCM operational analysis.

Multilevel Analysis

There is nothing unusual about CALSIG's modules or analysis techniques. In fact, all of the parameters used in CALSIG are accounted for when the existing HCM procedures are used. What is unique about CALSIG's structure is that not all parameters listed (and illustrated in Figure 1) need to be incorporated when performing the CALSIG procedure.

As shown in Figure 1, the procedure can be concluded immediately after each level of analysis. The analysis level at which the procedure is concluded will depend on the degree of detail needed for the specific application. This degree of detail may vary considerably, depending on the specific scenario and user preference. The modules required to perform each of CALSIG's analysis techniques are those modules that precede each analysis in Figure 1. Thus the CALSIG procedure can be performed with varying amounts of comprehensiveness.

At either extreme, the CALSIG procedure parallels the 1985 HCM planning and operational analysis. Thus in some sense the CALSIG methodology represents the existing HCM procedures with additional features incorporated. These additional features do not necessarily make the CALSIG procedure less complex than the HCM's existing operational procedure. In fact, the inclusion of additional analyses may often make the CALSIG procedure more time consuming—although use of the CALSIG microcomputer software will significantly reduce this additional time.

There are scenarios for which use of the existing HCM operational procedure is preferable. The more straightforward procedures of the HCM operational method are most appropriate when an operational analysis is to be performed and average stopped delay is the desired measure of effectiveness.

Nonetheless, CALSIG's multilevel analysis format does offer distinct advantages for many applications. CALSIG's enhancements, in some sense, bridge the gap that currently exists between the simplicity of the HCM planning method and the complexity of the operational analysis. This provides additional flexibility for the CALSIG user.

The results given by each of CALSIG's analyses will not necessarily be compatible when applied to identical scenarios. For example, a scenario analyzed by the CALSIG intermediate analysis may yield a LOS C, but the same scenario may yield a LOS D, or lower, when the CALSIG comprehensive analysis is used. This is not necessarily a deficiency in the CALSIG methodology. Such inconsistencies can be used to determine which parameters are creating predicted operational problems. For example, a comprehensive analysis that yields a lower level of service than the preceding intermediate analysis indicates that improved signal timing should be considered (refer to Figure 1). Conversely, a comprehensive analysis that yields a higher level of service than the preceding intermediate analysis indicates that optimal signal timing has improved an otherwise problematic operation.

There is a particular inconsistency in using V/c rather than average stopped delay as the MOE. Longer cycle lengths generally reduce V/c and increase delay. Conversely, shorter cycle lengths reduce delay and increase V/c values. To deal effectively with this inconsistency, the user should establish, a priori, which measure (V/c or delay) is to be considered as the primary MOE.

Each level of analysis within the CALSIG procedure yields a different measure of effectiveness. This can be confusing at times. In addition, direct comparisons of MOEs from different analysis levels are not possible. However, levels of service and general operational predictions can be compared at all analysis levels. Thus predicted operational quality can be compared, in general terms, at all levels of analysis. A more detailed description of CALSIG's multilevel analysis techniques is offered in the next section.

Design Elements

As stated in the previous section, CALSIG incorporates both design and analysis elements. The dashed vertical lines in Figure 1 delineate the design and analysis portions of the CALSIG procedure. Symbols to the left of the dashed lines denote design elements; symbols located to the right represent analysis elements. Any parameter that is to be included in the procedure but has not yet been established by the analyst can be designed by the CALSIG procedure. CALSIG's design criteria can also serve as guidelines for establishing design improvements. Any design generated by CALSIG can be readily modified on the basis of user judgment or local practice.

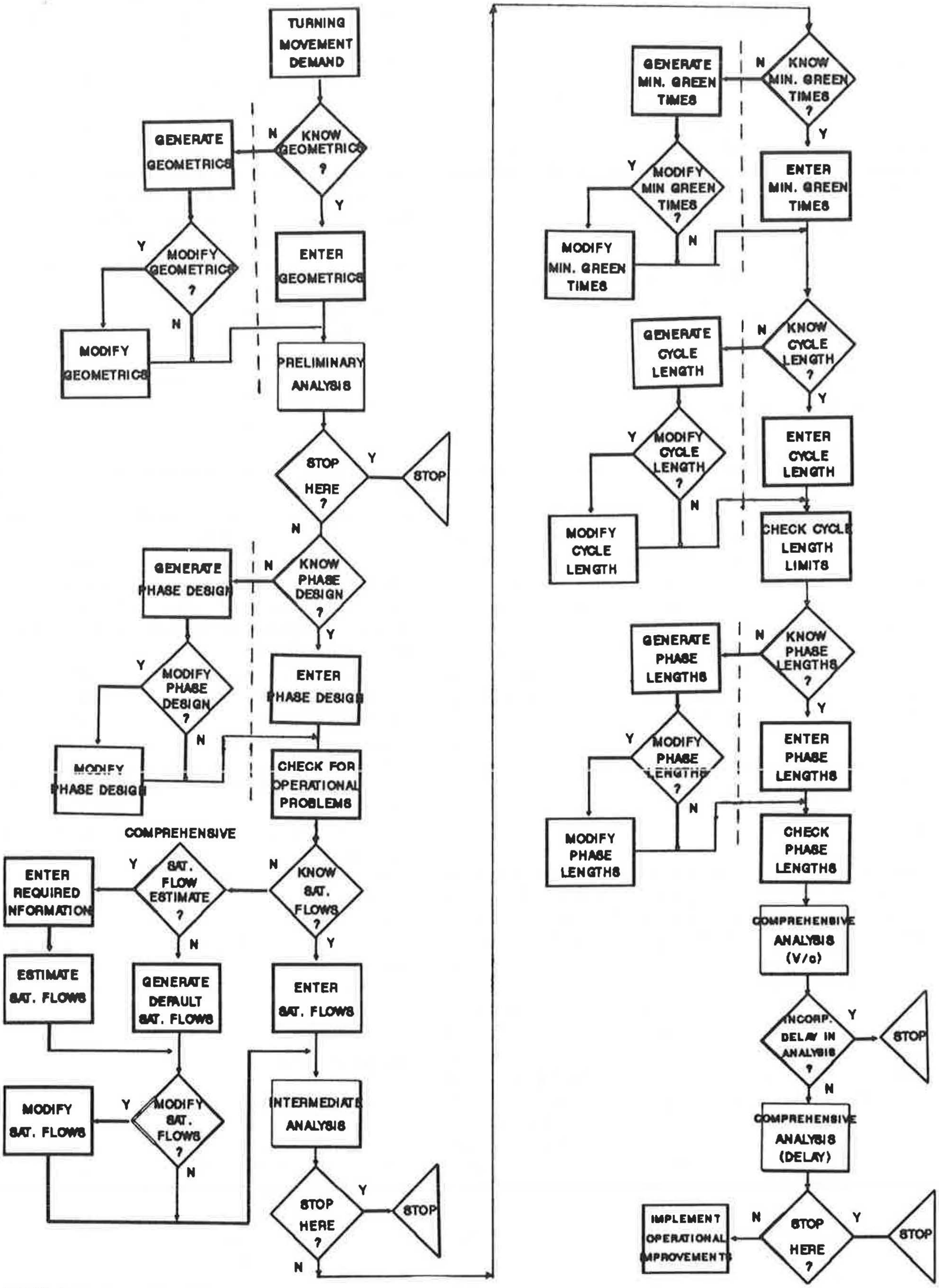

FIGURE 1 Procedural flow chart.

The CALSIG design elements are meant to provide the user with optimal (or near-optimal) design for unknown or poorly designed signalized intersection parameters. It is certainly true that microcomputer software packages have dramatically reduced the time and effort required to perform sensitivity analysis. Thus iterative computer-assisted evaluation can generally reveal optimal (or near-optimal) designs. CALSIG's design features merely simplify optimization. In addition, comparison of an existing design to CALSIG's design criteria can expedite identification of problem areas. CALSIG's design criteria are briefly highlighted in the following section.

ANALYTICAL DETAILS

In this section, the steps to follow when performing the CALSIG procedure are outlined. As stated in the introduction, the objective of this paper is to familiarize the reader with the CALSIG methodology. Additional information on the procedure can be found in the CALSIG user's manual (*1*).

The entire CALSIG procedure can be broken down into the steps or modules previously illustrated in Figure 1. Thus the procedure is documented in modular fashion in this section.

Turning Movement Demand Module

Volume, as defined in the 1985 HCM, is the total number of vehicles passing a point or section of a roadway during a specified time period. Rate of flow is an equivalent hourly rate at which vehicles pass a point or section of roadway for some period of time less than 1 hr.

When the CALSIG procedure is being performed, the traffic flows from each approach movement must first be established. The procedure can be performed with either 15-min rates of flow or full hourly volumes. The choice should be based on the specific design or analysis application and the degree of detail chosen to be incorporated in the procedure. For example, if a comprehensive analysis is to be performed on an existing intersection using existing traffic counts, peak rates of flow should typically be used. However, if the intersection is to be evaluated in a less comprehensive manner or if projected traffic volumes are to be used, full hourly volumes may be the most appropriate choice.

The choice of performing the procedure using either volumes or flow rates will require professional judgment. A more detailed explanation of peak rates of flow may be found in Chapter 1 of the 1985 HCM.

Geometrics Module

If approach geometrics exist or have been established by the user, the individual lane configurations are incorporated in the procedure. These geometrics will subsequently be considered in the preliminary analysis.

Design Procedure

If approach geometrics are not defined by existing conditions or by state or local practice, lane configurations may be determined on the basis of CALSIG's geometric design criteria. The design criteria may also be applied when analysis indicates intersection deficiencies that may be corrected by changes in geometric design. These design criteria are derived from Appendix I, Chapter 9, of the 1985 HCM.

The geometric design of an intersection involves several critical decisions about the number and use of lanes to be provided on each approach. The following material addresses these determinations.

Exclusive Left-Turn Lanes

The following guidelines govern the provision of exclusive left-turn lanes:

- Where fully protected left-turn phasing is to be provided, an exclusive left-turn lane should be used.
- Where left-turn volumes exceed 100 vph, an exclusive left-turn lane should be provided.
- Where left-turn volumes exceed 300 vph, provision of a double left-turn lane should be considered.

Exclusive Right-Turn Lanes

An exclusive right-turn lane should be considered when the right-turn volume exceeds 300 vph and the adjacent main-line volume also exceeds 300 vph per lane.

Number of Lanes

In general, enough main roadway lanes should be provided that the total of through plus right-turn volume (plus left-turn volume if present) does not exceed 450 vph per lane.

Preliminary Analysis

CALSIG's preliminary analysis incorporates the intersection turning movement demand and approach geometrics. The analysis, which is based on the critical movement technique, predicts the general operating performance of the overall intersection. CALSIG's preliminary analysis is identical to the 1985 HCM planning analysis.

- The given traffic demand is distributed over the available lanes as uniformly as flow conditions permit. The concept of uniform flow conditions is discussed in Chapter 9 of the 1985 HCM and is therefore not repeated here.
- The sum of the critical conflicting lane volumes (i.e., through plus opposing left-turn traffic) is computed for both intersecting streets.
- The sum of the critical conflicting lane volumes can be related to a corresponding capacity criteria. Capacity criteria values are taken directly from the 1985 HCM:

Critical Value for Intersection, vph	*Relationship to Probable Capacity*
0 to 1200	Under capacity
1201 to 1400	Near capacity
≥1401	Over capacity

- The preliminary analysis also results in a general statement that describes predicted vehicle queueing.

The results of the preliminary analysis give a general indication of the acceptability of the intersection's capacity. As its name suggests, the analysis is preliminary in nature. The evaluation is carried out entirely in mixed vehicles per hour. The existence of adequate signalization is assumed, as are average values for all prevailing conditions.

This analysis technique is a useful tool for evaluating the overall adequacy of an intersection or comparing alternative geometric design. In addition, this preliminary procedure may be adequate in itself for evaluating intersections by using predicted future volumes.

The CALSIG user has three options once the preliminary analysis has been completed:

- Discontinue the procedure and obtain the preliminary analysis results.
- Modify the turning movement demand, the approach geometrics, or both and repeat the analysis.
- Continue the CALSIG procedure so that a more comprehensive design or analysis will be performed.

The CALSIG user's manual (*2*) and the 1985 HCM provide more complete descriptions of the preliminary (planning) analysis.

Phase Design Module

If the user elects to continue the CALSIG procedure beyond the preliminary analysis, the number and the sequence of phases is the next parameter to be incorporated. If the phase plan is already established, it is used in the procedure. This phase plan will be accounted for in CALSIG's intermediate analysis.

If the phase design has not yet been determined, an appropriate phase number and sequence may be established by using CALSIG's design criteria. As a general rule, the number of phases used by a signal should be kept to a minimum, especially for pretimed controllers. As the number of phases increases, the green time available to other phases is reduced. Additional signal phases create added change intervals, which can increase cycle length and delay. Thus two-phase control should generally be used, unless turn volumes, safety considerations, or special intersection geometrics dictate the need for protective phasing.

Multiphase control is used at intersections that require a protected phase for one or more left or right turns. Local policy and practice most often determine the need for protected phasing. In the absence of local policy, however, the following design guidelines can be used for establishing phase plans.

Geometric Considerations:

- If more than one lane of an approach can turn left, a protected left-turn phase should be provided.
- If left-turn traffic must cross three or more lanes of opposing through traffic, a protected left-turn phase should be provided.

Volume Considerations.

- A protected left-turn phase should be provided if the approach left-turn volume is between 50 and 120 vph and the product of the left-turning and conflicting through volume exceeds 100,000.
- If an approach's left-turn volume is between 120 and 240 vph, a protected left-turn phase should be provided if

$$V_{LT} \times V_o' > 50{,}000$$

where

V_{LT} = approach left-turn volume,
V_o' = V_o (opposing volume) if one opposing lane exists,
= $0.55\ V_o$ if two opposing lanes exist, and
= $0.40\ V_o$ if three opposing lanes exist.

- If an approach left-turn volume exceeds 240 vph, a protected left-turn phase should be provided.

Additional Considerations:

- A protected left-turn phase should be provided if sight distance to opposing traffic is less than 250 ft when the opposing traffic is traveling at 40 mph or more.
- A protected left-turn phase should be provided if an approach left-turn volume exceeds 50 vph and through traffic speed exceeds 45 mph.

Phases that protect left turns can be provided in a variety of ways (e.g., leading protected lefts, protective-permissive, etc.). The CALSIG user's manual contains a more detailed discussion of phase plans.

Saturation Flow Rate Module

Saturation flow rate is defined as the maximum rate of flow that can pass through a given intersection approach or lane group under prevailing traffic and roadway conditions, given that effective green time was available to the approach or lane group 100 percent of the time. Saturation flow rates can be measured in the field by the user, comprehensively computed by using the saturation flow procedures of the 1985 HCM, or estimated by using CALSIG-recommended saturation flow rate default values.

The CALSIG procedure, like the operations procedure of the 1985 HCM, performs a signalized intersection capacity and LOS analysis based on saturation flow rates for each lane group. Thus, to continue with the CALSIG procedure, the intersection's individual lane groups must be determined.

The 1985 HCM defines a lane group as one or more lanes on an intersection approach serving one or more traffic movements. The designation of the individual lane groups within each approach considers both the geometry and the distribution of traffic movements. Guidelines for determining lane groups can be found in the CALSIG user's manual (*2*) and Chapter 9 of the 1985 HCM (*1*).

Once the lane groups have been established, the saturation flows for each lane group must be determined. Lane group saturation flow rates can be established in one of three ways.

Method 1

Field-measured saturation flow rates may be used. If a signalized intersection is to be evaluated under existing conditions, the analyst may wish to incorporate saturation flow values measured directly at the subject intersection. A procedure for directly measuring prevailing saturation flow rates can be found in Appendix IV, Chapter 9, of the 1985 HCM.

The most reliable results will generally be obtained by performing the CALSIG procedure with field-measured saturation flow rates. This method is therefore recommended when such data are readily available.

Method 2

Estimates of lane group saturation flow rates may be computed by the 1985 HCM (*1*) saturation flow rate equation. Computations begin with the selection of an "ideal" saturation flow rate. The term ideal is used here to reflect near-perfect operating conditions, such as wide lanes, no on-street parking, no grades, and so on. The 1985 HCM has selected 1,800 passenger automobiles per hour of green time per lane as a national average value that reflects ideal saturation flow rate. This ideal value is then adjusted to account for a variety of prevailing conditions that are not ideal.

The 1985 HCM saturation flow rate prediction computation is rather complex and is not reproduced in this paper. The CALSIG user's manual and Chapter 9 of the 1985 HCM contain complete discussions of this topic.

Method 3

The third and final option for establishing lane group saturation flow rates is to use default values. These default values may be used when existing data are not readily available or when the analyst feels that the complex saturation flow computations of the 1985 HCM are not warranted for a particular application. CALSIG-recommended default values assume average operating conditions; for example, 10- to 12-ft-wide lanes, 5 to 10 percent heavy vehicles in the traffic stream, no significant approach grades, and minor on-street parking turnovers and bus activity.

A value of 1,600 vehicles per hour of green time per lane (vphgpl) may be used for exclusive through lanes and shared right-turn/through lanes, and a value of 1,550 vphgpl may be used for exclusive left-turn lane traffic with protective phasing. A value of 1,500 vphgpl may be used for exclusive right-turn lane traffic with protected or permitted phasing.

The estimation of default saturation flow rate values for exclusive left-turn lane traffic with permitted phasing will depend on the number of total opposing vehicles. These opposing vehicles impede left-turn traffic. Figure 2 illustrates the relationship between the saturation flow rate of exclusive left-turn traffic, S (in vphgpl), with permitted phasing and the total opposing traffic flow, V_o (in vph). The curve in Figure 2 was constructed for multilane approaches by using the saturation flow equation (Equation 9-8) of the 1985 HCM. Thus Figure 2 may be used to estimate an appropriate saturation flow rate default value for exclusive left-turn lane traffic with permissive phasing.

The default values for unique lane groups with unusual phasing (for example, shared through/right-turn lane traffic with protected plus permitted phasing) may be estimated by using Tables 9–11 and 9–12 of the 1985 HCM. However, for such situations, it may be more appropriate to measure the saturation flow rates in the field or to estimate them comprehensively. If a lane group consists of a shared through/left-turn lane and has permitted phasing, field-measured or comprehensively estimated saturation flow rates should be used.

Intermediate Analysis

CALSIG's intermediate analysis incorporates all of the previously described intersection parameters (i.e., traffic demand, approach geometrics, phase design, and lane group saturation flow rates). This analysis procedure is identical to the "intersection capacity utilization" technique originally developed by Crommelin (*3*):

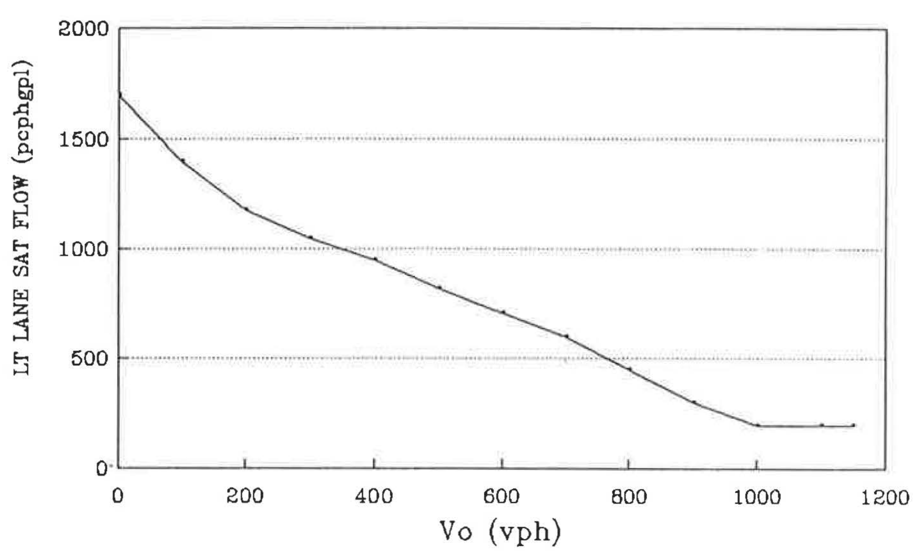

FIGURE 2 Shared left-turn lane (protected phasing) saturated flow rate versus opposing traffic.

- The flow ratio, V/S (volume/saturation flow rate), is computed for each lane group. Each lane group volume can be adjusted to reflect unequal lane distribution by incorporating the lane utilization factor, as explained in Chapter 9 of the 1985 HCM.
- Critical lane groups are then identified. A critical lane group is simply the lane group with the highest flow ratio in each phase.

If there are no overlapping signal phases (concurrent phase timing) in the signal design, there will be only one critical lane group for each signal phase.

Critical lane groups become a little more complicated to determine when overlapping phases exist. Combinations of lane groups that may consume the largest amount of available capacity must be identified on the basis of the phase plan. When a phase design includes an overlap, the critical lane groups for the subject phase sequences will be the highest sum of the through and opposing left-turn (if any) flow ratios. Examples of this technique are contained in the CALSIG user's manual and the 1985 HCM.

If optional phases exist within a phase plan, the analysis considers only the phase sequence that is most likely to occur during the time period that is being analyzed. Optional phases are controlled by the directions that have the heavier traffic flows.

- The flow ratio (V/S) values for each critical lane group are then summed.
- The sum of these critical lane group values can be modified to take into account the yellow and all-red clearance periods.

The equation that can be used to compute this clearance interval is as follows:

$$YA = (\Sigma\ V/S_{\text{crit}})\ (c/ny\ -\ 1)^{-1}$$

where

YA = clearance internal flow ratio;
$\Sigma\ V/S_{\text{crit}}$ = sum of the flow ratios for the critical lane groups;
c = established (or probable) cycle lengths (sec);
n = number of phases; and
y = average clearance interval (yellow and red-all) (sec).

The clearance interval flow ratio computed in this equation is simply added to the sum of the flow ratios for the critical lane groups. Incorporation of this clearance interval flow ratio into the analysis is optional. It should be noted that addition of a clearance interval flow ratio to the flow ratios of the critical lane group implies that longer cycle lengths (with fewer yellow periods) lead to improved operation. Such is not the case.

- The summed flow ratios of the critical lane groups can be related to a specific level of service designation. Table 1 lists flow ratio value ranges for level-of-service values A through F. These V/S values were originally developed by Crommelin (*3*) and are derived from the signalized intersection load factor diagrams in the 1965 HCM.

TABLE 1 V/S_{crit} AND LEVEL OF SERVICE (LOS)

V/S_{crit}	Level of Service
0.00–0.60	A
0.61–0.70	B
0.71–0.80	C
0.81–0.90	D
0.91–1.00	E
>1.00	F

If the summed flow ratios of the critical lane groups are greater than 1.0, extensive operational problems can be expected to occur at the intersection during the time period analyzed. If $\Sigma\ V/S_{\text{crit}} \geq 1.0$, the analyst should consider taking steps to mitigate congestion problems.

- If the analyst desires, the CALSIG computer program can perform a queueing analysis at this point in the program. The analysis utilizes the negative binomial distribution to compute the probability that overflow will occur on a given left-turn lane during the analysis period.

The intermediate analysis is a fairly simple method of relating traffic flows, geometric design, and signal phasing strategies with an overall level of service. However, this intermediate analysis assumes that the intersection is operating under optimal signal timing. This is often not the case. Therefore, if the CALSIG user wishes to perform an analysis that takes the signal timing information into consideration, or if the signal timing is to be established by using CALSIG, the procedure should be continued.

As before, the user has essentially three options once the intermediate analysis has been completed:

- Discontinue the procedure and obtain the intermediate analysis results.
- Modify any of the parameters incorporated into the procedures thus far so that more desirable operating conditions can be achieved.
- Continue the CALSIG procedure so that a more comprehensive design or analysis will be performed.

Minimum Green Time Module

The CALSIG module ensures that adequate green times are provided for crossing pedestrians and/or critical traffic volumes. The concept of establishing minimum time green is applicable to both pretimed and actuated signal timing.

If pedestrian traffic is involved, the minimum green time for the phase can be established on the basis of the time required for pedestrians to cross the approach (i.e., the approach intersecting the movement permitted by the subject phase). Equation 9-5 of the 1985 HCM is as follows:

$$gm = 7.0 + (W/4.0) - Y \qquad (1)$$

where

gm = minimum green time (required for pedestrians) (sec);
W = distance from the curb to the center of the furthest travel lane on the street being crossed or to the nearest pedestrian refuge island (ft); and
Y = change interval (yellow and all-red) (sec).

The 1985 HCM uses 7 sec as the initial pedestrian interval. However, other interval lengths may be used. For example, research has indicated that if fewer than 10 pedestrians cross the subject approach per cycle, a 4-sec initial interval is generally adequate. In Equation 1, 4.0 is used as the pedestrian walking speed (expressed in feet per second). Other values can be used as local policy dictates.

If signals with pedestrian-actuated push buttons are present, the minimum green time, as computed by Equation 1, need only be satisfied when the push buttons are actuated. At all other times, minimum green times can be established on the basis of vehicle demand.

Minimum green times should be established to accommodate given traffic volumes for phases without pedestrian movements. A rough estimate for such a required minimum green time may be computed on the basis of critical lane volumes for each phase. The critical lane volume is the largest flow entering the intersection in a given lane. The following equation can be used for determining the minimum green time to accommodate vehicles (*4*):

$$gm = (Vc/C) \times 2.5$$

where

gm = minimum green time required for vehicles (sec);
Vc = critical lane volume (vph); and
C = established (or probable) cycle length (sec).

The value of 2.5 used in this equation represents an average vehicle headway.

The user should check that minimum green times are not larger than actual green times (the CALSIG computer program checks green times automatically). If the actual green times are less than corresponding minimum green times, changes in the cycle splits should be considered.

Cycle Length Module

The cycle length, C, is the time required for the signal to complete a sequence of signal indications. For pretimed signals the cycle length remains fixed, but for actuated and semi-actuated signals the cycle length may vary from cycle to cycle. Principles involved in determining appropriate cycle lengths are similar for both actuated and nonactuated operation.

When cycle length is incorporated into the CALSIG procedure, the average or typical cycle time is used. This typical cycle length is the one that occurs most often during the time period being analyzed. Methods for determining the typical cycle length for actuated operation are discussed later in this section.

Design Procedure: Pretimed Signals

If the cycle length for a pretimed signal is not yet established, it may be computed by using Equation II. 9-1 of the 1985 HCM (Chapter 9, Appendix II):

$$C = LXc/[Xc - \sum_i V/S_{crit}] \qquad (2)$$

where

C = cycle length (sec);
L = lost time per cycle (sec) (may be assumed equal to the sum of the nonoverlapping phase change intervals for each phase);
Xc = critical V/c ratio (volume/capacity) for the intersection (this can be a desired value, selected by the user); and
$\sum_i V/S_{crit}$ = sum of the critical flow ratios for each phase.

Design Procedure: Actuated Signals

An actuated signal's maximum cycle length is a function of the sum of the maximum green intervals for each actuated phase. An appropriate maximum cycle length can be determined by computing an optimum cycle length with an equation developed by Webster (*5*) and then multiplying the resulting value by a factor ranging from 1.25 to 1.50. Webster's equation is as follows:

$$C = \frac{1.5\,L + 5}{1.0 - \sum_i V/S_{crit}}$$

(All terms have been defined previously.) Note that the resulting value will be multiplied by a factor ranging from 1.25 to 1.50.

The actuated signal's typical cycle length must be used in the CALSIG analysis. The typical cycle length may be determined in two ways:

- The typical cycle can be determined by field observation,

or

- The typical timing can be estimated by computation, again using Equation 2.

For semi-actuated signals, user-selected values of $Xc = 0.80$ to 0.90 are used; for actuated signals, a user-selected value of $Xc = 0.95$ is used. A more thorough discussion of the cycle length module is contained in the CALSIG user's manual.

Phase Length Module

The techniques used to incorporate phase lengths into the CALSIG procedure will vary somewhat, depending on the type of control used at the subject intersection. Green times for pretimed signals are fixed, but green times for actuated and semi-actuated signals will vary from cycle to cycle. Average green times of actuated signals are used in a CALSIG analysis.

Design Procedure: Pretimed Signals

If phase durations for pretimed signals are not yet established, they can be established by allocating green times such that the V/c ratios for critical movements in each phase are equal. The V/c ratio for the overall intersection can be computed using Equation 9-2 of the 1985 HCM:

$$Xc = \sum_i V/S_{\text{crit}}\ C/(C - L)$$

(All terms have been defined previously.)

The resulting value for Xc is then incorporated in Equation II.9-2 of the 1985 HCM:

$$gi = V/S_{\text{crit}} \times (C/Xc)$$

where gi is the green time for phase i (sec), and all other terms have been defined previously.

The sum of all green times plus the total lost time per cycle equals the signal's cycle length. If pretimed greens are less than their minimum greens, allocation of enough additional green to meet the minimum green can be considered.

Design Procedure: Actuated Signals

An actuated phase typically has three timing parameters: the initial interval, the vehicle extension, and the maximum green interval. A thorough discussion of the design and analysis techniques for actuated green times is contained in the CALSIG user's manual. A summary of this topic would be lengthy and is therefore not included in this paper.

Comprehensive Analysis Incorporating V/c as Primary MOE

This comprehensive analysis takes all previously described intersection parameters into account. The procedure yields a LOS value for each lane group, as well as for each approach and the overall intersection.

- The capacity of each approach lane group is computed. The value is the product of the lane group's saturation flow rate and the corresponding green ratio (green time/cycle length).
- The volume to capacity ratio, V/c, is computed for each lane group. Relationships between lane group V/c and level of service are taken directly from *Transportation Research Circular 212* (*6*). These values are presented in Table 2.
- The critical V/c ratio for the entire intersection, Xc, is computed using Equation 9-3 of the 1985 HCM:

$$Xc = \frac{\sum_i (V/S_{\text{crit}})\ C}{C - L}$$

(All terms have been defined previously.) A corresponding LOS can be taken from Table 2.

TABLE 2 V/c AND LEVEL OF SERVICE (LOS)

V/c Ratio	Level of Service
0.00–0.60	A
0.61–0.70	B
0.71–0.80	C
0.81–0.90	D
0.91–1.00	E
>1.00	F

The aforementioned analysis procedure comprehensively estimates the portion of an intersection's capacity that is actually utilized by traffic during a specific time period. When this analysis yields unacceptable results, modifications to one or more of the incorporated parameters can be considered. CALSIG's design elements may be consulted for modification suggestions.

Some transportation professionals prefer to use the intersection V/c ratio as a primary measure of effectiveness. V/c is a measure of facility utilization. The volume to capacity ratio can be used by designers when an intersection's size and geometric layout are being determined. However, the 1985 HCM maintains that the portion of capacity actually used at a signalized intersection is not completely indicative of the intersection's LOS. The new HCM states that delay is a better measure of driver discomfort, frustration, fuel consumption, and lost travel time. Thus the LOS criteria for the final comprehensive analysis is average stopped delay per vehicle.

In summary then, V/c may be thought of as a performance measure of the system. Delay is a performance measure describing operation from a user's perspective.

As before, the CALSIG user has three options once the first comprehensive analysis has been completed:

- Discontinue the procedure and obtain analysis results based on V/c.
- Modify any of the intersection parameters and repeat the procedure.
- Continue the CALSIG procedure so that the intersection LOS can be estimated on the basis of average stopped delay per vehicle.

Comprehensive Analysis Incorporating Delay as Primary MOE

This ultimate step in the CALSIG procedure relates the intersection's level of service to the average stopped delay per vehicle that is estimated to occur during the analysis period. This final comprehensive analysis is identical to the 1985 Capacity Manual's operational procedure.

- Delay on each lane group is computed by using Equation 9-18 of the 1985 HCM:

$$d = 0.38\ C \frac{[1 - g/C]^2}{[1 - (g/C)X]} + 173\ X^2\ [(X - 1)] + [(X - 1)^2 + (16X/c)]^{1/2}$$

where d is the average stopped delay per vehicle (sec) and all other terms have been defined previously.

• A progression adjustment factor, taken from Table 9-13 of the 1985 HCM, is applied to each lane group delay value. The adjustment factor ranges from 0.40 to 1.85 and is multiplied by the delay value. This adjustment factor accounts for the vehicle platooning characteristics and the type of signal control at the intersection.

• Corresponding levels of service can be related to each lane group adjusted delay. Relationships between delay and level of service are presented in Table 3 of this paper and are taken directly from Table 9-1 of the 1985 HCM.

• By computing weighted averages, levels of service can be determined for each intersection approach and for the intersection as a whole.

TABLE 3 DELAY AND LEVEL OF SERVICE (LOS)

Stopped Delay per Vehicle	Level of Service
0.00– 5.0	A
5.1 –15.0	B
15.1 –25.0	C
25.1 –40.0	D
40.1 –60.0	E
>60.0	F

In determining the intersection level of service, this analysis takes into consideration a wide variety of known or projected prevailing conditions. If the comprehensive analysis yields unacceptable results, modifications to one or more of the intersection parameters should be considered. CALSIG's design elements can be consulted for suggestions on methods of implementing operational improvements.

SAMPLE CALCULATION

This section consists of an annotated example problem. Because the CALSIG software dramatically reduces the time and effort required to perform the entire CALSIG procedure, the computations for the following example problem were performed by using this program. Most of the figures for this section are the interactive screens produced by the software. The example illustrates CALSIG's design and analysis elements. The intersection to be evaluated is taken directly from the 1985 Capacity Manual (Calculation 4).

Turning Movement Demand and Geometrics Module and Preliminary Analysis

For this problem, projected turning movement demand and approach geometrics are established by the user. The problem examines future traffic demand, so full hourly volumes are used. It is assumed that an entire CALSIG procedure is to be performed for this intersection.

• Total turning movement volumes are entered in each corner of Figure 3. The figure also illustrates the vehicle distribution per lane. The distribution is computed in the preliminary analysis. This step is performed automatically by the CALSIG computer program.

• The sum of the critical turning movement volumes is illustrated in Figure 4. The analysis predicts that the intersection is operating near capacity.

• To achieve a better predicted operation, CALSIG's geometric design criteria are consulted. The criteria indicate that an exclusive right-turn lane should be added to the eastbound approach. This modified scenario is illustrated in Figure 5. The distribution per lane is again computed.

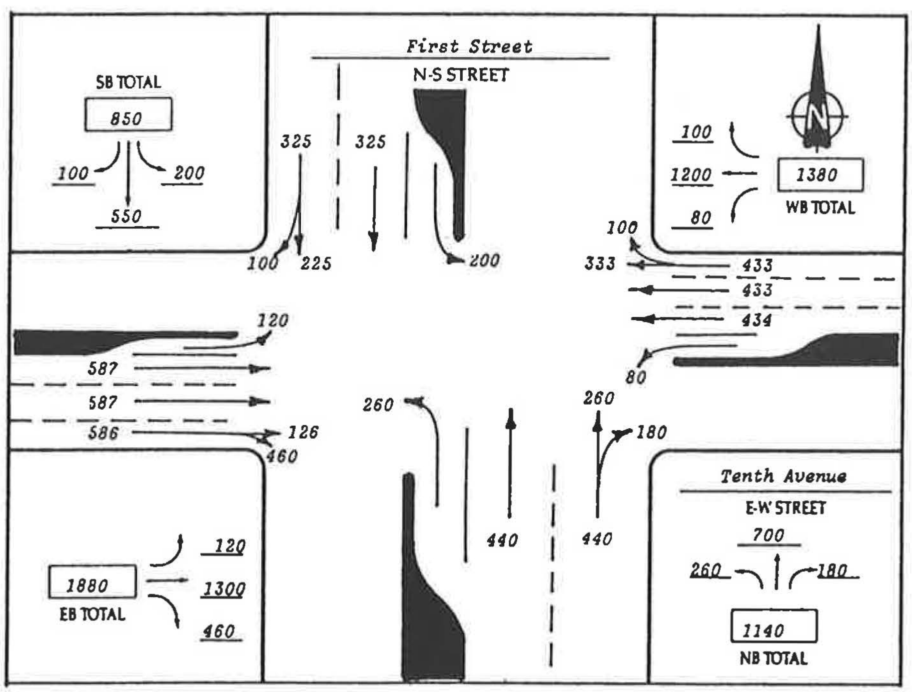

FIGURE 3 Example problem (taken from 1985 HCM).

```
                                                        The 1985 HCM estimates
EBLT =          120         NBLT =          260         Capacity Criteria as
WBTH =          433         SBTH =          325         follows:
         ----------                  ----------
                553                         585         MAXIMUM SUM      CAPAC.
                                                        OF CRIT. VOL     LEVEL
WBLT =           80         SBLT =          200
EBTH =          587         NBTH =          440            0 to 1200     under
         ----------                  ----------         1201 to 1400     near
                667                         640              > 1400      over
            667                         640              1307
----------------      +     ----------------  =  -----------
E-W Critical                N-S Critical

According to the 1985 HCM, this intersection is estimated to be
operating   near   capacity.

Extended vehicle queues may occur at the intersection approaches
during the time period analyzed.
```

FIGURE 4 Critical movement analysis.

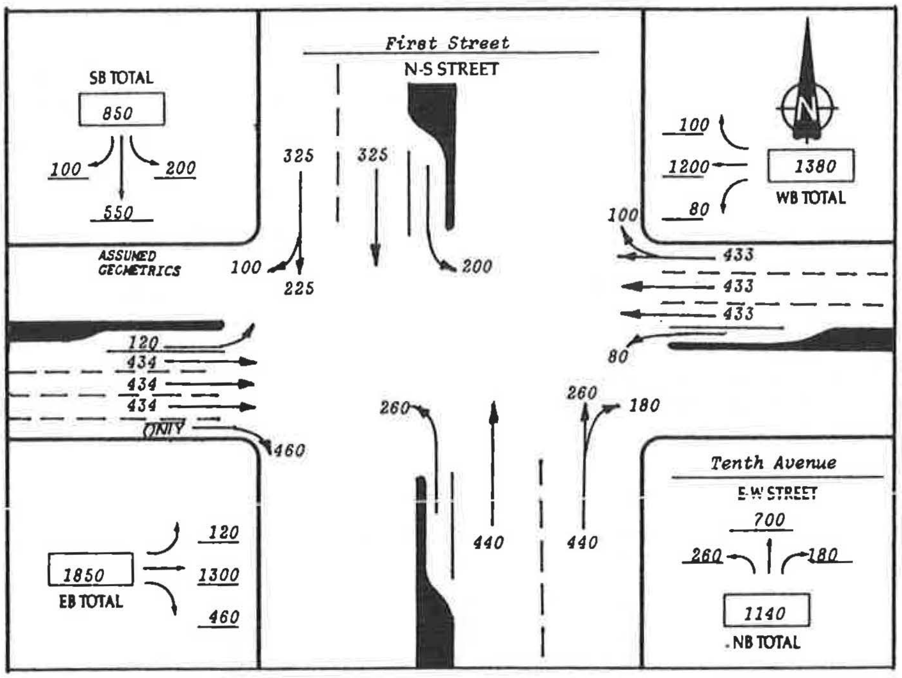

FIGURE 5 Example problem: modified geometrics (taken from 1985 HCM).

• The critical movement analysis now predicts that the intersection will be operating under capacity (Figure 6). This modified geometric plan is therefore adopted by the user.

Phase Design Module

No information concerning the phase design is given in this sample problem. CALSIG's design criteria are therefore used to establish the phase plan. The CALSIG-generated design is shown in Figure 7.

• The CALSIG design criteria have designed a multiphase operation to accommodate heavy left-turn volumes.

The crosses in Figure 7 indicate the movements that occur for each phase. Eastbound and westbound traffic have leading protected left-turn phases. Northbound and southbound traffic have leading protected left-turn (LT) phases, with a phase overlap for NB traffic. (The overlap was designed because the level of NB LT traffic is significantly higher than that of the SB LT traffic.)

• The CALSIG-generated phase design is adopted by the user.

Saturation Flow Rate Module and Intermediate Analysis

Relatively little information is given about the prevailing conditions of this intersection, so CALSIG default values are used for estimating the lane group saturation flow rates. These saturation flows and the intermediate analysis are shown in Figure 8.

```
                                                         The 1985 HCM estimates
EBLT =        120              NBLT =          260       Capacity Criteria as
WBTH =        433              SBTH =          325       follows:
       ----------                     ----------
              553                              585       MAXIMUM SUM       CAPAC.
                                                         OF CRIT. VOL      LEVEL
WBLT =         80              SBLT =          200
EBTH =        433              NBTH =          440          0 to 1200      under
       ----------                     ----------         1201 to 1400      near
              513                              640            > 1400       over
         553                             640              1193
-----------------     +        ----------------- = -----------
E-W Critical                   N-S Critical
```

According to the 1985 HCM, this intersection is estimated to be operating under capacity.

Extended vehicle queues are unlikely to occur at the intersection approaches during the time period analyzed.

FIGURE 6 Critical movement analysis: modified geometrics.

	PHASES						SPECIAL PHASES						
	1	2	3	4	5	6		1	2	3	4	5	6
EB LT	X						EB/WB PED						
EB TH		X											
EB RT		X											
WB LT	X						Protected NB RT SB RT						
WB TH		X											
WB RT		X											
NB LT			X	X			NB/SB PED						
NB TH				X	X								
NB RT				X	X								
SB LT			X				Protected EB RT WB RT						
SB TH					X								
SB RT					X								

FIGURE 7 Phase design.

	Lane Group	No of Ln	Flow in Lane Group	Satur. Flow p/lane	Lane Group Sat.Fl	LUF	v/s	Cr. v/s
EB	ELT	1	120	1500	1500	1.00	0.08	0.08
	ETH	3	1299	1600	4800	1.10	0.30	0.30
	ERT	1	460	1550	1550	1.00	0.30	
WB	ELT	1	80	1500	1500	1.00	0.05	
	ETH+RTTH	3	1299	1600	4800	1.10	0.30	
NB	ELT	1	260	1500	1500	1.00	0.17	
	ETH+RTTH	2	880	1600	3200	1.05	0.29	
SB	ELT	1	200	1500	1500	1.00	0.13	0.42
	ETH+RTTH	2	650	1600	3200	1.05	0.21	

LOS	Σv/s
A	0.00-0.60
B	0.61-0.70
C	0.71-0.80
D	0.81-0.90
E	0.91-1.00
F	> 1.00

Σv/s=0.80
crit

FIGURE 8 Default saturated flows and intermediate analysis.

		PHASES 1	2	3	4	5	6
EB	LT	X					
	TH		X				
	RT		X				
WB	LT	X					
	TH		X				
	RT		X				
NB	LT			X	X		
	TH				X	X	
	RT				X	X	
SB	LT			X			
	TH					X	
	RT					X	

SPECIAL PHASES	1	2	3	4	5	6
EB/WB Peds						
NB RT Protect. SB RT Protect.						
NB/SB Peds						
EB RT Protect. WB RT Protect.						

MINIMUM GREEN TIMES					
1	2	3	4	5	6
3	15	5	1	10	

FIGURE 9 Minimum green times.

An overlap exists for the NB traffic, so the special procedure discussed previously is used to sum the critical flow ratios. The sum of these V/S_{crit} values (0.80) corresponds to a level of service C. No adjustment was made for clearance intervals.

Signal Timing Modules

The CALSIG design elements are used to generate minimum green times, cycle length and phase change intervals, and phase lengths. These values are illustrated in Figures 9–11. It is assumed that pretimed operation is in place.

- Minimum green times are established on the basis of vehicular demand for phases 1, 3, and 4, and on the basis of pedestrian requirements in phases 2 and 5 (Figure 9).
- Computed cycle length is 100 sec. This length was calculated by using Equation II.9-1 of the 1985 HCM (Figure 10).
- Phase lengths are established such that the V/c ratios for critical movements are equal (Equations 9-3 and II.9-2 of the 1985 HCM). Note that the sum of the green times and all nonoverlapping phase change intervals is equal to the cycle length. Also note that all actual green times are larger than their corresponding minimum green times (Figure 11).

```
CYCLE LENGTH:100

PHASE CHANGE INTERVAL 1:  3
PHASE CHANGE INTERVAL 2:  3
PHASE CHANGE INTERVAL 3:  3
PHASE CHANGE INTERVAL 4:  3
PHASE CHANGE INTERVAL 5:  3
```

FIGURE 10 Cycle length and phase change intervals.

Comprehensive Analysis Using V/c as MOE

Lane group capacities are the product of each lane group saturation flow rate and g/C ratio. The computed V/c value for each approach, and the intersection as a whole, indicates LOS D (Figure 12).

		PHASES 1	2	3	4	5	6
EB	LT	X					
	TH		X				
	RT		X				
WB	LT	X					
	TH		X				
	RT		X				
NB	LT			X	X		
	TH				X	X	
	RT				X	X	
SB	LT			X			
	TH					X	
	RT					X	

SPECIAL PHASES	1	2	3	4	5	6
EB/WB Peds						
NB RT Protect. SB RT Protect.						
NB/SB Peds						
EB RT Protect. WB RT Protect.						

ACTUAL GREEN TIMES					
1	2	3	4	5	6
9	34	15	9	24	

FIGURE 11 Phase length module.

	Lane Group	Flow in Lane Group	Lane Group Capac.	Lane Group v/c	Lane Group LOS	Appro. v/c	Appro. LOS
EB	ELT	120	135	0.89	D	0.88	D
	ETH	1429	1632	0.88	D		
	ERT	460	527	0.87	D		
WB	ELT	80	135	0.59	A	0.86	D
	ETH+RTTH	1429	1632	0.88	D		
NB	ELT	260	360	0.72	C	0.84	D
	ETH+RTTH	924	1056	0.87	D		
SB	ELT	200	225	0.89	D	0.89	D
	ETH+RTTH	682	768	0.89	D		

V/C (Crit)=0.88

Intersection LOS = D

FIGURE 12 ***V/c*** **and level of service (LOS).**

	Lane Group	v/c Ratio	Green Ratio (g/C)	Delay d1	Lane Group Capac	Delay d2	Prog Fact	Lane Group Delay	Lane Group LOS	Appr. Delay	Appr LOS
EB	ELT	0.89	0.09	34.20	135	31.71	1.00	65.91	F	31.64	D
	ETH	0.88	0.34	23.57	1632	4.07	1.00	27.64	D		
	ERT	0.87	0.34	23.54	527	10.47	1.00	34.01	D		
WB	ELT	0.59	0.09	33.24	135	4.78	1.00	38.02	D	28.24	D
	ETH+RTTH	0.88	0.34	23.57	1632	4.07	1.00	27.64	D		
NB	ELT	0.72	0.24	26.55	360	4.76	1.00	31.31	D	30.25	D
	ETH+RTTH	0.87	0.33	23.98	1056	5.95	1.00	29.94	D		
SB	ELT	0.89	0.15	31.68	225	22.38	1.00	54.06	E	40.79	E
	ETH+RTTH	0.89	0.24	27.90	768	8.81	1.00	36.71	D		

Intersection Delay:31.93 sec/veh Intersection LOS:D

FIGURE 13 Delay and level of service (LOS).

Comprehensive Analysis Delay as MOE

Average stopped delay per vehicle is computed by using Equation 9-18 of the 1985 HCM. A LOS D is predicted (Figure 13).

An inspection of the results in each analysis shows that all levels of analysis are in fairly close agreement. Because CALSIG ultimately generated all intersection parameters, improvement of the operation by implementing design changes is probably not worthwhile.

CONCLUSIONS

This paper has described a proposed procedure, called CALSIG, for the design and analysis of signalized intersections. CALSIG was developed in an effort to enhance existing procedures in Chapter 9 of the 1985 HCM. The CALSIG procedure is not actually a new procedure but is rather an integration of existing methodologies. As a result of this integration, CALSIG has a multilevel analysis structure. This structure does not necessarily render the procedure less complex than the existing HCM operational analysis; however, the multilevel format does offer additional flexibility.

The CALSIG methodology also possesses design elements. These design guidelines serve to generate unestablished intersection parameters and help to determine effective operational improvements.

CALSIG was designed to aid transportation professionals in the state of California. CALSIG is applicable, however, to any location in the United States. Adjustments for local conditions are always recommended.

ACKNOWLEDGMENTS

The research described in this paper would not have been possible without the aid of numerous people. The project staff wish to thank the many knowledgeable professionals who participated in this project's critical review phase. Without their thoughtful insights, a thorough evaluation of the 1985 Capacity Manual could not have been performed. We wish to explicitly acknowledge Ignacio Sanchez, who programmed the CALSIG computerized software package while he was attending graduate school at the University of California. We also wish to thank Ranguald Sagen, visiting scholar to the Institute of Transportation Studies, for his support and advice.

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Abridgment

Signal Complaint Aid for Dispatchers (SCAD): An Expert System

KENNETH A. SHARP AND PETER S. PARSONSON

A predictive, knowledge-based expert system to increase the efficiency of responding to traffic-signal malfunctions is outlined. Signal Complaint Aid for Dispatchers (SCAD) does not modify published research into a problem-solving routine. Instead, SCAD is the result of programming the signal engineer's decision-making process, in which the engineer first compares a traffic signal complaint with his or her knowledge and experience in the operation and maintenance of that traffic signal and then arrives at a plan of action. SCAD is designed to provide an engineer's in-depth knowledge of traffic signal theory and practice to a dispatcher who does not have the engineer's expertise in handling complaints. Expert systems are designed to solve complex problems that are poorly defined and not well understood. SCAD exhibits several of the advantages of an expert system. The program substitutes for the expert when it is impractical for the expert to be present. It provides expertise to lower-level personnel, such as a dispatcher with a high school education and a rudimentary knowledge of the roadway system. It is capable of learning from its mistakes by comparing its filed predictions with trouble call/response reports from the maintenance agency. SCAD documents the problem-solving knowledge that is being lost because signal engineers are leaving the public sector to avoid the increasing liability associated with their duties.

The potential for personal injury and liability caused by traffic signal malfunction has caused many jurisdictions to establish formalized reactions for handling complaints from the public (trouble calls). Strategies that have been implemented to increase efficiency in handling trouble calls include a well-publicized telephone line dedicated to trouble calls, maintenance scheduling, inspection scheduling, computerized filing and reporting systems for traffic signal operations and maintenance, and the creation of specialized crews to respond to trouble calls. An expert system has been designed to make use of these strategies.

Accuracy is required in responding to a trouble call. The initial telephoned trouble call is often garbled, inaccurate, or incomplete by the time it is relayed to the response crew. Precious time is lost by notifying the wrong agency, dispatching the crew to the wrong intersection, or not taking the right equipment.

An expert system is designed to solve problems in the same manner in which an expert deals with them. The expert in the domain of trouble calls is the traffic signal engineer who is directly responsible for the timing and operation of the signals within the jurisdiction. The signal engineer has specialized knowledge of equipment and timings, traffic patterns, and maintenance history. Through telephone contact with and skillful interrogation of the person with the complaint, the signal engineer can often predict the cause of the field problem and instruct the response crew on what equipment to take, what to look for, what procedures to follow, and what procedures to avoid.

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Signal Complaint Aid for Dispatchers (SCAD) is an expert system that has been developed to assist a dispatcher who receives a trouble call on a telephone. SCAD guides the dispatcher in asking questions of the caller, in contacting the right personnel to act on the matter, and in giving those personnel the information that they need. This abridgment summarizes the major points of SCAD. Copies of the unabridged paper are available from author K. A. Sharp.

KNOWLEDGE BASE

A knowledge base is the portion of the expert system that contains the data describing the problem domain (*1*). In the case of trouble call/response the knowledge base includes specific information for each intersection. A data base of seven cross-referenced files was created to represent different aspects of the knowledge in the problem domain: the operational characteristics, the appropriate jurisdiction and the agencies involved, the recent history of incidents, the signalized locations with duplicate names, and specialized knowledge of the type of equipment, the phasing, and the timing. This information must be input to the knowledge base by the signal engineer. The expert system is designed to "think" like the signal engineer and must have the benefit of his specialized knowledge.

The data base used to develop and test the SCAD expert system consisted of 50 signalized intersections located within four jurisdictions known to author K. A. Sharp from his field experience in Richmond County, Georgia. The intersections were chosen to encompass the possible combinations of operational characteristics. The data base is designed to handle over 1,000 signalized intersections located within multiple jurisdictions. The maximum number of intersections that SCAD can handle is dependent on the data base software and available memory. MicroPro International Corporation's DataStar™ software was used to create the data base because this software produces an ASCII file that can easily be manipulated with BASIC. Ten BASIC programs were written to perform all data manipulations on the knowledge base files, to handle string processing, and to transfer data both ways between the knowledge base and the inference engine, which is described next.

INFERENCE ENGINE

The inference engine contains the general problem-solving knowledge that is used to arrive at a solution (*1*). An inference engine consists of two parts: the production rules (IF . . . THEN . . . ELSE statements that use symbolic logic to represent strategies) and a shell program that supports the rules. Production rules are appropriate when the domain knowledge results from empirical associations developed through years of experience with solving problems in a particular area (*1*). Insight 2+™ version 1.3 by Level Five Research, Inc., was used to create the inference engine.

The SCAD inference engine is based in part on interviews with experts in the domain of trouble call/response (Howell Lancaster, Georgia Department of Transportation; Peter Parsonson, Georgia Institute of Technology; Marvin Rickard, Gwinnett County Traffic Engineering Department; David Amerson, Richmond County Traffic Engineering Department). During these interviews, situations were presented to each expert, and the method by which the expert arrived at a solution was noted and dissected in detail. The resulting rule-of-thumb decisions (heuristics) are put into the form of production rules and run with sample problems to compare the expertise gained by the computer program with the expert's real-world solutions.

For purposes of discussion, SCAD can be broken into three sections: contact-interrogation session, complaint-cause analysis, and notification procedure.

Contact-Interrogation Session

The expert system is started at the moment of contact between the dispatcher and the contact. The computer's clock is accessed to set the time and date. The first item of input data needed by the expert system is the trouble call complaint. The request for information is presented to the dispatcher in the form of a multiple-choice question (menu) concerning typical signal complaints. The response to the question is then checked for overlap with another complaint on the list. For example, "conflicting indications" can be a symptom of a "twisted signal head." Clarification is made by questioning the contact.

After the complaint has been chosen, the expert system requests and verifies the location of the complaint. Then the contact is asked a series of questions about the complaint. These questions, which are based on the complaint type, reduce ambiguity in the description of the problem. They are presented to the dispatcher in the form of menu choices, true-false questions, and prompts for keyboard entry. Queries are refined with help screens that tell the dispatcher exactly what is desired in the way of a response. Some responses cue additional questions. When all questions have been asked, the dispatcher is told to hang up the telephone, and the contact-interrogation session ends.

The contact-interrogation session was designed to minimize execution time. The interrogation occurs in real time (i.e., while the contact is still on the telephone). The number of questions asked is also minimized. SCAD not only decides which questions to ask but also decides which questions not to ask on the basis of the complaint and on the answers to previous questions. Menus are used for data input whenever feasible to minimize the number of key strokes. On an IBM PC/AT or similar computer, the most complex interrogation routine lasts about a minute.

Execution time is the nemesis of an expert system. The control mechanism of the inference engine uses forward-chaining logic and pattern matching to search a tree in pursuit of its goal. Additional rules therefore increase the search time exponentially because alternate rules are being applied to the same situation. (With algorithms, on the other hand, the addition of rules changes the execution time only linearly or logarithmically.) Because of this exponential factor, many expert systems look good in prototype form but prove to be time consuming and unwieldy in the more complex production form (*2*).

Because of this exponential factor, SCAD is written in a modular arrangement to facilitate expansion of the variety of complaints (18 typical complaints are programmed currently) without jeopardizing the total execution time. Thus the execution time is dependent primarily on the time needed to search a file for the proper record. The greater the number of intersections, the longer the total execution time. External data access is kept to a minimum during the contact-interrogation session to ensure a short interaction time between the dispatcher and the contact.

Complaint-Cause Analysis

After the dispatcher is told to hang up the telephone, the expert system uses a BASIC program to find the operational characteristics record for the location. SCAD compares this information with the complaint and other relevant information gathered in the contact-interrogation session.

The analysis is done in two steps. In the first step, the complaint and the gross operational characteristics are used to establish which general areas of signal operation (control, coordination, actuation, display, and/or timings) may be causing the complaint. Each general area is assigned a certainty factor ranging from 0 to 100. A certainty factor is a number that measures the analyst's confidence that a statement is valid (*1*). The sum of the certainty factors equals 100.

In the second step of the complaint-cause analysis, each general area of operation is examined and subdivided into specific operational characteristics: such as "solid-state controller" and "signal-conflict monitor" in the control area. On the basis of these specific characteristics, predictions are made of possible equipment malfunctions and the actions that will be taken to solve the complaint. Each solution has an overall certainty factor that is computed by multiplying the certainty factor of the general area times the certainty factor of that solution.

The expert system pursues all possible paths that yield solutions to the complaint. It checks for combinations of complaint and operational characteristics that are either impossible or else not indicative of a malfunction. For example, a flashing beacon that is reported to be on flash would not appear to be a malfunction. In such a case, a crew would be dispatched, but SCAD would form the prediction "Solution IS Do nothing—Location operating normally" and assign to it a high certainty factor.

Notification Procedure

After predictions of the solution for the complaint have been made, SCAD enters the final phase of processing: the notification procedure. In this procedure, a printout is produced that lists the location and the description of the complaint (using data from the contact-interrogation session), the urgency of the response, the agencies to notify, the equipment to take on the call, the special instructions for each agency that will be dealing with the inspection or the complaint, and the actions that would require the trouble call crew to contact the signal engineer call (on the basis of the complaint-cause analysis). This printout contains information that will produce a quicker, more thorough response. SCAD's final action is to log the trouble call into the knowledge base's signal-incident history file, fixing the time at which the agency was notified of the complaint.

SYSTEM REQUIREMENTS

To field test SCAD properly, an interested agency must be found. SCAD requires a dedicated telephone line, an IBM PC/AT-compatible with hard disk and internal clock, and existing computerized files of signal equipment inventories and signal incident histories. The files have to be stored in a form accessible by Insight 2+ or BASIC.

SCAD's memory requirements are extensive. Over 700 kilobytes of storage (700K) are needed. The SCAD inference engine (which presently consists of 415 production rules) requires 132K, and the Insight 2+ software that supports it needs 490K. The DataStar software occupies 72K, and the DataStar data files, input forms, and index files (the knowledge base) total 36K for the 50-intersection sample database. The BASIC programs that interface SCAD with the data base total 25K.

In this breakdown, memory required for storing the intersection data is only 5 percent of the total memory requirement. The final version of SCAD and a 1,000-intersection data base will require 1,500K (1.5 megabytes) of storage. A "fast" computer (based on 80286 or 80386 chip technology) will be a necessity for the final version.

NEED FOR FURTHER DEVELOPMENT

As is often the case with expert systems, the developmental software is not the appropriate software for the production version. The software currently used is inadequate, particularly in the area of file manipulation. In addition, the software used to develop SCAD was acquired for educational purposes and not production purposes—distribution of SCAD in its present form may infringe on copyright laws. For these reasons, software for the production version (both the expert system and the data base) would have to be purchased, and SCAD would have to be rewritten.

The design of expert systems is an infant programming field. Adequate software does not yet exist, and new systems are being marketed almost weekly. The production software should have these characteristics: an affordable price, a compiler with run-time debugging features and variable name tables (extremely important because the expert system uses pattern matching, and names must be exact), certainty factors, extensive use of symbolic logic, complete and direct access (random access using index files) to a variety of commercial data base software, support for numeric calculations and string processing within the rule structure, good documentation, and for final distribution purposes, affordable run-time versions.

The data base software should be affordable, support cross-referenced and indexed files, and allow random access by the expert system. It would be preferable for the software to support an existing traffic operations data base. Before the SCAD expert system was written, a search was made for a commercial traffic signal incident data base and reporting system that was extensively used in the traffic engineering community. The intent of the search was to match the expert system to a standard traffic data base. The researchers found that there is no standard design for an incident file. A jurisdiction in need of such a system usually develops its own files, reports, and programs (Ken G. Courage, unpublished data). A data base was created for development of SCAD, but an existing data base would be preferred.

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Sketch Planning Process for Urban Isolated Signalized Intersection Improvements

FAZIL T. NAJAFI

A simple and practical procedure involving operational, economic, and safety impact considerations is developed for evaluating improvements to isolated signalized intersections. The result is a step-by-step technique that allows planners and engineers to compare the benefits and costs of improvements to isolated signalized intersections. The procedure (or sketch planning process) was synthesized from the literature and from a survey of current practices in Florida at city, county, and state levels. It incorporates benefit/cost techniques and the Signal Operations Analysis Package (SOAP), thus improving on previous methods. Factors that were incorporated into the process include right-of-way needs, fuel consumption, benefit/cost ratio, staged improvement options, and safety considerations. The average delay and level of service attributable to alternative staged improvement plans during the planning horizon is exhibited graphically in a case study, illustrating the adaptability of the system in achieving an acceptable level of service at a specified future date. The case study shows that the sketch planning process developed through this research can be applied to almost any urban isolated signalized intersection, providing that accurate input data are available and that practical results can be generated.

The objectives of this study were to develop a sketch planning process (SPP) that could be used by planners and design engineers to

- Evaluate the operational impacts of various improvement alternatives on the performance of urban isolated signalized intersections (ISIs),
- Facilitate appropriate right-of-way acquisition and stage improvements of ISIs to meet standards,
- Optimize the benefit/cost ratio of intersection improvement,
- Coordinate the SPP with the application of the Signal Operations Analysis Package (SOAP) and 1985 *Highway Capacity Manual* (HCM),
- Conduct a survey of current methods used to improve urban intersections by the Florida Department of Transportation (FDOT) and by selected counties and cities, and
- Develop a case study of an existing hazardous intersection in the city of Gainesville, Florida, to demonstrate the use of the SPP with a generic example.

The SPP for urban ISIs and its application were developed in response to a request from FDOT in support of long-range planning activities. Florida is experiencing rapid population growth. As a result, a high number of ISIs are operating below acceptable standards. To justify capital expenditures for intersection improvement, benefits must exceed costs. This is particularly true in cases for which improvement funds are limited or when a specific project may be controversial. The application of an SPP could identify critical intersections and would generate alternatives, with average delay as level-of-service (LOS) values versus time. The results would enable local transportation officials to identify the alternatives that could be planned for staged improvements over a specified time.

The variables that are used in the description and analysis of intersection performance are level of service, volume capacity (*V/C*) ratios, saturation flow rates, delay, peak hour volume, headway and so on. Most of these are factors relevant to this research, and they have subsequently been incorporated into the SPP.

In planning improvements to intersections, the SPP takes into account three basic types of consideration: (a) operational, (b) safety, and (c) economic. SOAP was developed at the University of Florida as an operational tool and was incorporated into the SPP for the signal timing optimization and benefit-cost evaluation of ISIs in this research effort.

Sets of questionnaires that covered concepts relevant to this research were developed for a survey of current practice as part of an attempt to improve ISIs within cities, counties, and the state of Florida. Fifteen cities, counties, and districts in Florida were selected, and officials there were interviewed. Survey results showed that there was no step-by-step procedure by which engineers could determine cost-efficient intersection improvements. As a matter of practice, most decisions were being made by engineering judgment and accident records. Consequently the SPP, based on the principle of benefit-cost analysis and signal optimization, was developed as an aid.

The SPP for the improvement of ISIs is a systematic technique that allows the analyst to input existing operational, safety, and intersection geometry data and then estimate future conditions, use SOAP (*1*) and benefit-cost technique to compare alternatives, identify solutions to implement staged construction options to make best use of the available funds, and determine the future right-of-way needs.

The application of the SPP was demonstrated in a case study of a hazardous intersection in Gainesville, Florida. The results of the case study showed that SPP could generate practical results when applied to a typical ISI for which accurate input data are available. The results of the case study allowed local transportation officials in Gainesville to judge the conditions

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under which intersection improvement would be most economically viable (level of service versus time). The improvements can be staged for implementation with specified future timing within the planning horizon.

FORMULATION OF THE SKETCH PLANNING PROCESS

The systematic planning process developed in this research was designed to improve urban isolated signalized intersections. With slight modifications, it can also be applied to other types of urban intersections. It is a systematic process (Figure 1) in which each step requires an input and then a computation or decision, or a combination of the two.

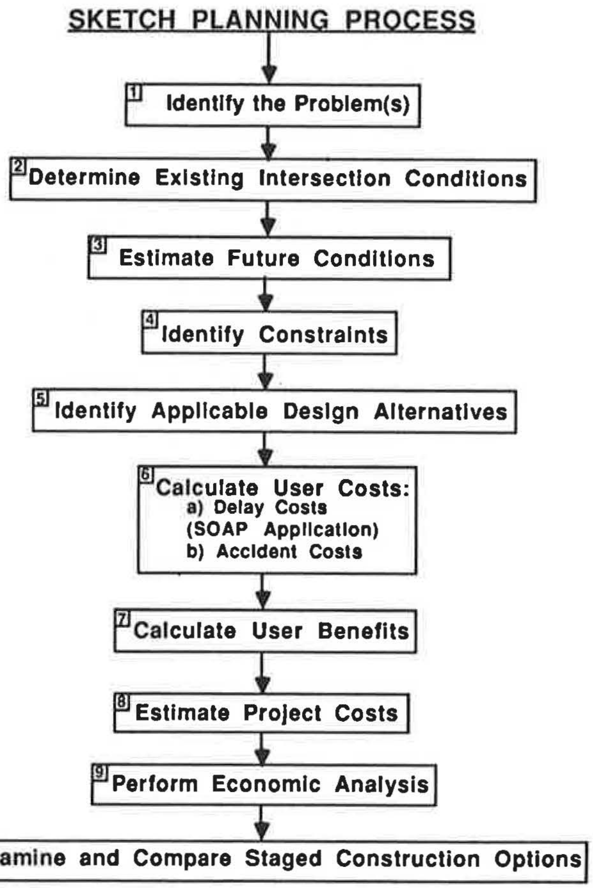


FIGURE 1 Formulation of a systematic planning process to improve urban isolated signalized intersections.

Step 1: Problem Identification

In Step 1 of the SPP (Figure 1) the problems must be identified. Ordinarily, intersection improvements are needed for four basic reasons: (a) problems with signal operations (excessive delay, congestion), (b) safety problems (high rate of property damage, injury, or fatal accidents), (c) occurrence of land development (establishment of new facilities or businesses in the vicinity of the intersection), and (d) need for additional right-of-way to sustain the intersection capacity (excessive delay and fuel consumption, among other problems). It is essential for the analyst to identify any anticipated future problems that are likely to occur within the planning horizon, as well as any existing problems. Periodic traffic volume counts and accident record summaries will help identify existing and future sources of difficulty.

Step 2: Determination of Existing Intersection Conditions

The second step involves the documentation of existing conditions, including traffic counts, and evaluation of available data. The existing intersection conditions are classified into three main categories:

- Geometrics and traffic control, including
  - Plan of the entire intersection layout;
  - Pavement and lane widths;
  - Median geometry (both length and width);
  - Extent of curb parking, with measurements;
  - Right-of-way requirements, with extent of development in adjacent areas that may need to be acquired; and
  - Speed limit.
- Operational conditions, including
  - Traffic volume counts, conducted hourly or in multiples of 15 minutes throughout the prime use hours of a representative day;
  - Number of lanes or capacity of every traffic movement for all directions (SOAP can accommodate both values);
  - Percentage of heavy vehicles (trucks) in each traffic movement for all directions; and
  - Existing phasing and signal timing.
- Safety conditions, including
  - Accident rate (accidents per million vehicles per year);
  - Accident severity distribution (property damage, injury, fatal); and
  - Accident type (rear end, head-on, sideswipe, etc.).

Step 3: Estimation of Future Conditions

The third step of the SPP (Figure 1) is to estimate the future condition of the intersections. Consideration of future traffic movements is required in determining the optimum improvement plan over the planning horizon. The future travel demand can be estimated as a factor of the type of development in the area, population characteristics, and other socioeconomic factors. The growth rate technique is a simplified procedure that was used in the case study to estimate the annual traffic growth rate over the planning horizon. In some urban areas, future development scenarios are reasonably predictable and traffic demand can be forecast with some degree of accuracy. In such areas, planning analyses provide reasonable prediction of trip generation, trip distribution, modal split, and traffic assignment. Assistance from a local planning agency may increase the accuracy and reliability of future projections. Common practice involves a planning period of 20 years. However, whenever possible, better long-term results are obtained by estimating the traffic movement that will occur at the time the area is fully developed, regardless of when this is expected to occur.

At an urban intersection it is usual to expect different rates of traffic volume increase for each approach or even for each traffic movement. This predictable divergence among future traffic movement characteristics can cause important repercussions in the planning process. SOAP allows the user to assign an appropriate growth rate for each traffic movement. When growth is rapid, erratic, or both, frequent future reanalysis of the intersection with the latest available information is recommended.

Projected future conditions over the planning period should include estimates of (a) annual traffic growth rate, (b) proportion of heavy vehicles in the traffic stream, and (c) safety conditions. In an analysis of alternative improvement options for a given intersection, it sometimes happens that no single improvement alternative will last for the entire planning period. In such cases, alternative improvements can be analyzed for an optimum combination. In other words, a series of improvement alternatives can be planned and scheduled for sequential construction at suitable time intervals spanning the planning horizon.

Step 4: Identification of Constraints

It is not unusual to find that only a few alternative solutions are available for dealing with urban intersection improvement projects. For example, as the cost escalates for additional right-of-way, the designers must decide whether to purchase additional road area or to employ other solutions, such as narrowing the existing lanes, removing on-street parking, and so on. The sketch planning process can help designers determine the break-even point for purchasing the added right-of-way needed to improve the intersection condition (this is shown in Figure 5 and illustrated in the case study). In addition, designers also can consider purchasing or otherwise reserving additional strips of right-of-way to be used for staged development of the intersection later within the planning period.

Step 5: Identification of Applicable Design Alternatives

The planner should identify improvement alternatives that are safe and applicable under physical, operational, and economic constraints. Possible alternatives for improving urban at-grade intersections are

- Installation of exclusive turn lanes,
- Upgrading traffic control system and signal coordination,
- Signal timing optimization,
- Addition of through lanes,
- Access control,
- Turning radius treatment,
- Installation of traffic islands, and
- Improvement of sight distance and angle.

For the case study, only signalization improvements and installation of exclusive turn lanes are considered.

Once all the existing and future intersection conditions have been determined, SOAP can be run for two time frames—once for the base year (Year 0) and once for the final year (Year 20)—to determine the operational performance of the intersection over the planning horizon. (It must be recognized that a higher degree of accuracy would result from using shorter intervals of 5, 10, 15, and 20 years.) The measures of effectiveness (delay, percentage stops, fuel consumption, queue length, and V/C ratio) and the results of the left-turn capacity analysis can help evaluate the need for improvement.

Step 6: Calculation of User Costs

The next step, once the applicable design alternatives have been identified, is to calculate the user costs associated with each one of these alternatives. The user costs are divided into two categories: (a) delay costs and (b) accident costs.

Delay Costs (Step 6a)

Delay costs consist of additional time and operating costs due to deceleration prior to a stop and acceleration after a stop at an intersection, plus the cost of idling while stopped. Operating costs include fuel and oil consumption, tire wear, maintenance, depreciation, and other related costs. In the user cost calculations, the operating costs due to deceleration before and acceleration after stops will be referred to as running costs, and those that are incurred while stopped will be called idling costs. Intersection delay costs depend primarily on the type and configuration of the traffic control devices employed, the level of traffic on the section, and the speed at which the signal is approached (2). After the procedures given in the AASHTO manual and two other studies conducted by the California Department of Transportation (2–4) were reviewed, a combined value (for cars and trucks) of $5.50 per vehicle hour was selected for use in the case study.

The running and idling cost factors are obtained from the Federal Highway Administration (FHWA) report Vehicle Operating Costs, Fuel Consumption, Pavement Type and Condition Factors (5). These operating cost factors reflect the 1980 values for passenger cars and must be updated to the year of analysis. The updating procedure outlined in the AASHTO manual (2) is used to convert the 1980 values into 1987 values. The updating multiplier equations for running and idling costs are

$$M_r = c_g\mathrm{CPI}_g + c_o\mathrm{CPI}_o + c_m\mathrm{CPI}_m + c_t\mathrm{CPI}_t + c_d\mathrm{CPI}_d \quad (1)$$

$$M_i = c_g\mathrm{CPI}_g + c_o\mathrm{CPI}_o + c_m\mathrm{CPI}_m + c_d\mathrm{CPI}_d \quad (2)$$

where

M_r = multiplier for updating running costs due to speed change cycles;
M_i = multiplier for updating idling costs;
c_g, c_o, c_m, c_t, c_d = coefficients of multiplier equation for gasoline, oil, maintenance and repair, tires, and depreciation [calculated as the proportion of total cost contributed by a cost item (see Table 1) divided by 1980 Consumer Price Index (see Table 2) for that item];
CPI_g = Consumer Price Index, gasoline;
CPI_o = Consumer Price Index, motor oil;

CPI_m = Consumer Price Index, maintenance and repair;
CPI_t = Consumer Price Index, tires; and
CPI_d = Consumer Price Index, new cars.

TABLE 1 PROPORTIONS OF VEHICLE OPERATING COSTS

	Cost (as percent of total cost)	
Item	Running	Idling
Gasoline	70	85
Motor oil	1	1
Maintenance and repair	3	5
Tires	15	–
Depreciation	11	9
	100	100

TABLE 2 1980 AND 1987 CONSUMER PRICE INDEXES

Item	December 1980	February 1987
Gasoline	373.3	287.5
Motor oil[a]	138.8	154.9
Maintenance and repair	280.1	373.0
Tires	182.1	171.1
New cars	184.5	230.2

NOTE: 1967 = 100, unless otherwise noted.
[a]December 1977 = 100.

The calculation of coefficients of the multiplier equations are shown in Tables 3 and 4. When the values in these tables are used, the multiplier equations to update 1980 running idling cost factors become

$$M_r = 0.0019\ CPI_g + 0.0001\ CPI_o + 0.0001\ CPI_m + 0.0008\ CPI_t + 0.0006\ CPI_d \quad (3)$$

$$M_i = 0.0023\ CPI_g + 0.0001\ CPI_o + 0.0002\ CPI_m + 0.0005\ CPI_d \quad (4)$$

Equations 3 and 4 can be used to update the 1980 running and idling cost factors. If the proportions given in Table 1 change significantly due to a differential rate of inflation, the multiplier coefficients have to be recalculated on the basis of new proportions. The 1987 Consumer Price Indexes are applied to Equations 3 and 4 to determine 1987 running and idling cost factors:

$$M_r = 0.0019(287.5) + 0.0001(154.9) + 0.0001(373.0) + 0.0008(171.1) + 0.0006(230.2) = 0.87 \quad (5)$$

$$M_i = 0.0023(287.5) + 0.0001(154.9) + 0.0002(373.0) + 0.0005(230.2) = 0.87 \quad (6)$$

Once the user cost factors have been determined and updated, the annual delay costs can then be calculated. The user cost equations require total intersection delay and

TABLE 3 COEFFICIENTS OF THE MULTIPLIER FORMULA TO UPDATE 1980 RUNNING COST FACTORS

Item	Coefficient
Gasoline	70%/373.3 = 0.0019
Motor oil	1%/138.8 = 0.0001
Maintenance and repair	3%/280.1 = 0.0001
Tires	15%/182.1 = 0.0008
Depreciation	11%/184.5 = 0.0006

TABLE 4 COEFFICIENTS OF THE MULTIPLIER FORMULA TO UPDATE 1980 IDLING COST FACTORS

Item	Coefficient
Gasoline	85%/373.3 = 0.0023
Motor oil	1%/138.8 = 0.0001
Maintenance and repair	5%/280.1 = 0.0002
Depreciation	9%/184.5 = 0.0005

percentage of stops values, which can be obtained from SOAP analysis (other computer models also can be used to determine delay). SOAP should be run twice for each alternative, once with the existing traffic volumes and a second time with the estimated future traffic volumes. To convert daily delay cost values into annual values, 365 days/yr has been assumed. (It should be noted that the volumes used are usually weekday volumes.) In the areas for which traffic demand drops significantly during certain periods of the year, a lower value can be used for the length of time during which the low traffic demand occurs. The annual user costs due to delay at an intersection are calculated as shown below.

Travel Time Cost

$$C_t = 365\ u_t D h_t \quad (7)$$

where

C_t = travel time cost ($/yr),
u_t = unit value of travel time ($/vehicle hour),
D = total intersection delay (vehicle hour/day), and
h_t = adjustment factor for heavy vehicles.

Running Cost Due to Speed Change and Stopping

$$C_r = 365\ (VS/1{,}000)\ (f_r M_r) h_r \quad (8)$$

where

C_r = running cost due to speed change and stopping ($/yr),
V = traffic volume (vehicles/day),
S = percentage of stops,
f_r = running cost factor ($/1,000 cycles),

M_r = running cost multiplier for updating 1980 values, and
h_r = adjustment factor for heavy vehicles.

Idling Cost

$$C_i = 365\ D\ (f_i M_i) h_i \tag{9}$$

where

C_i = idling cost ($/yr),
D = total intersection delay (vehicle hours/day),
f_i = idling cost factor ($/vehicle hour),
M_i = idling cost multiplier for updating 1980 values, and
h_i = adjustment factor for heavy vehicles.

Then the total delay cost (DC) is calculated as the summation of total travel time cost, running cost due to speed change and stopping, and total idling cost:

$$\mathrm{DC} = C_t + C_r + C_i \tag{10}$$

Accident Costs (Step 6b)

Costs of accidents are the product of estimated accident rates and unit costs of accidents by degree of severity. The procedure for accident cost calculations is shown in Figure 2. Many studies and manuals are available (*2, 6–10*). The analyst is advised to use a reference that is based on statistical data measured in the particular region or state in which the subject intersection is located. Otherwise, a source derived from a large sample localized to describe the subject intersection adequately may be employed. A number of these references give estimated accident reductions in dollars. Here too the analyst should carefully examine the unit accident costs used and then judge the validity of their application to a subject intersection. Otherwise, the estimation of an accident cost in a noncomprehensive manner is speculative and creates questions concerning the accuracy and credibility of the analysis.

Step 7: Calculation of User Benefits

Benefits due to intersection improvements include two major components: user benefits and signal operating benefits. User benefits are calculated as the difference of total user costs associated with existing and improved conditions (*11*), as follows:

$$\mathrm{UB}_i = \mathrm{UC}_0 - \mathrm{UC}_i \tag{11}$$

where

UB_i = total user benefits for alternative i ($/yr),

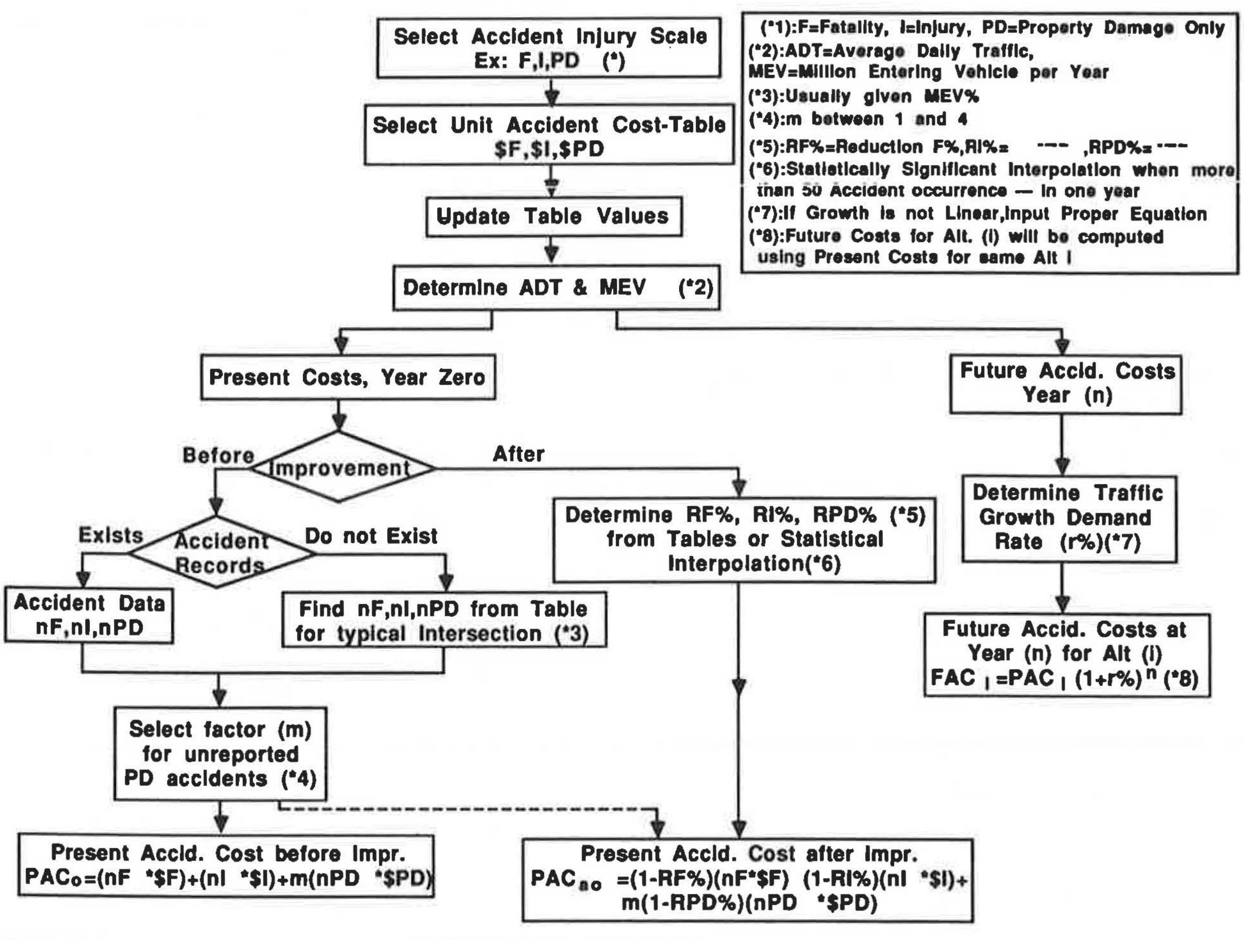


FIGURE 2 Systematic process to formulate accident cost calculation.

UC_0 = total user costs for alternative 0 (the base condition; \$/yr), and
UC_i = total user costs for alternative i (\$/yr).

The total user costs are calculated by using the following equation:

$$UC_i = DC_i + AC_i \qquad (12)$$

where

DC_i = total delay costs (time, running, and idling) for alternative i (\$/yr), and
AC_i = total accident costs for alternative i (\$/yr).

The signal operating benefits or disbenefits are calculated similarly, as follows:

$$OB_i = OC_0 - OC_i \qquad (13)$$

where

OC_0 = current signal operating costs (\$/yr), and
OC_i = signal operating costs due to alternative i (\$/yr).

Then total benefits are obtained by adding the two components:

$$B_i = UB_i + OB_i \qquad (14)$$

Step 8: Estimation of Project Costs

The project costs can be divided into investment costs (construction, planning and design, right-of-way acquisition and preparation) and annual costs (maintenance and operations).

Investment Costs

An appropriate estimate for the planning and design expenses would be about 15 percent of construction costs. The right-of-way acquisition costs include the purchase price, legal, title, and other fees (2). The construction costs include labor, materials, equipment, and contractor overhead.

Annual Costs

Annual costs include maintenance costs (patching, striping, painting, etc.), replacements (e.g., pavement, resurfacing), and equipment upkeep. Operating costs include utility charges and traffic surveillance. The signal operating expenses are not included in project costs. Instead, they are considered benefits or disbenefits in the benefit/cost equation.

Step 9: Economic Analysis

The benefit/cost ratio method has been found to be an appropriate tool for the economic analysis of urban intersection improvements. The economic analysis includes (a) determination of present value of benefits (PVB) and present value of costs (PVC), as well as (b) benefit-cost analysis.

Determination of Present Values

Benefits and costs that occur at different times throughout the analysis period can be discounted with an appropriate interest rate to obtain present values. In the case study, a discount rate of 7 percent was used.

The steps in the AASHTO manual (2) for calculating annual benefits and costs are well-documented. First, estimate the rate of growth of annual value (assuming continuous compounding) by

$$r = \ln (a)/Y \qquad (15)$$

where

r = rate of growth of annual value (continuous compounding);
a = ratio of future benefits (final year) to early benefits (base year) or the ratio of Year 20 benefits to Year 0 benefits; and
Y = period of the estimate (20 years).

Next, calculate the present worth factor by

$$f = [\exp (r-i)n - 1]/(r - i) \qquad (16)$$

where

f = present worth factor,
i = discount rate (interest rate), and
n = analysis period (20 years).

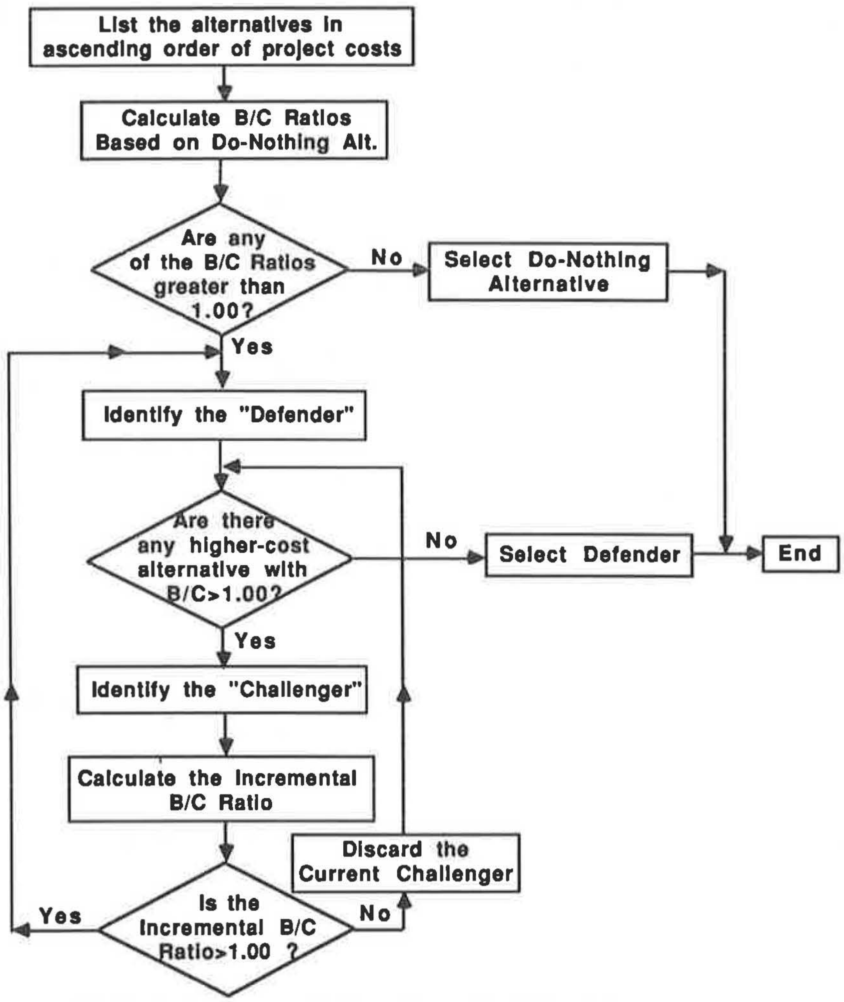
Then the present value is calculated as

$$PV = f * \text{first year's (Year 0) benefits} \qquad (17)$$

The use of the present value procedure is limited to determination of the present value of a stream of values that increase or decrease at an equal annual rate. For this reason, this simplified procedure cannot be used to determine the present value of isolated lump sum expenditures for project costs because the project costs occur irregularly over the planning period. In this case, the analyst can estimate these lump sum costs and the year in which the expenditure takes place. Then the future lump sum can be discounted back to the present value at the assumed interest rate to determine the present value of all project costs.

Benefit-Cost Analysis

After the present values of benefits and costs are calculated, the incremental benefit-cost analysis can be performed to select the optimum improvement alternative. The flowchart of the procedure that will be used for this purpose is shown in Figure 3.



"Defender": lower-cost alternative with B/C > 1.00
"Challenger": next higher-cost alternative with B/C > 1.00

FIGURE 3 **Incremental benefit cost procedure.**

Step 10: Examination of Staged Construction Options

At this step, the analyst will have to consider whether to implement the selected alternative immediately or whether to implement a less costly alternative now and the higher-cost alternative later. The LOS criterion can be utilized for this purpose and will measure how well the intersection will operate after the improvement until the end of the planning horizon.

CASE STUDY

To illustrate the practical use of the SPP, an existing intersection was analyzed. The intersection used was southwest 34th Street and southwest 2nd Avenue in Gainesville, Florida. This intersection experienced the highest accident rate in the city during 1986 and 1987.

The SPP step-by-step procedure applied and the intersection signalization (Figure 4) used in this case study are assumed to have the following characteristics. Three pretimed dials were in use:

- Dial 1 (90 sec per cycle) from 9:00 a.m. to 4:00 p.m. on weekdays,
- Dial 2 (110 sec per cycle) from 7:00 a.m. to 9:00 a.m. on weekdays,
- Dial 3 (110 sec per cycle) from 4:00 p.m. to 6:00 p.m. on weekdays.

A traffic count was taken in multiples of 15 min over the prime use hours of a representative day (7:00 a.m. to 6:00 p.m.). Five different alternatives to the existing conditions were selected for the purpose of analysis:

1. Add a northbound left-turn lane;
2. Change signal control of an existing condition from pretimed to actuated;
3. Same as alternative 2, but with an added northbound (NB) left-turn lane;
4. Same as alternative 2, but with an added westbound (WB) left-turn lane; and
5. Same as alternative 2, but with added NB and WB left-turn lanes.

The speed limit was 45 mph north-south (N-S), 35 mph east-west (E-W).

All data were input to SOAP and run to determine existing intersection measures of effectiveness. These measures include

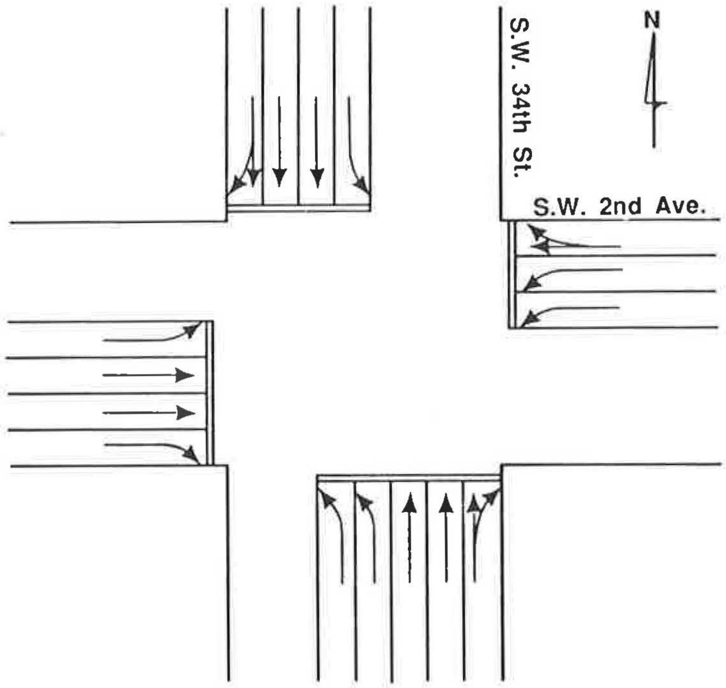


FIGURE 4 Alternative 5, southwest 34th Street and Second Avenue, Gainesville, Florida. Signal changed from actuated to pretimed; NB and WB left lanes added.

delay values, percentage of vehicles stopped, excess fuel consumption, maximum queue lengths, and volume to capacity ratio (V/C). Similar runs were performed for all the alternatives, and Table 5 presents SOAP output for Alternative 5. It must be emphasized that the projection of benefits 20 years into the future (used in the case study) may require more speculation and assumptions than can be justified. This problem can be reduced by using shorter intervals of 5, 10, 15, and 20 years.

Step 6 of the SPP was to calculate the user costs (Table 6). Unit value of travel time is assumed to be \$5.50/vehicle hour for both passenger cars and heavy vehicles; therefore, an adjustment for heavy vehicles is not necessary (i.e., h = 1.0).

Running cost factors for N-S and E-W approaches are obtained from reports by the Federal Highway Administration (*5*) and Ismart (*12*) and then weighted by traffic volumes:

	N-S	*E-W*
Speed limit (mph)	45	35
Running cost factor (\$/1,000 cycles)	25.8	17.5
Traffic volume (vehicles/day)	10,912	6,120

The weighted running cost factor is then found to be

$$f = [(10{,}912 * 25.8) + (6{,}120 * 17.5)]/17{,}032$$
$$= \$22.8/1{,}000 \text{ cycles}$$

For all user cost factors, an annual rate of increase of 5 percent was found to be appropriate to account for the effect of inflation.

Accident costs were calculated on the basis of historical accident records provided by the city of Gainesville. Accident cost calculations were based on the procedure described earlier. Table 7 presents the total benefits.

The project cost was estimated in Step 8 of the SPP. The cost of installing a new signal (\$14,000) is the average of the values obtained from several Florida traffic departments.

To determine the amount of right-of-way to be acquired, the required length and width of the left-turn lane must be known in each case. The required length of left-turn lane for each alternative is determined by using the maximum queue value

TABLE 5 SOAP OUTPUT FOR ALTERNATIVE 5 AFTER 20 YEARS: MEASURES OF EFFECTIVENESS

Movements	Delay (vehicle hours)	Stops (%)	Excess Fuel (gal.)	Left Turn Interference (number of vehicles)	Maximum Queue	*V/C Ratio*
NB Through	65.43	92.5	128.09		100.8	1.15
NB Left	19.75	99.3	33.18	14.6	24.4	1.15
SB Through	47.84	92.5	99.60		63.7	1.04
SB Left	5.78	99.7	8.73	7.0	6.0	1,000.00
EB Through	57.78	96.7	101.02		83.8	1.15
EB Left	4.31	99.7	6.74	3.7	4.2	1,000.00
WB Through	14.77	94.1	29.39		23.8	1.04
WB Left	15.79	99.5	26.09	10.3	26.5	1.15
Summary	231.45	94.6	432.84	35.6	100.8	1,000.00

TABLE 6 TOTAL DELAY COST CALCULATIONS

Alternative	Year	Volume (Veh/Day)	Total Delay (Veh-hr/day)	% Stops	Time Cost ($/year)	Running Cost ($/year)	Idling Cost ($/year)	Total Delay Cost ($/year)
0	0	17,032	145.03	88.8	291,148	109,587	38,566	439,301
1	0	17,032	113.61	87.5	228,072	107,983	30,211	366,266
2	0	17,032	69.31	93.7	139,140	115,635	18,431	273,205
3	0	17,032	64.14	94.1	128,761	116,128	17,056	261,945
4	0	17,032	63.38	93.2	127,235	115,017	16,854	259,107
5	0	17,032	60.32	93.1	121,092	114,894	16,040	252,027
0	20	30,762	454.80	95.2	2,422,490	563,009	320,887	3,306,386
1	20	30,762	395.99	93.2	2,109,239	551,181	279,393	2,939,813
2	20	30,762	354.27	93.4	1,887,017	552,364	249,957	2,689,338
3	20	30,762	285.99	94.9	1,523,324	561,235	201,782	2,286,341
4	20	30,762	294.78	92.6	1,570,144	547,633	207,984	2,325,761
5	20	30,762	229.98	94.6	1,224,987	559,461	162,264	1,946,712

TABLE 7 CALCULATION OF TOTAL BENEFITS

Alternative	Year	Total Delay Cost ($/year)	Total Acc. Cost ($/year)	Total User Cost ($/year)	User Benefits ($/year)	Signal Opr. Costs ($/year)	Signal Opr. Benefits ($/year)	Total Benefits ($/year)
0	0	439,301	157,100	596,401	---	117,000	---	---
1	0	366,266	114,872	481,138	115,263	117,000	0	115,263
2	0	273,205	140,872	414,077	182,324	101,000	16,000	198,324
3	0	261,945	102,838	364,783	231,618	101,000	16,000	247,618
4	0	259,107	102,838	361,945	234,456	101,000	16,000	250,456
5	0	252,027	76,256	328,283	268,118	101,000	16,000	284,118
0	20	3,306,386	752,847	4,059,233	---	310,436	---	---
1	20	2,939,813	550,484	3,490,297	568,936	310,436	0	568,936
2	20	2,689,338	675,080	3,364,418	694,814	267,983	42,453	737,267
3	20	2,286,341	492,815	2,779,156	1,280,076	267,983	42,453	1,322,529
4	20	2,325,761	492,815	2,818,576	1,240,657	267,983	42,453	1,283,109
5	20	1,946,712	365,430	2,312,142	1,747,091	267,983	42,453	1,789,543

provided in the SOAP output for Year 20 conditions. The calculations are as follows: 85 percent of maximum queue and 20 ft average headway between vehicles are found to be appropriate for design purposes. Alternative 5, in which signal control is changed from actuated to pretimed control and NB and WB left-turn lanes are added, is considered:

Maximum queue per lane (NB)	=	24.4/2 (Table 5)
	=	12.2
Required length of lane	=	(0.85)(12.2)(20) 207.4 ft (say, 250 ft)

∴ Design value: provide 250 ft of lane

The amount of right-of-way was calculated similarly for the other four alternatives. As a unit cost of land, including administrative and other related expenses, a value of $25/ft^2 was recommended by experts. The cost of right-of-way for Alternative 5 is

Lane width = 12 ft

(250 ft)(12 ft)($25/ft^2) + (250 ft)(12 ft)($25/ft^2)

= $150,000

The right-of-way costs were calculated similarly for the other four alternatives.

In Step 9 (Figure 1), the first part of the economic analysis was to determine the present value of annual benefits. The equations used earlier in Step 9 were applied with the following information to determine the present value of the stream of benefits for each alternative: discount rate (i) = 7 percent, analysis period (n) = 20 years, and period of the estimate (Y) = 20 years. The Year 0 and Year 20 benefits were taken from Table 7, and the results are shown in Table 8. Once the present values of benefits (PVB) and project costs (PVC) were determined, the incremental benefit-cost procedure could be applied (Figure 3).

In Step 10, staged construction options were examined for Alternative 5. As stated previously, this is a combination of signalization improvement (Alternative 2), addition of a northbound left-turn lane (Alternative 3), and addition of a westbound left-turn lane (Alternative 4). Alternative 5 was found to be the most economically justified improvement alternative for this particular intersection over a period of 20 years. Because three independent alternatives are included within the selected alternative, staged construction possibilities exist and should be examined. The traffic volumes in Table 9 and total delay values in Table 10 were used to calculate the average delay values for each alternative in 5-year intervals. These values were then used to prepare the average delay versus time graph shown in Figure 5.

The staged construction option for improving Alternative 5 to maintain LOS C is (from Figure 5)

- Year 0, Stage 1: Change signal control (Alternative 2 is accomplished).
- Year 12, Stage 2: Add a NB left-turn lane (Alternative 3 is accomplished).
- Year 15, Stage 3: Add a WB left-turn lane (Alternative 5 is accomplished).

Note that because the present value of benefits associated with Alternative 3 is higher than that for Alternative 4 (Table 11),

TABLE 8 CALCULATION OF PRESENT VALUES OF BENEFITS

Alternative	B_0 ($/year)	B_{20} ($/year)	a	r	f	PVB ($)
1	115,263	568,936	4.94	0.080	22.14	2,551,923
2	198,324	737,267	3.72	0.066	19.22	3,811,787
3	247,618	1,322,529	5.34	0.084	23.08	5,715,023
4	250,456	1,283,109	5.12	0.082	22.6	5,660,306
5	284,118	1,789,543	6.3	0.092	25.12	7,137,044

TABLE 9 TRAFFIC VOLUMES AT STUDY YEARS

	Year				
	0	5	10	15	20
Growth factor	1.000	1.159	1.344	1.558	1.806
Traffic volume (vehicles/day)	17,032	19,745	22,890	26,535	30,762

NOTE: Growth rate: 3 percent/year.

TABLE 10 TOTAL DELAY VALUES

	Total Delay Values (vehicle hours/day)				
Alternative	Year 0	Year 5	Year 10	Year 15	Year 20
0	145.03	185.17	242.56	342.35	454.80
1	113.61	162.39	212.66	289.22	395.99
2	69.31	96.90	138.43	232.72	354.27
3	64.14	82.94	120.40	184.10	285.99
4	63.38	82.31	119.01	184.48	294.78
5	60.32	75.24	99.57	149.90	229.98

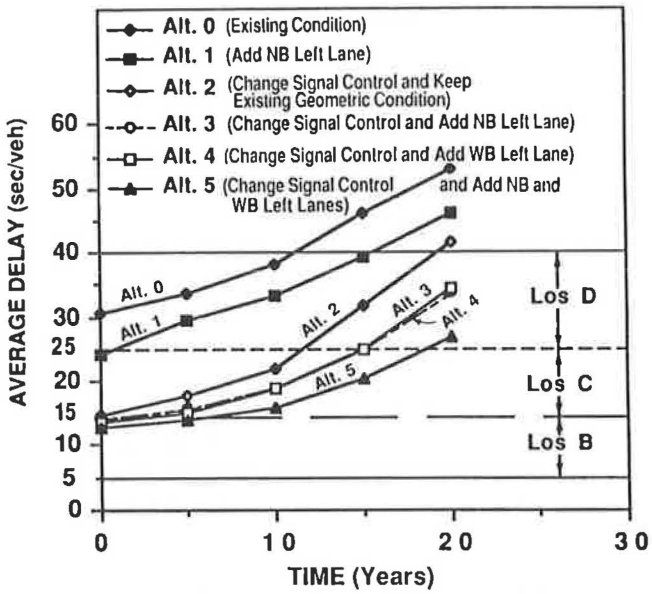


FIGURE 5 Average delay versus time graph for examination of staged construction options.

the choice is made for Alternative 3 at Stage 2. By completion of Stage 3 at Year 15, Alternative 5 will be automatically accomplished. Although the LOS C requirement is not satisfied for the last 2 years of the planning period, this improvement plan is still acceptable. However, further improvement of the intersection has to be planned for Year 20.

TABLE 11 *B/C* RATIOS

Alternative	PVB	PVC	*B/C*
1	2,551,923	101,000	25.3
2	3,811,787	14,000	272.3
3	5,715,023	130,000	44.0
4	5,660,306	130,000	43.5
5	7,137,044	241,000	29.6

The results presented in Table 11 are produced by the following incremental *B/C* procedure. The first defender is the lowest-cost alternative (Table 12), which is Alternative 2.

Defender = Alternative 2 (the lowest-cost alternative with $B/C > 1.0$).

Challenger = Alternative 1 (next higher-cost alternative with $B/C > 1.0$).

$$\begin{aligned} \text{IBCR}_{1\text{-}2} &= (B_1 - B_2)/(C_1 - C_2) \\ &= -1{,}259{,}864/87{,}000 \\ &= -14.5 < 1.0 \end{aligned}$$

Therefore eliminate Alternative 1:

Challenger = Alternative 3

$$\begin{aligned} \text{IBCR}_{3\text{-}2} &= (B_3 - B_2)/(C_3 - C_2) \\ &= 1{,}903{,}236/116{,}000 \\ &= 16.4 > 1.0 \end{aligned}$$

Therefore, eliminate Alternative 2:

Defender = Alternative 3

Challenger = Alternative 4

Because Alternative 3 and Alternative 4 are equal-cost alternatives, the one with higher benefits is preferred; therefore eliminate Alternative 4:

Challenger = Alternative 5

$$\begin{aligned} \text{IBCR}_{5\text{-}3} &= (B_5 - B_3)/(C_5 - C_3) \\ &= 1{,}422{,}021/111{,}000 \\ &= 12.8 > 1.0 \end{aligned}$$

Therefore eliminate Alternative 3 and select Alternative 5.

CONCLUSIONS AND RECOMMENDATIONS

The main objective of this research was to develop a simple and practical step-by-step procedure to improve urban ISIs at grade. The SPP developed in this research will aid engineers and planners in developing intersection improvement options to deal effectively with present and future problems at 5-year intervals or over the long-range 20-year planning horizon. The SPP generates alternative solutions that take into account safety, operational, and economic considerations. From the results of the application of the SPP, engineers and planners will be able to select and schedule desired improvement plans at a fixed future date (stage construction). For instance, average delay versus time (in 5-year intervals until the 20-year planning horizon is reached) could be plotted graphically for each improvement alternative.

TABLE 12 PRESENT VALUES OF PROJECT COSTS

	PVC ($)				
Alternative	Equipment	Construction	Right of Way	Maintenance	Total
1	–	35,000	60,000	6,000	101,000
2	14,000	–	–	–	14,000
3	14,000	35,000	75,000	6,000	130,000
4	14,000	35,000	75,000	6,000	130,000
5	14,000	65,000	150,000	12,000	241,000

From the results, the analyst will be able to determine, in advance, details of right-of-way acquisition and needs for additional lanes or other types of improvements that might be desired. Such preplanning will help avoid such problems as excessive payments for business damage.

SOAP is incorporated into the SPP, along with a cost-benefit technique. The SPP is flexible and is applicable to most isolated signalized intersections. As with any other technique, the results are only as valid as the input data (e.g., future costs of fuel, right-of-way, construction, users' fees, discount rates, accident costs, etc.). Other uncertainties include future traffic growth, traffic distribution, accident rate, and so on. Because of the SPP's step-by-step format, computerization of the procedure is recommended. This will enable the user to generate additional alternatives (or combinations of alternatives) that could be analyzed and implemented over shorter time intervals. The SPP can be incorporated into TRANSYT-7F or NETSIM analysis. This combination is particularly recommended for cases in which the intersection under consideration for improvement is influenced by neighboring intersections.

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Guiderail Delineation

JOHN S. CAMPI, JR.

This paper investigates delineating guiderails and evaluates the performance of different types of guiderail delineation under a variety of field conditions. A thorough search of literature on the topic determined that virtually no research has been conducted previously on the delineation of guiderails. A determination of the various benefits that could be passed on to the motoring public through delineation of the guiderail is discussed. The installation procedures and the labor involved for each type of delineator or delineation treatment are also discussed. The effects of soil and dirt accumulation on guiderail delineators were measured under different environmental conditions at different geographical locations. Information taken from the results of an actual behind-the-wheel driver evaluation survey revealed that motorists generally respond favorably to guiderail delineation. The selection of an appropriate device for delineating guiderails was based on various performance-related requirements that the device or reflective treatment had to meet. The criteria used for selecting a device for delineating guiderail were ease of installation, resistance to soil, durability, and cost. An improvement in the nighttime visibility of guiderails through delineation should result in a reduction in guiderail accidents, which would help to offset the initial cost of delineation.

The state of New Jersey has approximately 1,039 mi (5,485,920 ft) of guiderails on its state-maintained highway system. The predominant type of guiderail used on New Jersey's state highway system is zinc-galvanized steel W-beam, of which there are some 934 mi. About 75 mi of older cable-and-wood-post guiderail is also present on the state highway system. The steel W-beam guiderail is used on all new installations and is gradually replacing the aging cable guiderail.

More than one variety of guiderail may exist at the county and municipal level. Some installations may include box-beam guiderails or an older version of the W-beam guiderail that may be flared at the top and bottom sections.

Before this research study on guiderail delineation was conducted, a guiderail visibility needs analysis report was drafted (*1*). In this report, attempts were made to to determine the benefits and advantages that guiderail delineation could bring to the motoring public. A reduction in accidents involving guiderails could be one of these benefits. Table 1, which was compiled with information provided by the National Safety Council, presents costs that are characteristic of accidents.

The FHWA classifies guiderails as a typical fixed object hazard (*2*). Reflectors or delineators can make guiderails more conspicuous during nighttime hours. Enhancing the nighttime visibility of guiderails should increase their detectability and recognition by motorists. Table 2 presents guiderail property damage accidents, fatalities, and injuries for nondaylight conditions during a 3-year period in New Jersey. By using accident data from 1983 to 1984, it was determined that the proportion of total fixed object accidents that involved guiderails increased at night, especially during wet nights.

By using the information supplied in Tables 1 and 2, a total cost of $2,520,000 can be attributed to fatal guiderail accidents from 1982 to 1984. Guiderail injury accident costs amounted to $7,026,200 during this period. Property damages related to guiderail accidents for the same period account for a total cost of $1,282,250 (Table 3). Guiderail-related accident severity and frequency have a direct influence on guiderail repair costs and maintenance. The guiderail repair cost for 1982–1984 on New Jersey state highways was $1,190,133. The total cost figure for guiderail accidents and repair costs for the 1982–1984 period was $12,018,585, or about $4,000,000 per year.

TABLE 1 CHARACTERISTIC ACCIDENT COSTS

Type of Accident	1983 Cost Values ($)
Fatal accident	210,000
Injury accident	8,600
Property damage accident	1,150

TABLE 2 GUIDERAIL ACCIDENTS, FATALITIES, AND INJURIES ON NEW JERSEY STATE HIGHWAYS, 1982–1984

	1982	1983	1984
Fatalities	4	4	4
Injuries	311	255	251
Property damage accidents	372	384	359

TABLE 3 GUIDERAIL ACCIDENT COSTS, 1982–1984

Type	Cost ($)
Fatalities	2,520,000
Injuries	7,026,200
Property damage	1,282,250
Total	10,828,452

Delineating all of the guiderails on the state's highway system would cost about $1,280,000. If a 5-year lifetime is assumed for the delineators, the yearly cost would be $256,000. A 6.4 percent reduction in accidents over a 5-year period would offset the cost of delineation. The cost figure for delineating state-maintained guiderails could be reduced appreciably by development of criteria for delineating guiderails that would suggest when guiderails should be delineated.

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There are many relatively new devices and methods that can be used to delineate guiderails under low light conditions. Most of the devices are intended to increase nighttime visibility of the guiderails. Modern guiderail delineators may utilize various types of reflective sheeting (e.g., encapsulated bead sheeting, cube corner sheeting, etc.) or acrylic prismatic reflectors as their primary reflective component.

Many types of guiderail delineators are mounted to the post bolt of the guiderail. There are also a number of guiderail delineators that affix to the guiderail with an adhesive. Delineator posts that are independent of the guiderail were also evaluated to determine if they could serve as suitable guiderail delineators.

STUDY DESIGN

Five guiderail delineation test sites were selected throughout the state of New Jersey for evaluation and monitoring of 19 different guiderail delineators under a variety of field conditions. One test site was located in northeastern New Jersey, in an area where environmental conditions are relatively severe. Soil, dirt, and oil film accumulate at an accelerated rate at this site. These conditions can provide insight about the effects of dirt and soil on guiderail delineation. Another three guiderail delineation test sites were located in central New Jersey. The final guiderail delineation test site was in southeastern New Jersey, near the coast. One of the reasons for selecting this site was to ascertain the effects that a saltwater environment may have on guiderail delineation. Additionally, because pedestrian traffic is fairly common at this and one of the central New Jersey sites, problems relating to vandalism were investigated at both locations.

Originally, 12 different types of guiderail delineators or delineation treatments were installed at all five test sites during December 1983. Most of the 12 original delineators consisted of devices that mounted in the W-beam of the guiderail. As second- and third-generation guiderail delineators became available, they were installed at the test sites along with the remaining original devices. A majority of the second- and third-generation devices were installed on the top portion of the guiderail or on the top of the guiderail post itself.

All five guiderail delineation test sites were monitored on a monthly basis. The two sites that were subjected to pedestrian activity were monitored on a biweekly basis during the summer months. Five different descriptions or categories of dirt covering were created to indicate the surface condition of each individual device. Table 4 lists the surface description nomenclature that was used while the devices were being monitored. In addition to rating the surface condition of each device or delineator, the physical characteristics (i.e., damage, cracking, chipping, etc.) of each device were also recorded while the test sites were monitored. The first generation of devices was field tested for 38 months. Second-generation devices were field tested for 31 months, and third-generation devices were tested for 12 months.

TABLE 4 NOMENCLATURE FOR SURFACE DESCRIPTIONS

Surface Description	Reflective Surface Area Concealed by Dirt or Soil (%)
Clean	0–19
Light	20–39
Moderate	40–59
Heavy	60–79
Covered	80–100

RESULTS OF THE FIELD DURABILITY STUDY

During the evaluation and durability phase of the project, 22 different guiderail delineators were field tested at five sites. Each site consisted of five or more subdivided groups with at least 12 delineators in each group. The delineators were arranged in succession in the first group, and this arrangement was then repeated in the four following groups.

Guiderail Delineation in the W-Beam of the Guiderail

The majority of delineators that were installed initially at all of the test sites were mounted in the W-beam of the guiderail. Usually, delineators that mount in this location are attached to the guiderail by a post bolt, but a few are held in place with adhesives.

Guiderail delineators that attach behind the post bolt of the guiderail can be difficult to install. When the post bolt of the guiderail is loosened to accept the delineator, the entire bolt assembly may turn together as one, making the installation process very difficult. At older sections of guiderail, which may not be zinc-galvanized, some of the post bolts may be fused to the locking nut.

Two models of a plastic, trapezoid-shaped guiderail delineator that mounts in the W-beam of the guiderail with adhesive were also field tested. The plastic outer portion of this device experienced cracking and severe breakage at the field evaluation sites (Figure 1). Installation of this particular delineator is more involved than that of some others because the surface of the mounting area must be prepared, and the outdoor temperature must be above 40°F to permit the adhesive to be dispensed easily from the tube.

A treatment of white paint and glass beads in the middle of the W-beam of the guiderail was field tested at each test site.

FIGURE 1 Stimsonite acrylic guiderail delineator with broken casing and face.

The paint and glass bead treatment requires a time-consuming surface preparation of the involved area of the guiderail with a steel brush. Another disadvantage of the paint treatment is its poor visibility on tangent sections of guiderail that are aligned parallel to the road edge.

Information from field evaluations and inspections indicates that guiderail delineators mounted in the W-beam accumulate about 23 percent more dirt film than guiderail delineators that mount on top of the guiderail post. The results of this comparison were shown to be significant at the 99 percent confidence level when a *t*-test was performed. Once a delineator in the W-beam is heavily soiled, it is unlikely that the delineator will be sufficiently cleansed by rain because the delineator is shielded by the top portion of the guiderail (Figure 2). Figure 2 demonstrates that delineators mounted both inside and above the W-beam of the guiderail accumulated soil at the same rate for a period of six months. Delineators inside the W-beam remained at or above this level for the next 12 months, while the soil accumulation level for delineators mounted above the guiderail decreased. Delineators mounted in the W-beam of the guiderail become inoperable when snow is pushed against the guiderail during snowplowing operations (Figure 3). Figures 4 and 5 present the percentages of missing and damaged delineators in the test group of delineators that may be used inside the W-beam of the guiderail.

Guiderail Delineators Mounted on Posts Independent of the Guiderail

Two different types of guiderail delineators that attach to steel U-posts with metal rivets were evaluated in the field. One of the delineators tested consisted of an aluminum panel with a face of reflective sheeting. No major problems relating to vandalism or dirt collection were experienced with the reflective panel portion of this device in the field. Another post-mounted delineator evaluated during the field study utilized an acrylic reflective face. Over an 18-month field evaluation period, 43 percent of the acrylic-faced reflectors were damaged and 22 percent of the devices were missing or stripped from the steel supporting posts. (Reflector damage refers to a cracked, broken, or impaired reflector that may still remain functional.)

The steel U-posts supporting both types of delineators were installed independent of the guiderail, behind the guiderail support post. Installing the steel U-post units is a relatively

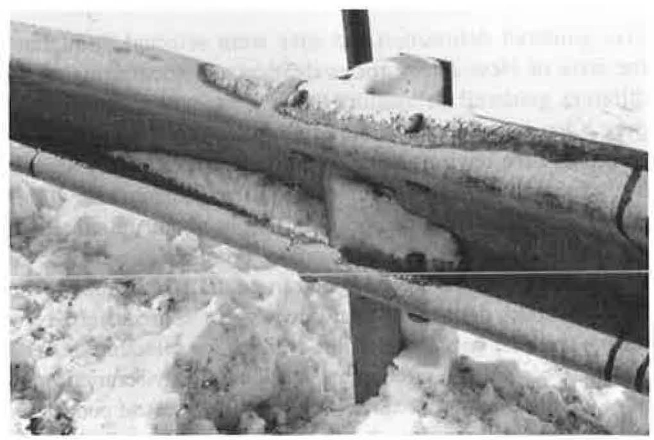

FIGURE 3 Transpo (triangular) delineator mounted inside W-beam of guiderail covered with snow and ice.

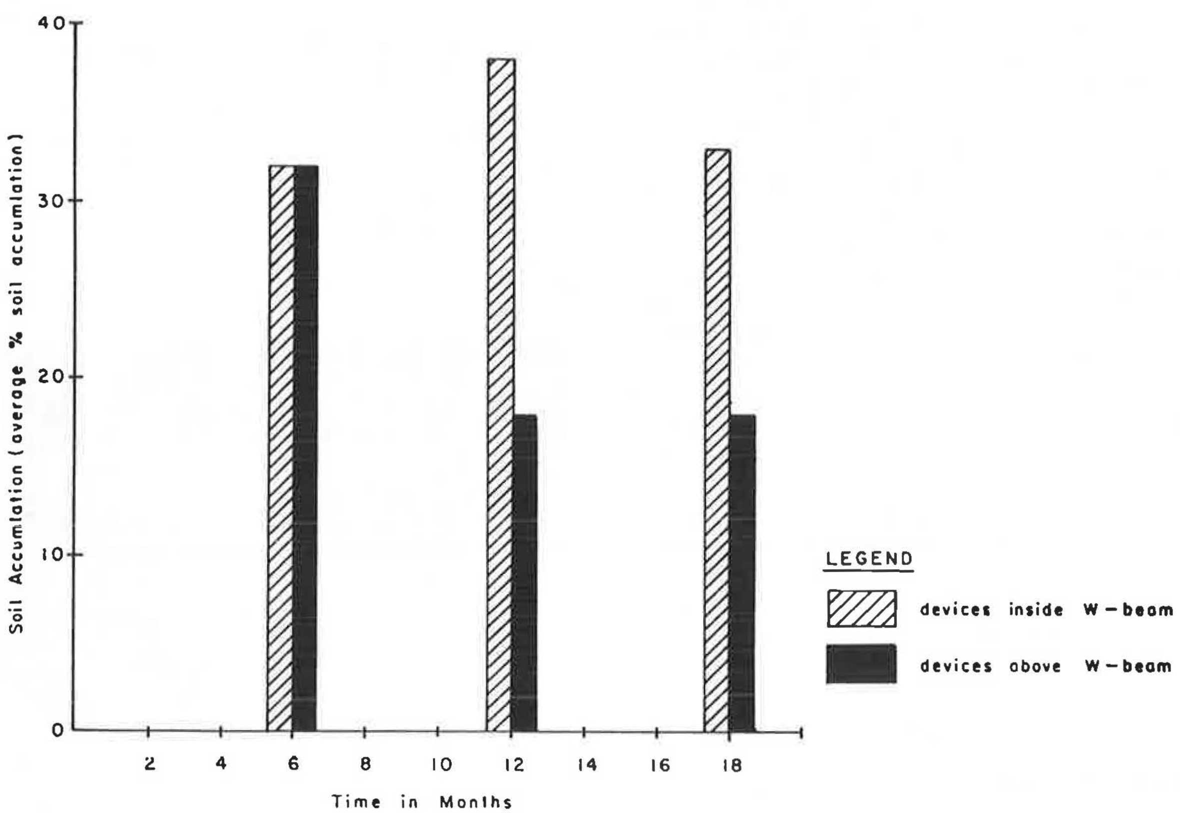

FIGURE 2 Soil accumulation of guiderail delineators inside and above the W-beam.

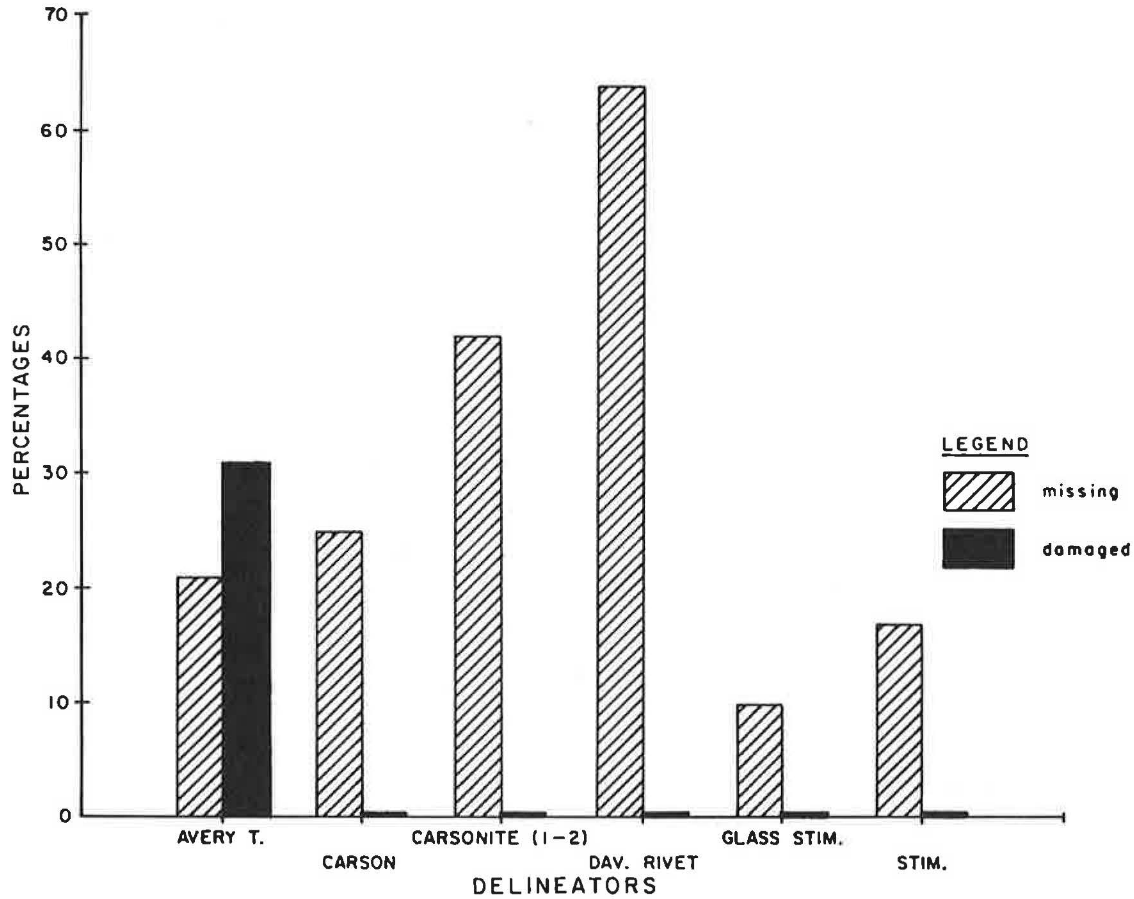

FIGURE 4 Adhesive-mounted delineators found missing or damaged.

strenuous task that requires the use of a large and heavy sledgehammer or slidehammer. In colder weather, the ground often becomes hard, making the installation of the metal post even more difficult. The placement and angle of each delineator post should be determined by the vehicle location and position. A delineator post that is installed improperly, at an incorrect angle to a vehicle's headlights, may be virtually useless. Delineators that are attached to the guiderail are more likely to be placed in the proper orientation to the view of the motorist. Another problem associated with the delineator posts is their vulnerability to lawn mowing and maintenance equipment.

The cost of the galvanized steel U-post, the reflector, periodic maintenance, and the labor involved in installation make post-mounted delineators unattractive for use as guiderail delineators. Delineators that attach directly to the guiderail system eliminate the additional expense and need for an independent mounting post.

Guiderail Delineation on Top of or Above the W-Beam of the Guiderail

A variety of guiderail delineators that attach to or mount on the top portion of the W-beam or on top of the guiderail spacer bracket were also field tested. The delineators that mount on top of or above the guiderail were attached with screws, rivets, or adhesives. If the delineator is to be attached to the guiderail with screws or rivets, a hole must be drilled or punched in the guiderail or post. Drilling these holes requires an electrical power source and equipment, and the whole process demands more effort than attaching the delineators with adhesive or a bracket mounting system.

One of the problems associated with attaching the guiderail delineators with adhesive is the possibility of vandalism occurring in areas that are frequented by pedestrians. Field inspections of some of the delineators that were attached with adhesive revealed instances of the adhesive cracking and separating from the surface of the guiderail. This cracking or damage to the adhesive weakens the adhesive bond between the delineator and the guiderail and makes the delineator more susceptible to vandalism or stress from turbulence. Figure 6 shows an example of damage to adhesive on a guiderail delineator. A manufacturer of one of the adhesives does not recommend application of the material in temperatures below 40°F. In some geographic areas this restriction could delay the installation of delineators for months at a time.

Two types of reflective material that attach to the top bend of the guiderail were field tested. Treatments of paint and glass beads were field tested in this configuration, but there were installation and visibility problems. The glass beads that were applied over the painted surface were not distributed uniformly, compromising the reflective quality of the treatment (Figure 7). Pressure sensitive reflective tape was also evaluated in the field. The tape was difficult to handle during installation and did not adhere well to the cold surface of the guiderail in low temperatures.

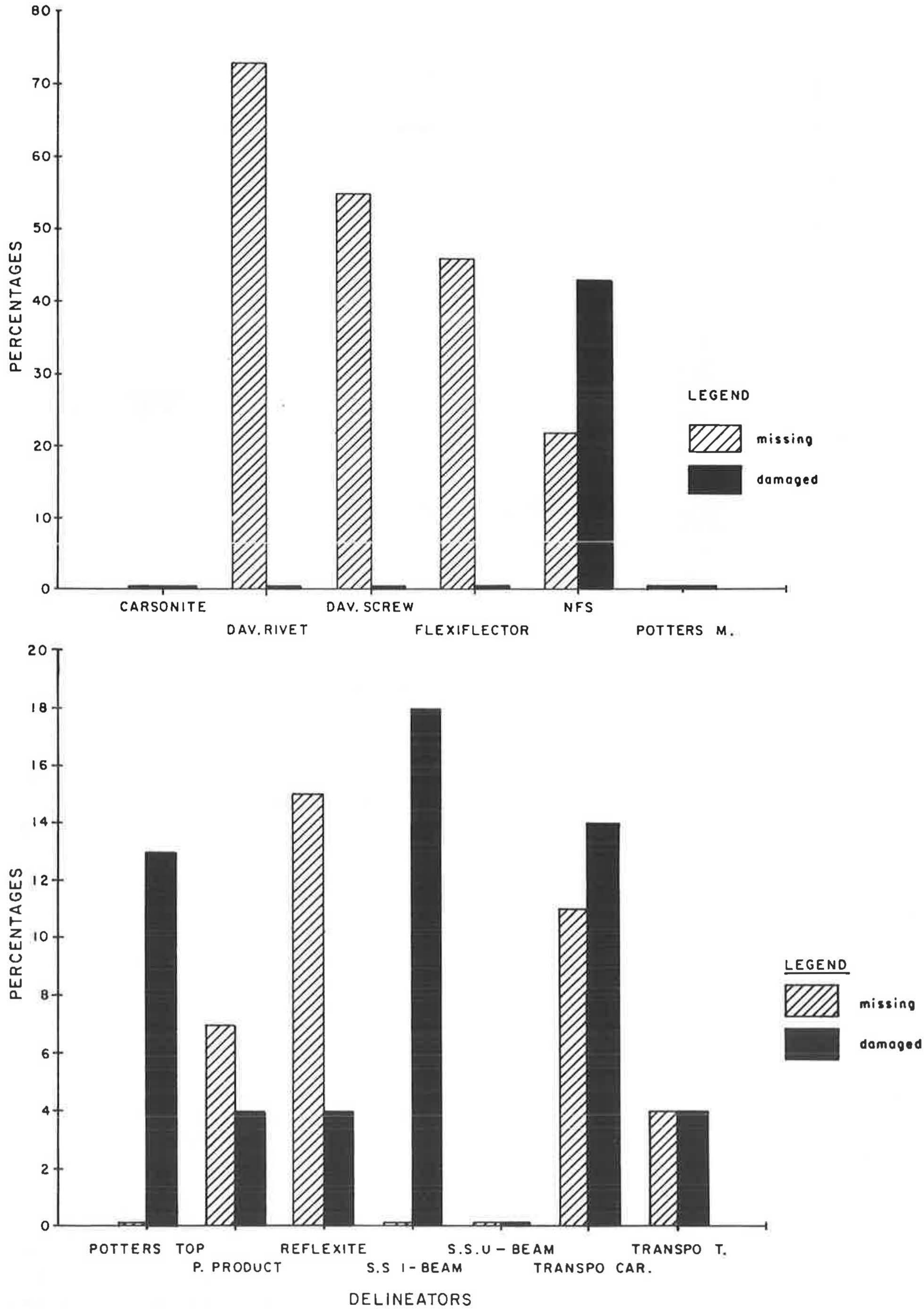

FIGURE 5 Bolt-on and bracket-mounted delineators found missing or damaged.

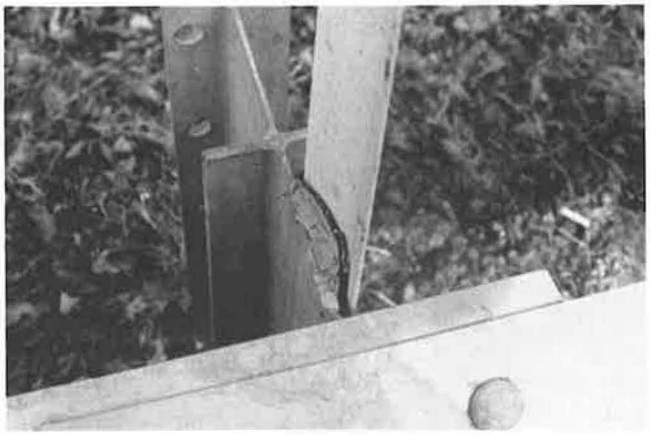

FIGURE 6 Carson delineator panel suffering from cracked and separated adhesive mounting.

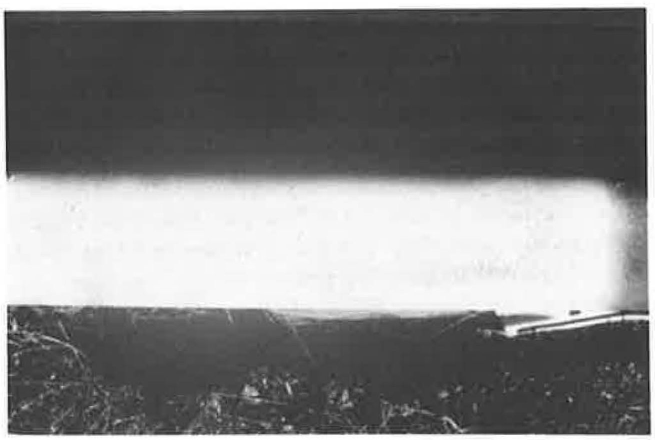

FIGURE 7 Potter's top paint treatment with an uneven application of glass beads over the painted surface.

Bracket-Mounted Guiderail Delineation

A unique two-part guiderail delineator system, which mounts on top of the guiderail post or spacer bracket, was evaluated at each test site. This two-part delineation system consists of a flexible panel and a metal bracket that is secured to the guiderail support post by one or more self-contained bolts. Installation of this delineator is quick and uncomplicated. The only tool required for installation is a small open-end or Allen wrench, depending on which type of bolt is used.

The bracket-mounted guiderail delineators performed well in the field. During an 18-month field evaluation period, none of the devices were lost or damaged at any of the five test sites. None of the bracket-mounted delineators showed any signs of vandalism after this test period.

The 1977 *Guide for Selecting, Designing and Locating Traffic Barriers* (*4*) regulates the material and dimensional characteristics for guiderail installations. Attaching the delineator to a uniform guiderail structure assures a consistent delineator installation. Variables such as placement, offset, and spacing of the delineators can be kept constant by attaching them to the guiderails.

When the bracket-mounted delineator is attached to the guiderail post, the reflector face usually appears to be perpendicular to the roadway; thus a consistent angle of incidence throughout the length of the guiderail is achieved. The mounting height of this particular delineator conforms to the *Manual on Uniform Traffic Control Devices* (MUTCD) (*5*) standards that require roadside delineators to be 4 ft above the near roadway edge.

The height and flexibility of this delineator is an asset, especially during the winter months. As snow is plowed and forced against and above the height of the guiderail, the panel of the delineator system usually remains visible. New Jersey standard specifications require the top of W-beam guiderails to be $27^5/_8$ in. above ground level. The flexible quality of the reflectorized panel enables this delineator system to rebound and to withstand snow and ice that may be hurled from nearby snowplows. This flexibility contrasts with the behavior of certain rigid post-mounted delineators evaluated at the field test sites, which had a tendency to be displaced from their original vertical position. This problem would require periodic maintenance to provide optimum performance.

DRIVER EVALUATION STUDY

The impressions and opinions that motorists have about guiderail delineation was surveyed at six different test sites in the local Trenton, New Jersey, area. Only members of a small segment within the author's immediate divisional group were available as participants for this survey. A limitation on project funds was also a factor in restricting the survey size. The results collected at the test sites are summarized in the following paragraphs.

In a comparison of motorists' responses to field test sites, both with and without delineation treatments, 11 of 18 responses indicated that guiderail delineation was beneficial to drivers. Only 3 of 18 responses showed a decrease in driver responses between the before and after test sites.

Guiderail delineation was shown to be useful in determining the available shoulder space on the roadway. Table 5 presents the responses of participants to the question of whether the guiderail made it easier to determine the usable space of the roadway. The percent responses both before and after delineators were added to the guiderail are given. Recognition of the usable space off the roadway increased at five field test sites after delineators were added to the guiderail.

TABLE 5 RECOGNITION OF SHOULDER SPACE BEFORE AND AFTER DELINEATION

	Test Sites					
	1	2	3	4	5	6
Before delineation (%)	0	67	18	67	10	50
After delineation (%)	27	80	55	83	30	67

No difference before and after delineation was indicated in 4 of a total of 18 driver responses. When delineated guiderail was compared to nondelineated guiderail, it was rated more effective in emphasizing roadway alignment and the road edge at all six test sites. Participants in the survey indicated that delineation of the guiderail was helpful at 4 of the 6 test sites.

In summary, the results from the driver evaluation sites show that guiderail delineation can benefit the motorist through an increase in driver comfort. The results of the survey also suggest that motorists have a high opinion of and support delineation of the guiderail.

SUMMARY

Guiderails are usually installed on highways as a means of protecting motorists from objects or situations that are more hazardous than the guiderails themselves. From 1982 to 1984, guiderail-related accident and repair costs totaled over $12 million on New Jersey state highways.

One objective of this study was to ascertain whether there is a need to delineate guiderails. Determination of a suitable device for delineating guiderails was another objective of the study. A variety of guiderail delineators that mount in the W-beam, on the top of the guiderail, and above the guiderail were field tested. Over a 38-month-long evaluation period, delineators mounted inside the W-beam of the guiderail accumulated more soil than delineators mounted above.

The installation procedure for most guiderail delineators that mount inside the W-beam is labor intensive. The attachment of guiderail delineators with adhesive is unreliable because the adhesive can fail with time. Delineators mounted with adhesive were also vulnerable to damage at locations frequented by pedestrians.

The results of a driver effectiveness study revealed that recognition of the guiderail system increased 16 percent after delineators were added. Of those surveyed, 88 percent rated delineated guiderails as more effective than conventional guiderails in emphasizing roadway alignment and the road edge.

Enhancing the nighttime visibility of guiderails through delineation can increase the detectability and recognition of guiderails. Early detection and identification of guiderails can allow more time for drivers to perform hazard-avoidance maneuvers. Delineation of guiderails could thus help to improve driver comfort during nighttime driving.

CONCLUSIONS

After more than 20 different types of guiderail delineators were evaluated in the field, it was determined that a flexible panel and metal bracket system manufactured by the Carsonite Company was the most suitable device with regard to durability, soil accumulation, and ease of installation. The flexible panel of this system utilizes a face of reflective sheeting, which is a material that has been approved by the New Jersey Department of Transportation. This delineator is one of the few tested that also conforms to MUTCD specifications requiring that the reflector heads of roadside delineators be 4 ft above ground level.

A full investigation of the topic of delineator spacing was beyond the scope of this study. Information that was obtained through driver demonstration sites and a survey of other state practices in guiderail delineation suggests that delineators on curves should be spaced at 37.5-ft intervals (the distance of six guiderail posts spaced 6.25 ft apart) and that delineators on straight sections of guiderail posts should be spaced 75 ft apart (12 guiderail posts spaced 6.25 ft apart). This spacing arrangement is similar to the New Jersey specifications governing the spacing of snowplowable raised pavement markers, which require markers to be spaced 80 ft apart on tangent sections of road. On curves of 3° or greater, markers are placed at 40-ft intervals in accordance with the specifications. Guiderail delineators could be installed on the terminal ends of guiderails, especially those that may lack breakaway cable terminals, in an effort to enhance their visibility.

ACKNOWLEDGMENTS

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Delineation of Concrete Safety Shaped Barriers

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In this paper, the results of a study of five delineation treatments for concrete safety shaped barriers are presented. These treatments were tested along a lighted urban freeway in Houston, Texas. A low-light video camera and time-lapse video recorder were mounted above each treatment to record nighttime traffic next to the barrier before and after the treatments were installed. Nighttime subjective evaluations were conducted when the treatments were newly installed and also after the treatments had been in place for several months and had become dirty. Study researchers also measured the visibility distances of the treatments at periodic intervals after delineation installation. The results showed that the treatments had little effect on lane distributions and vehicle lateral distances from the barrier. Subjects rated the side-mounted cube-corner lenses at 50-ft spacings as the brightest and most effective treatment of those studied. However, lane straddling rates may have increased slightly next to this treatment. Visibility data showed that the cube-corner lenses lost less of their original visibility over time than did reflective sheeting. Also, side-mounted delineation was found to become dirty and lose its visibility faster than top-mounted delineation. On the basis of the measurements taken, top-mounted cube-corner delineators at spacings no greater than 200 ft were recommended for delineating concrete safety shaped barriers.

Concrete safety shaped barriers (CSSBs) are being used more and more on highway facilities to protect drivers from roadside hazards, to separate opposing traffic flows, and to protect workers from traffic during roadway rehabilitation and reconstruction activities. At many of these installations, the barrier must be placed immediately next to the travel lane. In these instances it is important that drivers be aware of the location of the barrier and the proper travel path next to the barrier.

Unfortunately, CSSBs may be quite difficult to see at night, especially in the rain. Their concrete composition provides little contrast with the roadway pavement. This problem may occur even where fixed illumination is provided. To further complicate matters, barriers tend to accumulate dirt and trash next to them, possibly obscuring the adjacent travel lane edgeline partially or completely. It is believed that barrier-mounted delineation could be extremely useful to drivers in some cases, identifying both the location of the barrier and the correct travel path next to the barrier. Such delineation could result in improved safety, operations, and driver comfort under nighttime driving conditions.

Previous CSSB delineation research has been limited. Most studies have considered only subjective driver evaluations of various delineation treatments (*1–3*); few have collected objective driver performance data, either in a controlled field or actual field situation (*4, 5*). The majority of the studies have focused on work zone CSSB delineation (*2–5*) because geometric and visibility constraints are generally more severe at such locations. The results of these studies have been mixed. For example, one study suggests that delineation should be mounted on top of the barrier (*1*) so that it will retain its reflectivity longer and require less maintenance. On the other hand, another study recommends side-mounted CSSB delineation so that the delineators are not "hidden" by oncoming headlight glare (*3*). Larger but less bright (as measured by the specific illuminance) devices are recommended by some (*2, 4, 5*), while smaller, brighter reflectors are recommended by others (*1, 3*). Even the spacing of delineation is not without debate: distances recommended in the various studies have ranged from 25 to 200 ft.

Engineers must currently decide on the type of delineator to use, how far apart the delineators should be spaced, and where on the barrier the delineators should be placed without knowledge of the impacts that these choices have on traffic operations and safety. In addition, the effects that road film and grime have on the continued effectiveness of delineation are unknown. To address these questions, the Texas Transportation Institute conducted a study for the Texas State Department of Highways and Public Transportation to develop improved procedures for delineating concrete safety shaped barriers (*6*). The results of this research are summarized in this paper.

The specific objectives of this study were threefold:

- Determine how different delineator types, spacings, and mounting positions on the barrier affect nighttime traffic operating in the travel lane next to the barrier;
- Determine driver preference and perception of different delineator types, spacings, and mounting positions; and
- Determine how the visibility and brightness of different types of delineators deteriorate over time because of dirt and road film.

These objectives were addressed through the collection and analysis of (a) driver performance data, (b) subjective evaluations, and (c) reflectivity measurements of selected delineation treatments taken over a period of time.

STUDY DESIGN

Delineation Treatments

This research was designed to evaluate a select number of different delineator types, spacings, and mounting positions in

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a coherent, consistent manner. On the basis of the literature review, it was decided to limit the analysis to three different types of delineators:

- A round (3.25 in. diameter) acrylic cube-corner reflector,
- A small plastic bracket (about 3 in. high and 4.25 in. wide) covered with high-intensity (HI) sheeting, and
- A cylindrical tube (3 in. in diameter by 6 in. high) wrapped with HI reflective sheeting, thereby providing reflectivity at all viewing angles.

The study also considered both top-mounted and side-mounted (6 in. from the top) positions on the CSSB. As a final factor, two spacings were selected for study, at 50 ft and 200 ft.

A block experimental design to evaluate these different factors would have required 12 (3 × 2 × 2) different delineation treatment combinations. Because of limitations in study funding and scope, a quasi-Latin square design was used to select five combinations of delineator type, spacing, and mounting position on the CSSB for analysis. These treatments are given in Table 1.

These treatments were installed along a 3-mi section of urban freeway (illuminated with high-mast lighting) in Houston, Texas. A high-occupancy vehicle transitway was retrofitted in the median of the freeway, with CSSBs (located 1 ft away from the inside travel lanes) used to separate the transitway from the travel lanes. The layout of the treatments through this section is shown in Figure 1. The freeway section was primarily four lanes in each direction, with each lane approximately 12 ft wide. On the basis of 1985 data, traffic flow through the section was considered to be 180,000 vehicles per day. A number of businesses were located on the frontage roads on each side of the freeway. The signs and lights of these businesses added to the general nighttime visual complexity of the section. A gently rolling freeway alignment provided substantial sight distance throughout.

Data Collection

Driver Performance Data

Immediately before and after the delineators were installed, nighttime driver performance data were collected at each treatment

TABLE 1 SUMMARY OF DELINEATION TREATMENTS

Treatment	Delineator	Mounting Position	Spacing (ft)	Cost per Delineator ($)	Cost per Mile of Barrier ($)
1	Cube-corner	Top	200	2.50	66
2	Cube-corner	Side	50	2.50	264
3	Brackets with HI sheeting[a]	Top	50	1.50	158
4	Brackets with HI sheeting	Side	200	1.50	40
5	Reflective cylinder	Top	50	4.50	475

[a]HI = high-intensity reflective sheeting.

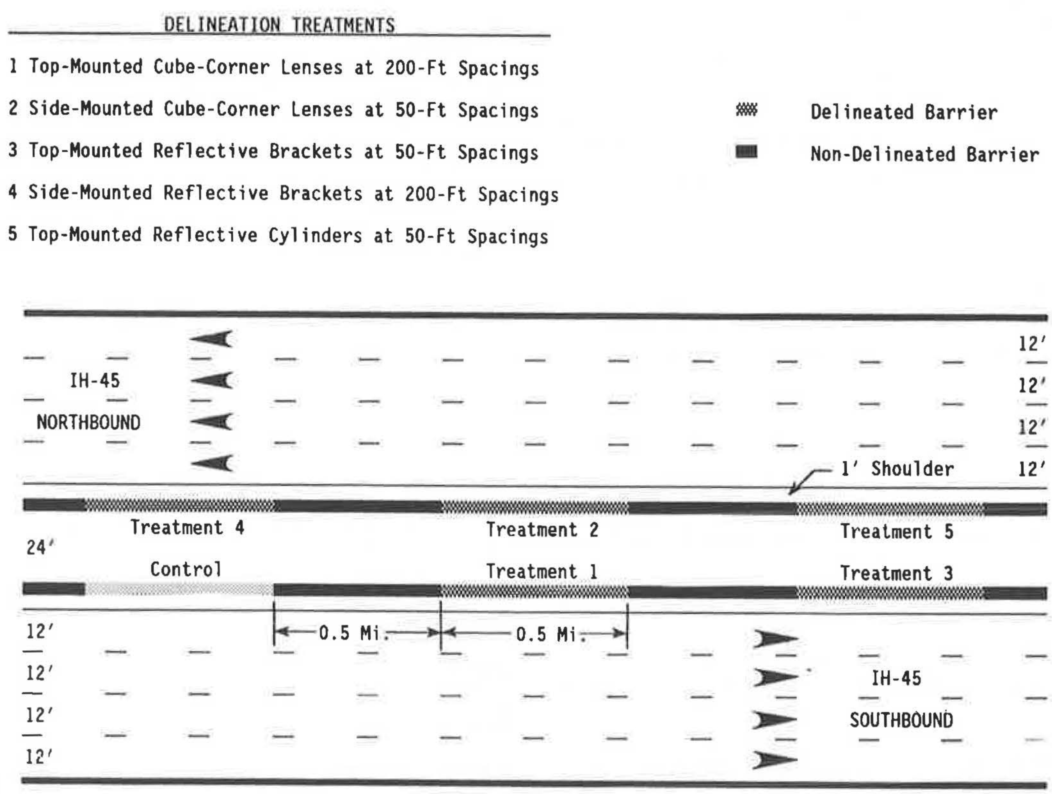

FIGURE 1 Layout of delineation treatments at the I-45 (Houston, Texas) study site.

segment by means of a low–light level video camera. The camera was mounted on overhead sign supports spanning the freeway and positioned to provide a top-down view of traffic traveling next to the barrier at each treatment segment. Videotape data were collected continuously throughout the nighttime hours on two weeknights (Monday–Thursday) at each treatment segment before and immediately after the delineators were attached to the CSSB. Although data were collected primarily under dry pavement conditions, some rain data were collected at Treatment 4 (side-mounted brackets with HI sheeting at 200-ft spacings).

To account for any time-related or other unidentified effects present during the study, data were also collected at a "control" location upstream of any delineation. Data were collected starting at the downstream treatment segment in each direction of travel (segments B and E in Figure 1). Once "before" and "after" data were obtained at a segment, the camera was moved to the next upstream segment, and the process was repeated. This was done to ensure that traffic being observed and monitored at a particular treatment segment was not influenced by a previously installed delineation treatment upstream.

The nighttime hours were divided into two time periods. The first period, from 9 p.m. to midnight, was taken to be representative of higher-volume nighttime traffic conditions. The second period, representing lower-volume nighttime conditions, began at midnight and ended at 5 a.m. Three measures of effectiveness (MOEs) were used to evaluate the effect of delineation on driver performance:

- Lane distribution Measured for the two lanes closest to CSSB. It was assumed that delineation would affect traffic primarily in these two lanes.
- Lane straddling The number of vehicles straddling the lane stripe between the two lanes closest to the CSSB.
- Lateral distance Measured as the distance between the left rear tire and the bottom of the CSSB. This measure was estimated to the nearest foot from the videotape data.

The lane distribution and lane-straddling data were measured continuously throughout the nighttime hours. However, because it was not necessary to record the lateral distance for every vehicle in the inside travel lane, measurements were sampled throughout the night in direct proportion to the actual lane volumes present.

Subjective Evaluations

In this phase of the study, a limited number of subject drivers drove a test vehicle in the leftmost inside lane next to the CSSB. Subjects then ranked the treatments in terms of the relative brightness and effectiveness in helping them maintain a safe travel path. Subjects also provided indications as to whether they felt that each treatment was adequate in terms of brightness and effectiveness (independent of the other treatments).

Ten Houston-licensed drivers evaluated the treatments in a clean, new condition, and the same ten subjects, plus an additional 20-yr-old female, also evaluated the treatments after the delineators were in place for a period of time and had become dirty. The study sample consisted of seven women (eight in the evaluation of the dirty treatments) and three men. Ages of the subjects ranged from 18 to 56 years. The subjects, as a group, were well-educated, experienced drivers. None of the participants lived near the study site, so their familiarity with the site was limited to only occasional trips through the section. Full details of the study procedure may be found in the original study report (*6*).

Delineator Visibility

The delineators were in place on the CSSB from February to June 1987. The researchers periodically examined the delineators under nighttime conditions and recorded the maximum distance at which each could be seen from within a test vehicle with its headlights set to low beam. This technique provided a quick, consistent method for monitoring the changes in delineator visibility over time. The study procedure described for the collection of the driver performance data required that the treatments be installed at different times, causing them to be exposed to slightly different weather conditions. To normalize the visibility analysis, a new delineator was installed at each of the previous treatment segments when the final (fifth) treatment was installed. Subsequent visibility assessments were then based on these specific delineators. Visibility measurements were taken at the time of the final installation and at 2-, 6-, 10-, and 16-week intervals.

STUDY RESULTS

Driver Performance Data

Lane Distribution

Table 2 presents the results of the analysis of the lane distribution data. During the higher-volume nighttime hours, the proportion of drivers using the inside travel lane decreased 3 percent at Treatment 1 (the top-mounted cube-corner lenses at 200-ft spacings) and by 1 percent at Treatment 2 (side-mounted cube-corner lenses at 50-ft spacings). Meanwhile, the proportion of drivers in the inside lane increased 2 percent at Treatment 5 (the top-mounted cylinders at 50 ft). For the low-volume conditions, the proportion of vehicles traveling in the inside travel lane decreased by 2 percent at Treatment 1 but increased 3 percent at Treatment 5. These proportional changes are very small in terms of lane volumes, so the treatments appeared to have had very little practical effect on lane distribution.

Lane Straddling

Lane-straddling rates at all of the treatment segments were quite low during the higher-volume nighttime hours, as shown in Table 3. Statistical comparisons of the rates found only one significant change, an increase at Treatment 2 (side-mounted cube-corner lenses at 50-ft spacings).

Lane-straddling rates during the lower-volume nighttime hours, although greater than those in the higher-volume hours, changed little between before and after conditions. Only Treatments 4 (side-mounted brackets at 200-ft spacings) and 5 (top-mounted cylinders at 50-ft spacings) showed statistically significant changes. Given the extremely small sample sizes obtained in this comparison, it is not appropriate to draw any

TABLE 2 COMPARISON OF LANE DISTRIBUTION DATA BEFORE AND AFTER DELINEATION, I-45, HOUSTON

	High-Volume Nighttime Periods[a]					Low-Volume Nighttime Periods[a]				
	Before Delineation		After Delineation		Difference	Before Delineation		After Delineation		Difference
Treatment	Percent	*n*	Percent	*n*	(%)	Percent	*n*	Percent	*n*	(%)
Control (no delineation)	40.4	6,963	41.4	6,304	+1.0	22.8	2,823	24.0	2,570	+1.2
(1) Top-mounted cube-corner, 200-ft spacings	41.4	6,612	38.3	5,539	−3.3[b]	23.8	2,650	21.7	2,044	−2.1[c]
(2) Side-mounted cube-corner, 50-ft spacings	38.5	6,829	36.3	5,534	−1.2[b]	25.4	2,925	26.0	2,310	+0.6
(3) Top-mounted brackets, 50-ft spacings	39.2	5,726	38.7	5,627	−0.5	24.4	1,951	26.2	2,040	+1.8
(4) Side-mounted brackets, 200-ft spacings	34.7	6,598	33.5[d]	3,040	−1.2	22.0	2,568	20.3	1,596[d,e]	−1.7
(5) Top-mounted cylinders, 50-ft spacings	35.9	4,927	37.5	5,395	+1.6[b]	22.7	1,800	25.8	2,157	+3.1[b]

[a]Percent = percentage of inside and middle lane traffic in lane next to barrier; *n* = sample size in number of vehicles.
[b]Different at 0.05 level of significance.
[c]Different at 0.10 level of significance.
[d]Data represent only 1 night.
[e]Data collected under rainy conditions, with wet pavement.

TABLE 3 COMPARISON OF LANE STRADDLING RATES, BEFORE AND AFTER DELINEATION, IH-45, HOUSTON

	High-Volume Nighttime Periods[a]					Low-Volume Nighttime Periods[a]				
	Before Delineation		After Delineation			Before Delineation		After Delineation		
Treatment	Rate	*n*	Rate	*n*	Change	Rate	*n*	Rate	*n*	Change
Control no delineation	1.5	4	0.8	2	−0.7	4.7	3	6.5	4	+1.8
(1) Top-mounted cube-corner, 200-ft spacings	0.7	2	2.4	5	+1.7	7.9	5	6.8	3	−1.1
(2) Side-mounted cube-corner, 50-ft spacings	0.0	0	1.4	3	+1.4[b]	1.3	1	5.0	3	+3.7
(3) Top-mounted brackets, 50-ft spacings	0.4	1	0.0	0	−0.4	4.2	2	3.7	2	−0.5
(4) Side-mounted brackets, 200-ft spacings	0.9	2	2.0[c]	2	+1.1	3.5	2	15.4	5[c,d]	11.9[b]
(5) Top-mounted cylinders, 50-ft spacings	0.6	1	0.5	1	−0.1	4.9	2	0.0	0	−4.9

[a]Rate = lane-straddling rate per 1,000 vehicles in inside lane; *n* = number of lane straddlings observed.
[b]Different at 0.05 level of significance.
[c]Data represent only one night.
[d]Data collected under rainy conditions with wet pavement.

solid conclusions from the data. It is interesting, however, to note that the rate was again higher for Treatment 2 and was almost statistically significant. These data may suggest that the combination of close delineator spacing and the side-mounted position may make some drivers too apprehensive of the barrier.

It should also be noted that the "after" data at Treatment 4 were collected when the pavement was wet. The video recordings showed a significant glare problem; the high-mast lighting and vehicle headlights appeared to wash out the edgeline and lane stripes. Consequently, the large increase in the lane straddling rate is not necessarily an indication of the effect that this treatment had on traffic. Instead, it indicates that some drivers have more difficulty staying in their lane at night in wet pavement conditions, even where fixed illumination is present.

Lateral Distance

As stated previously, the lateral distance data collected were measured to the nearest foot rather than on a continuous scale (portions of a foot). The Kolmogorov-Smirnoff test (*7*) (a nonparametric goodness-of-fit test) was applied to determine whether the probability distributions of the lateral distance data differed. During the higher-volume nighttime hours, statistically significant differences were found at Treatments 4 (side-mounted brackets at 200-ft spacings) and 5 (top-mounted cylinders at 50-ft spacings). The lateral distance distributions for these segments are shown in Figures 2 and 3. The distribution appears to have shifted slightly away from the barrier at Treatment 4, while the distribution at Treatment 5 seems to have shifted closer to the barrier.

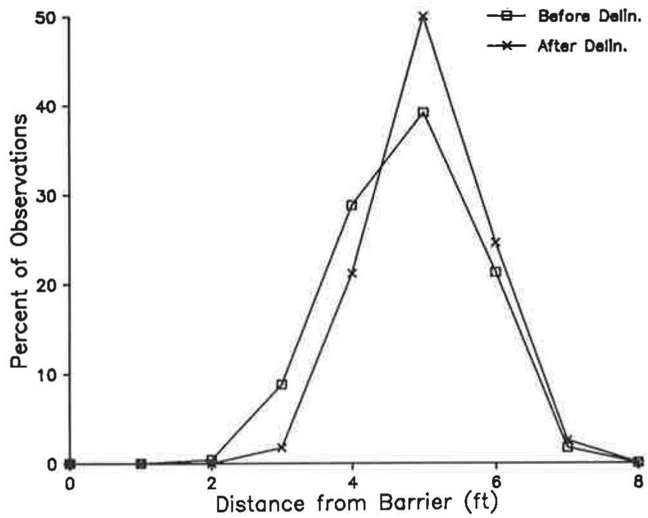

FIGURE 2 Lateral distance distribution for Treatment 4: reflective brackets, side-mounted at 200-ft spacings (9:00 p.m. to midnight).

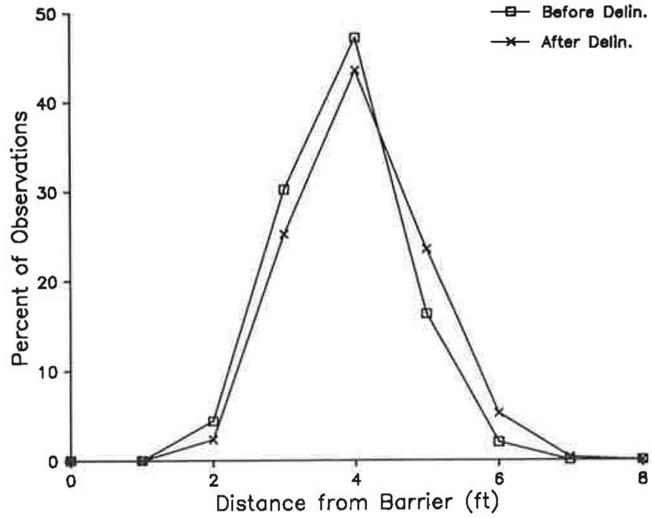

FIGURE 4 Lateral distance distribution for Treatment 1: cube-corner lenses, top-mounted at 200-ft spacings (midnight to 5:00 a.m.).

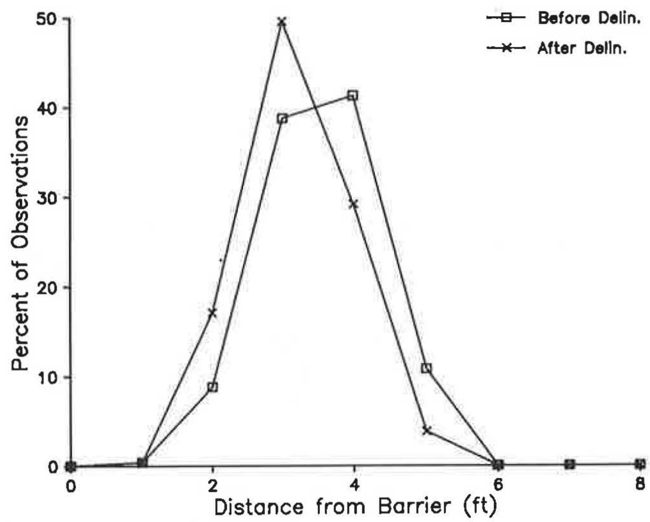

FIGURE 3 Lateral distance distribution for Treatment 5: reflective cylinders, top-mounted at 50-ft spacings (9:00 p.m. to midnight).

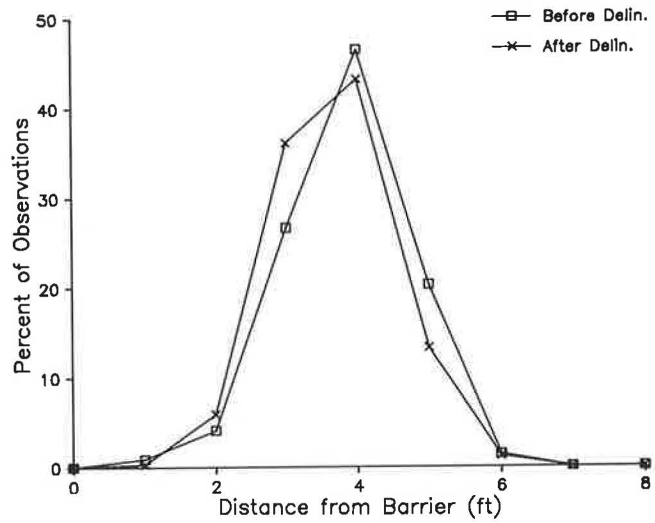

FIGURE 5 Lateral distance distribution for Treatment 2: cube-corner lenses, side-mounted at 50-ft spacings (midnight to 5:00 a.m.).

Results of the "before" and "after" comparisons at each treatment segment during lower nighttime volume hours indicate that the lateral distance distributions shifted slightly away from the CSSB at Treatment 1 (top-mounted cube-corner lenses at 200-ft spacings) but were slightly closer to the CSSB at Treatments 2 (side-mounted cube-corner lenses at 50-ft spacings) and 5 (top-mounted cylinders at 50-ft spacings). Plots of the lateral distance distributions for these treatments are presented in Figures 4–6.

Subjective Evaluations

Clean Delineators

Table 4 presents the total rank scores and adequacy ratings by the subjects of the clean delineation treatments. Overall, the brightness rankings showed very little difference between the high- and low-scoring treatments. In fact, a Friedman analysis of variance (ANOVA) test for ranked data (*8*) found no statistically significant differences, indicating that the subjects, as a group, ranked all the treatments about equal. However, the adequacy ratings obtained from the subjects indicate a different perspective. Treatments 1 through 4 received adequate ratings from at least 80 percent of the subjects. Treatment 5 (top-mounted cylinders at 50-ft spacings), on the other hand, received adequate ratings from only 50 percent of the subjects.

Table 4 also contains the total rank scores from the subjects with respect to each treatment's relative effectiveness in helping drivers maintain a safe travel path next to the CSSB. Again, a Friedman ANOVA test found that the rankings did not differ significantly. As with the brightness rankings, however, Treatment 5 received the worst total score.

During the evaluations, subjects were also asked to provide comments that they had about each treatment. Table 5 is a summary of these comments in terms of driver like or dislike of

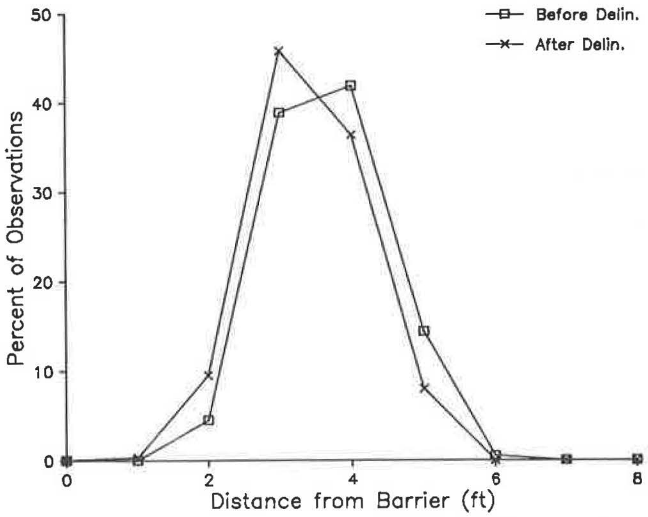

FIGURE 6 Lateral distance distribution for Treatment 5: reflective cylinders, top-mounted at 50-ft spacings (midnight to 5:00 a.m.).

the delineator type, spacing, or mounting position. No clear trend is evident with respect to delineator type: all received both positive and negative comments. The comments did show that subjects dislike the top-mounted delineation treatments, and a corresponding liking was shown for those treatments that were mounted on the side. Subjects indicated that the treatments mounted on top of the barrier seemed to make the travel lanes appear wider than they were and tended to draw them closer to the barrier. However, this perception was not demonstrated in the driver performance data, which showed vehicles closer to the barrier at Treatment 5 (top-mounted cylinders at 50-ft spacings) but farther away at Treatment 1 (top-mounted cube-corner lenses at 200-ft spacings).

Subjects offered several reasons for preferring side-mounted delineation, including a more direct line of sight for drivers, a better indication of the location of the barrier wall, and a more realistic perception of lane width. Subjects also had strong feelings about the spacings of the delineation treatments. As illustrated by the values in Table 5, the 200-ft spacing of Treatments 1 and 4 was disliked by several subjects, while a number of subjects specifically indicated that they liked the closer (50-ft) spacing.

Dirt-Covered Delineators

Subject evaluations of the treatments were also conducted after the delineators had been in place several months and had become covered with dirt and road film. Subject rankings of each treatment's brightness and effectiveness under this dirty condition are presented in Table 6. Also presented in the table is the proportion of the subjects who felt that the brightness of the particular delineation treatment was adequate.

On the basis of the Friedman ANOVA test, the rankings were found to differ significantly. Subjects ranked Treatment 2 (side-mounted cube-corner lenses at 50-ft spacings) as the brightest and Treatment 5 (top-mounted cylinders at 50-ft spacings) as the dimmest. Scores for the remaining treatments show that Treatments 1 (top-mounted cube-corner lenses at 200-ft spacings), 4 (side-mounted brackets at 200-ft spacings), and 3 (top-mounted brackets at 50-ft spacings) were ranked the second-, third-, and fourth-brightest treatments, respectively. Even in the dirt-covered condition, the brightness of Treatment 2 was rated adequate by all 11 subjects (100 percent), and 7 subjects (64 percent) rated Treatment 1 adequate. None (0 percent) of the subjects rated Treatment 5 adequate, while Treatment 3 was rated adequate by only one (9 percent) subject.

Table 6 also summarizes the subject rankings of the treatment's effectiveness in the dirty condition. The rankings were again found to be significantly different, with Treatment 2 ranked most effective and Treatment 5 ranked least effective by the subjects. The second-, third-, and fourth-place rankings corresponded to Treatments 1, 3, and 4, respectively. Even though Treatment 4 was ranked brighter than Treatment 3, it was ranked less effective by the subjects. This could be due in part to the closer spacing of the delineators for Treatment 4.

Subject comments about the dirt-covered treatments are presented in Table 7. Eight subjects (73 percent) stated that they did not like Treatment 5 (the top-mounted cylinders at 50-ft spacing), primarily because it was not bright enough. Ten subjects (91 percent) also had a strong dislike of the 200-ft spacing of Treatment 4 (the side-mounted brackets), and they mentioned that the spacing was too great to be effective. Conversely, nine subjects (82 percent) had positive comments for Treatment 2 (side-mounted cube-corner lenses at 50-ft spacings). Again, subjects stated that side-mounted delineation provided a better indication of the location of the barrier and helped guide them more effectively.

Delineator Visibility

The periodic measurements of the maximum visibility distance for each treatment are presented in Figures 7 and 8. For both mounting positions (top or side) the cube-corner lenses (Treatments 1 and 2) lost their original visibility at a slower rate than

TABLE 4 SUBJECT EVALUATION OF DELINEATION TREATMENTS, DIRTY CONDITION, IH-45, HOUSTON

	Brightness Evaluation[a]			Effectiveness Evaluation[a]	
Treatment	Total Rank Score	Relative Ranking	Number Rating Brightness Adequate	Total Rank Score	Relative Ranking
(1) Top-mounted cube-corner, 200-ft spacings	23	2	7 (64%)	31	2
(2) Side-mounted cube-corner, 50-ft spacings	13	1	11 (100%)	13	1
(3) Top-mounted brackets, 50-ft spacings	40	4	1 (9%)	35	3
(4) Side mounted brackets, 200-ft spacings	33	3	4 (36%)	36	4
(5) Top-mounted cylinders, 50-ft spacings	55	5	0 (0%)	53	5

[a]Rankings were 1 = brightest or most effective, 2 = next brightest or most effective, and so on.

TABLE 5 SUMMARY OF SUBJECT COMMENTS, DIRTY CONDITION, IH-45, HOUSTON

	Delineator Type (includes size, shape, and brightness)		Delineator Mounting Position		Delineator Spacing	
Treatment	Good	Poor	Good	Poor	Good	Poor
(1) Top-mounted cube-corner, 200-ft spacings	0	2	2	2	0	4
(2) Side-mounted cube-corner, 50-ft spacings	0	1	9[a]	1	3	0
(3) Top-mounted brackets, 50-ft spacings	2	5[b]	0	2	4	3
(4) Side-mounted brackets, 200-ft spacings	2	3	2	1	0	10[b]
(5) Top-mounted cylinders, 50-ft spacings	0	8[b]	0	1	5[a]	1

[a]Large number of positive comments.
[b]Large number of negative comments.

TABLE 6 SUBJECT EVALUATION OF DELINEATION TREATMENTS, CLEAN CONDITION, IH-45, HOUSTON

	Brightness Evaluation[a]			Effectiveness Evaluation[a]	
Treatment	Total Rank Score	Relative Ranking	Number Rating Brightness Adequate	Total Rank Score	Relative Ranking
(1) Top-mounted cube-corner, 200-ft spacings	30	3	10 (100%)	35	4
(2) Side-mounted cube-corner, 50-ft spacings	23	1	9 (90%)	19	1
(3) Top-mounted brackets, 50-ft spacings	32	4	10 (100%)	27	2
(4) Side-mounted brackets, 200-ft spacings	29	2	8 (80%)	36	5
(5) Top-mounted cylinders, 50-ft spacings	36	5	5 (50%)	33	3

[a]Rankings were 1 = brightest or most effective, 2 = next brightest or most effective, and so on.

TABLE 7 SUMMARY OF SUBJECT COMMENTS, CLEAN CONDITION, IH-45, HOUSTON

	Delineator Type (includes size, shape, and brightness)		Delineator Mounting Position		Delineator Spacing	
Treatment	Good	Poor	Good	Poor	Good	Poor
(1) Top-mounted cube-corner, 200-ft spacings	0	1	1	5[a]	0	6[a]
(2) Side-mounted cube-corner, 50-ft spacings	2	2	6[b]	1	6[b]	0
(3) Top-mounted brackets, 50-ft spacings	1	1	0	3	8[b]	1
(4) Side-mounted brackets, 200-ft spacings	1	3	3	0	0	7[a]
(5) Top-mounted cylinders, 50-ft spacings	1	2	2	2	4	0

[a]Large number of negative comments.
[b]Large number of positive comments.

did the brackets or cylinders covered with HI reflective sheeting (Treatments 3, 4, and 5). As the figures also show, the visibility distance of the delineators was greater after 16 weeks than it was after 10 weeks. The improvement is especially noticeable for the cube-corner lenses. Heavy rains that preceded the 16-week evaluation are believed to have washed some of the road film from the delineation, resulting in the improved visibility. It should be noted that the visibility distance of the brackets or cylinder with HI sheeting did not increase as noticeably as the visibility distance of the cube-corner lenses. Comparison of Figures 7 and 8 also shows, as expected, that treatments mounted on the side of the barrier lost visibility faster than the top-mounted treatments.

RECOMMENDATIONS

This paper has presented the results of a study of five CSSB delineation treatments on an illuminated high-volume urban freeway in Texas, where the CSSB was located 1 ft away from the travel lanes. Limitations in study scope and funding prevented a complete analysis of all combinations of delineator type, spacing, and mounting position examined in this study. Consequently, these results can not be taken as conclusive, and additional research on this topic will be necessary. Of the delineators examined in this study, cube-corner lenses are recommended for delineating CSSBs in narrow freeway median applications. These delineators do not lose their reflectivity due to dirt and grime as quickly as those covered with HI sheeting.

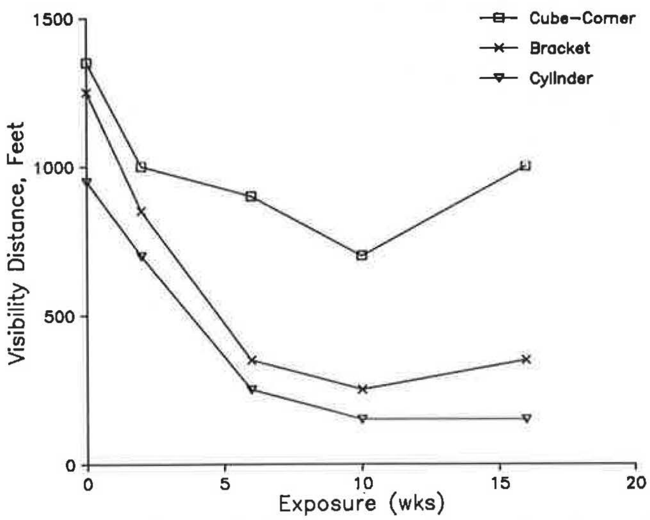

FIGURE 7 Visibility distances of top-mounted delineator treatments over time.

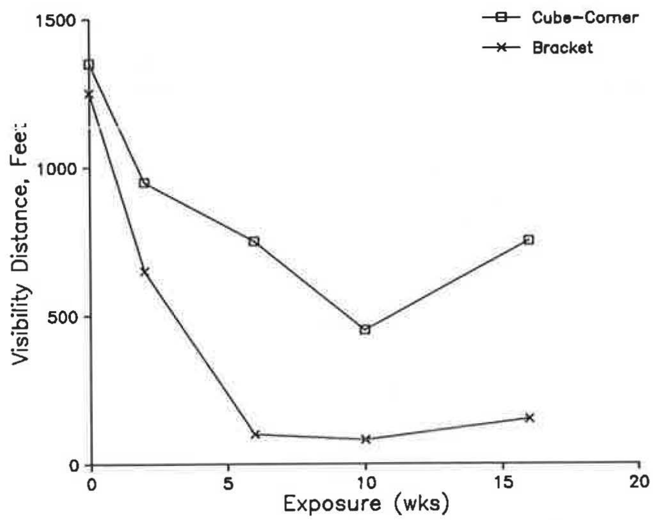

FIGURE 8 Visibility distances of side-mounted delineator treatments over time.

Lane-straddling data collected at Treatment 2 showed a slight increase, possibly indicating that the combination of the side-mounted position and the close delineator spacing may make some drivers too apprehensive of the CSSB if the barrier is located close to the travel lanes. Lane straddling could result in vehicle conflicts or other operational problems. Therefore, for situations with limited lateral clearance, top-mounted delineation is recommended.

Subjects indicated a preference for close (50-ft) spacings. However, driver performance data did not suggest that one spacing was better than the other. Therefore it is recommended that a 200-ft spacing be considered maximum. To ensure adequate control and guidance information for drivers, however, closer spacings may be necessary for CSSBs on sharp curves.

These recommendations are also suggested when CSSBs in work zones are to be delineated. Additional research is needed, however, to evaluate the effect of these and other delineation treatments in work zone applications. Research is also needed to determine what effects delineation may have on traffic safety in terms of accident potential and costs.

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