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Traffic Control in Work Zones

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Contents

Foreword

The papers included in this Record have two common themes: (a) safe control of traffic in construction and maintenance work zones and (b) objective methods to guide decision making in traffic safety planning for such work zones. Many, if not most, of the decisions guiding work zone traffic control have subjective bases, but the rapid development of objective information methods could mean that decisions based on objective data will become the rule of the future. Where there is a lack of objective information, newly developed expert systems capture the collective wisdom of a large number of experts, who would not be available at most work sites.

Featured authors offer suggestions that will improve lane discipline at lane closures, offer better control of speed through work zones, and provide guidelines for use of warning devices in work zones. One author concludes that a model that has been proposed for determining frequencies for inspection and repair of highways safety devices will provide conservative results, and another author notes that there is no relationship between the length of two-lane, two-way (TLTW) lane closures and the safety of TLTW operations.

Among the other results reported in this Record are indications that significant discrepancies exist between standards and practice in placement of traffic control devices. Sites with short tapers and missing arrow boards or signs are commonly observed. These sites were demonstrated to have higher speed variations in the work zone when speed was not dictated by traffic. In general, control devices are being placed closer to the lane taper than allowed by standards, and discrepancies were more frequent at short-term sites. At these sites, an average of two traffic control devices were missing, compared to only one missing device at long-term sites. Elsewhere, it is noted that supplemental traffic control devices, such as additional signs, variable message signs, and rumble strips, may be advisable when conventional traffic control devices fail to reduce late merges and there is excessive congestion.

A study was made of four methods to reduce traffic speed through highway construction sites. The four deterrents tested were the flagging procedure described in the *Manual on Uniform Traffic Control Devices*, MUTCD flagging augmented by having the flagger first motion the motorists to slow and then having the flagger point at a nearby speed limit sign with the free hand, a marked police car with flashing light and active radar, and a uniformed police office controlling traffic. As expected, the latter two methods proved to be most effective at controlling speed. In an investigation of the safety and operational performance of short-term and intermittent construction sites, it was observed that the short-term sites had a nearly constant accident rate of 0.80 accidents per mile-day. These results indicate that the average short-term site will experience either an increase or decrease in the previously existing accident rate, depending on whether that original rate was lower or higher than the observed rate. Caution is needed in interpreting these results, however, because one category of data was dominated by accidents at a single site, which constituted 85 percent of all the accidents observed in that particular category.

Whether the reader is a traffic engineer planning safety features for a construction site or a highway official pondering the logistics of traffic safety control device replacement, the papers in this Record should be both interesting and informative.

ARTWORK: A Simulation Model of Urban Arterial Work Zones

AHMAD SADEGH, A. ESSAM RADWAN, AND N. M. ROUPHAIL

Several studies pertaining to the modeling of freeway and arterial traffic movements were reviewed, but none of them were applicable to lane closure in construction work zone on arterials. Hence the goal was to develop a computer-based methodology for the evaluation of traffic control systems at arterial street lane closure in the vicinity of signalized intersections. More specific objectives were to develop a microscopic computer simulation model of traffic flow at arterial street lane closures, to derive a series of system measures of performance as an output of the model, and to validate the model's logic by using field data. Delay, fuel consumption, and queue buildup were used as the measures of effectiveness in validating the model. It was concluded that the model performs satisfactorily.

Most previous research related to traffic flow near work zones has been done with respect to freeways. While several techniques, theories, and methods (taking into consideration the number of open lanes, number of closed lanes, length of work zone, etc.) are available to calculate vehicle delay at freeway work zones, no literature is available with respect to work zones on arterials.

A review of previous research concerning the evaluation of arterial street lane closure performance revealed a common flaw. Although most studies attempted to investigate the impact of individual traffic control elements on some performance measure of traffic flow, many failed to treat the arterial lane closure site as a total information system. A controlled experiment was needed to develop a ranking procedure for the variables that affect traffic flow quality at arterial street lane closures.

The traffic flow model described in the next section is a first attempt at a more systematic approach to the lane closure problems of urban arterials. In addition to the traditional aspects of arterial traffic flow (such as traffic volume, speeds, headways, car following, and gap acceptance rules), the model explores a wide range of parameters describing driver behavior in a construction lane closure. Parameter effects on the overall system performance are also assessed.

ARTWORK, a simulation model of urban arterial work zone lane closure, was developed by using the SLAM II simulation language (*1*). In this study, a hybrid model was selected so that the fixed time mechanism could be used to update the vehicle positions in the system and the next event increment could be applied to the vehicle-generating pool, that is, the processing of vehicles that have just entered the system. This paper summarizes the development and the validation of the ARTWORK model.

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MODEL STRUCTURE

ARTWORK consists of a main program and 23 subprograms and functions. The model is microscopic in nature; that is, each driver-vehicle unit is identified as a separate entity. Periodic updating of each vehicle's status is performed at 1-sec intervals. The simulation model consists of the following elements:

Vehicle Representation

In the simulation model, each vehicle is assigned a set of 25 attributes upon entering the system. Some of these attributes, representing the status of vehicle in the system, stay constant, and the remaining attributes are updated during the simulation run.

Vehicle attributes are generated at the entry point either stochastically or deterministically and are updated throughout the system. Vehicle design characteristics such as length, width, and driver eye height are input to the model by the user. The model is capable of handling different types of vehicles.

Traffic Control Device Representation

The modeling of various traffic control devices (TCDs) is accomplished in the same way that vehicles are represented. All TCDs in the construction zone were categorized as "merge stimuli." Attributes that describe each TCD are related to their design features, such as height, maximum recognition distance, and message information processing time, as well as location characteristics, such as longitudinal and lateral position in the zone and TCD placement on one or both sides of the road.

Roadway Representation

Vehicles are generated at entry point and are released at exit point. The roadway includes two traffic signal lights. The locations of data collection points are dependent on the user, and up to 20 simulated data collection points can be used.

The primary concern in the collection of traffic flow data in the model is compatibility with the field data collection method. This method consisted of recording vehicle arrival times at tape switches (grouped in pairs) and obtaining vehicle speeds, headways, and frequency of lane changes at each tape switch pair. The simulated system works in an identical fashion. The locations of data collection elements are provided as input to the model.

Traffic Signal Representation

Traffic signals were worked into the model with green, yellow, and red signal phasings and provision for exclusive left turn phase, if needed. The model only handles pretimed signal settings. The offsets for each traffic signal can also be input by the user.

Vehicle Generation

Nine probability distribution functions (PDF) are available in the model, each of which has a separate code provided by the user as input. Vehicle arrivals, desired speeds, and reaction time are randomly generated by using the proper distribution. Before arrival of a vehicle, tests are performed internally by the model to ensure that the car-following rules are satisfied at the entry point.

Car-Following Rules

The car-following rules adopted in this study apply only to vehicles in platoons, that is, those cases in which drivers cannot maintain their desired speeds because of a slower vehicle ahead. Vehicle positions are updated according to their location from the entry point. In a fixed-interval scan of 1 sec the leader position and speed are first determined; following vehicle speed and position are then updated by the car-following rules. This approach closely follows the noncollision constraints developed in the INTRAS model (*2*). The car-following rules state that drivers in platoons will maintain a spacing at least equal to their reaction distance plus length of lead vehicle when their speed is less than or equal to that of the platoon leader. This spacing is incremented by the deceleration distance to the platoon leader speed if the speed of the following vehicles is higher than the lead vehicle speed.

Gap Acceptance Rules

The process of lane changing from the closed lane to the open lane is initiated as soon as the vehicle has advanced beyond the legibility distance of the assigned merge stimulus. A minimum information processing time is needed by the driver to initiate the desired action.

When a lane change is attempted, the car-following rules for the lead and follower vehicles in the open lane are tested to ensure a safe merge into the open lane. Next, the gap between the lead and follower vehicles in the open lane is compared against the critical gap randomly assigned to the vehicle from the observed distribution of critical gaps. Once these conditions are satisfied, the process of lane changing occurs.

Input Data

The following input data must be entered by the user to run the model: traffic characteristics, vehicle characteristics, roadway characteristics, construction data, signal settings, distribution functions, and simulation control parameters.

Output

The model produces three major output components:

- Listing of input data;
- Statistics and histograms of the following variables: speeds and headways at each data collection point, vehicle merging distribution, fuel consumption, delay, and number of queue buildups; and
- Trajectory of vehicles at any specified time during the simulation run.

MODEL VALIDATION

Field studies were conducted as a means of evaluating the effectiveness of mathematical traffic flow models in describing a phenomenon under investigation (car following, gap acceptance, etc.). The purposes of the field studies were as follows:

- To provide input parameters for the simulation model, such as traffic volume, composition and speeds upstream of the construction and maintenance zone, types and locations of traffic control devices, traffic signal settings, and position of simulated tape switches;
- To compare observed driver behavior in the field (in terms of car following and lane changing) with that predicted by the model, and
- To test the model logic by comparing the actual measures of effectiveness observed in the field with those predicted by the model.

McClintock Street, located in Tempe, Arizona, was selected as the first site for a field study. The northbound approach on McClintock at the intersection with University was under construction during the morning off-peak hour. This resulted in closure of a through lane and a right lane.

Mill Avenue, also in Tempe, was selected as the second site. The southbound approach on Mill at the intersection with Alameda was under construction. This resulted in closure of a through lane. Data were collected during the morning off-peak hour.

Validation of the simulation model was performed by entering the data collected at both sites into the model and comparing the simulation results against the observed measures of effectiveness.

McClintock Site

Certain traffic description parameters were targeted for comparison. First, the number of vehicles that traveled over four road tubes for a period of 30 minutes was compared against the count made by simulated tape switches entered into the model. The results were as follows:

	Observed	*Simulated*
Open lane tape 1	445	400
Closed lane tape 1	306	253
Open lane tape 2	532	482
Closed lane tape 2	126	99

The observed counts by the road tubes were slightly higher than the simulated counts by the model.

Next, from observation the simulation model was run with the following distribution of percentage of drivers reacting to each merge stimulus:

- 10 percent to construction activities;
- 20 percent to taper cones,
- 60 percent to the "lane ends–merge left" sign, and
- 10 percent to the "left lane closed" diagram.

The results of the simulation model were a merging distribution with mean value of 1,087 ft and standard deviation of 679 ft before the University and McClintock intersection. The model also stated that the earliest merge occurred at 2,182 ft and the latest merge occurred at 298 ft upstream of the University and McClintock intersection. These results were in an acceptable range when compared to the observed merging vehicles from closed lane into the open lane.

The simulation model then calculated an average queue length of 19 and a maximum queue length of 53 vehicles for the open lane at the intersection of University and McClintock. It also calculated an average queue length of 10 and a maximum of 34 vehicles behind the taper in closed lane. The observed maximum queue for the open lane was 68 and for the closed lane beyond the taper, 35. The observed queue lengths were higher in the open lane and approximately the same for the closed lane beyond the taper in comparison to the simulated queue length.

Finally, the average speeds of vehicles traveling over the four road tubes were compared against the average speed of vehicles traveling over the simulated tape switches in the model. The results were as follows:

	Observed (mph)	*Simulated (mph)*
Open lane tape 1	37	38
Closed lane tape 1	32	35
Open lane tape 2	15	16
Closed lane tape 2	8	12

Mill Avenue Site

Again, certain traffic description parameters were targeted for comparison. First, the speed and number of vehicles traveling over two road tubes 1,000 ft downstream of the Broadway and Mill intersection for a period of 30 minutes (twice the observed values) were compared against the count made by simulated tape switches entered into the model. The results were as follows:

	Observed	*Simulated*
Number of vehicles at open lane tape	532	494
Number of vehicles at closed lane tape	348	320
Speed of vehicles at open lane tape (mph)	33	35
Speed of vehicles at closed lane tape (mph)	33	32

The observed values were slightly higher than the simulated counts by the model.

Next, from observations the simulation model was run with the following distribution of percentage of drivers reacting to each merge stimuli:

- 10 percent to construction activities,
- 20 percent to taper cones,
- 50 percent to the "lane ends–merge left" sign, and
- 20 percent to the "left lane closed" diagram.

The results of the simulation model were a merging distribution with a mean value of 1,290 feet and a standard deviation of 608 ft before the Alameda and Mill intersection. The model also stated that the earliest merge occurred at 2,458 ft and the latest merge occurred at 477 ft before the Alameda and Mill intersection. These results were in the acceptable range when compared to the vehicles observed merging from the closed lane into the open lane.

The simulation model then calculated an average queue length of 6 and a maximum of 21 vehicles for the open lane and an average queue length of 1 and a maximum of 9 vehicles behind the taper in closed lane at the intersection of Alameda and Mill. The observed maximum queue for the open lane was 24 and for the closed lane beyond the taper, 13. The observed queue length was higher than the simulated queue length in both open the and closed lanes.

The discrepancies among the observed and the simulated results were caused by the following factors:

- Vehicle arrivals were simulated by using a lognormal distribution fitted to the field data. This distribution does not match the real arrival times in the field;
- The secondary platoon arrival was ignored because there was a very small number of vehicles outside the primary platoon;
- The vehicles arriving from Apache Boulevard were assumed to be uniformly distributed, with mean of one vehicle every 2 sec turning left and one vehicle every 5 sec turning right into the arterial while the signal was green. These distributions were chosen to be as close to the field data as possible.
- Arrivals from the side streets, if any, were assumed to be negligible; and
- The seed number used to generate the random arrivals does make a difference to the number of vehicles created.

The simulation model represents true system behavior closely enough, however, to be used as a substitute for the actual system.

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Comparative Study of Short- and Long-Term Urban Freeway Work Zones

NAGUI M. ROUPHAIL, ZHAO SHENG YANG, AND JOSEPH FAZIO

Freeway construction and maintenance work in major metropolitan areas is often limited to off-peak daytime or nighttime periods because of the heavy traffic volumes served by these facilities. Previous research has focused primarily on long-term work zones that continuously occupy the road space for several days or months, but little information exists regarding the safety and operational performance of short-term and intermittent sites. The objectives of this study were to compare the accident experience at both long-term and short-term sites before, during, and after freeway construction or maintenance work. In addition, an evaluation of traffic flow and traffic control device (TCD) layout in terms of adherence to state standards was undertaken for both types of zones. It was found that at long-term sites the accident rate increased by an average of 88 percent during the existence of the work zone site, in comparison to the before period, and decreased by an average of 34 percent in the after period. For short-term sites, a nearly constant accident rate of 0.80 accident/mile-day of construction or maintenance was observed. The evaluation of TCD layout revealed significant discrepancies between standards and practice. In general, devices were placed closer to the lane taper than is allowed by standards. Discrepancies were more frequent at the short-term sites, where an average of two TCDs were missing, in comparison to one TCD missing at long-term sites. Moreover, wider variations in warning signs placement were observed at the short-term sites. Finally, sites characterized by short tapers, missing arrow boards, signs, or any combination of these factors exhibited higher speed variations in the work zone when speed was not dictated by traffic.

Provision of traffic congestion relief in U.S. urban areas is a major challenge facing today's transportation engineers (*1*). The problem of providing safe and efficient conduct of traffic in and around highway work zones, coupled with the need to maintain and upgrade the physical facilities, has received national attention in recent years. In large metropolitan areas, it is becoming exceedingly difficult to serve the needs of motorists while providing adequate protection to the work crew at the same time. In addition, providing access to and from the work site for construction and maintenance vehicles, especially during peak flow periods, is not an easy task. To alleviate some of these problems, highway agencies have adopted procedures whereby routine construction and maintenance work (as opposed to major reconstruction or rehabilitation) is confined to off-peak daytime and nighttime hours. For example, in the Chicago Metropolitan Area, freeway lane closures on weekdays are typically limited to a 6-hour period between 9 a.m. and 3 p.m., but some continuous closures are allowed during weekends. These intermittent work zones pose problems to motorists because

- The location of the closure may vary from one day to the next;
- In comparison to the peak hours, the driving population during these periods contains fewer commuters, thus increasing the element of surprise; and
- Motorists are not likely to divert onto alternate routes because the work activities were scheduled only in the previous 24 to 48 hours.

In summary, driver anticipation of the geometric restrictions posed by the presence of the intermittent work zones is considerably less than it is for long-term closures. The ways in which this affects traffic operations and safety during construction or maintenance will be the focus of this paper.

This study had two major objectives. The first objective was to compare the accident experience at both long-term and short-term sites before, during, and after construction or maintenance work. The comparison was accomplished by developing a relative accident rate to allow examination of the variation in accidents at short-term and long-term sites. Because of volume data deficiencies, an absolute accident rate was not developed. Moreover, because of the nature of the comparison, the development of an absolute accident rate was not necessary. The second objective was to make a comparative evaluation of traffic flow and traffic control device layout in terms of adherence to Illinois Department of Transportation (IDOT) standards. This evaluation was undertaken for both short-term and long-term work zones.

LITERATURE REVIEW

The literature abounds with traffic safety and operational studies of work zones. Some authors use the term "construction zone" as a synonym for "work zone." To date, the study by Graham et al. (2) is the most comprehensive, encompassing 79 construction projects in seven states. Their analysis indicated an average increase of 7.5 percent in accidents during construction, although some sites actually experienced a reduction in accidents (the range varied from −3.4 to +37.6 percent by state). All the projects, however, were long term, ranging in duration from 2 months to almost 2 years. Regression models that were developed to consider the relationships between construction accident rates and the project length and duration indicated that long-length and high-duration projects normally exhibit lower accident rates. Whether this trend can be extrapo-

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lated to the short-term sites, however, remains to be seen. Finally, the study found that sites that had the most restrictive geometry during construction (i.e., six- to eight-lane freeways reduced to one lane in each direction) exhibited the sharpest increase in accident rates from the before period. Unfortunately, no results were reported separately for urban and rural Interstate facilities. Data of this type would help identify specific problems associated with urban construction projects.

A cross-sectional study of highway work zones was conducted by Martin and Hargroves in Virginia (*3*). In the study, all reported work zone accidents in Virginia (2,127 in 1977) were analyzed and compared to statewide accident characteristics (142,170 in 1977). The authors found that urban work zone accidents constituted about 1.72 percent of all urban accidents (versus 1.21 percent for rural areas) and that these accidents were more likely to occur on dry pavement (81.5 percent versus 72.7 percent overall), in daylight (69.4 percent versus 63.4 percent overall), and in clear weather (68.6 percent versus 58.2 percent overall). Moreover, passenger automobiles were slightly less involved in work zone accidents (77 percent versus 80.3 percent overall), whereas heavier vehicles were more involved (20 percent versus 15.5 percent overall). In terms of accident type, fixed object collisions were more likely in work zones (10.4 percent versus 2.8 percent overall), but collision accidents remained fairly stable (70 percent versus 70.5 percent overall). Although the results were very informative, many of these statistics were aggregated for all roadway types and for varying project durations, making it difficult to assess the effect of short-term freeway construction projects. A similar approach was adopted by Nemeth et al. (*4*) in their comparison of freeway work zone accidents and all accidents on the Ohio Turnpike.

Other studies were aimed at testing the accident performance of traffic control procedures, such as lane closures and two-lane, two-way operations (TLTWO) by Graham et al. (*5*) and Dudek et al. (*6*). A recent study by Shepard and Cottrell (*7*) alluded to the potential benefits of night work zone activities but provided no information regarding their accident experience. The authors commented that there was a lack of comparable data. A comprehensive summary of the accident literature for highway work zones is available in the final report of the study (*8*). In summary, although the literature provided in-depth coverage of the accident problem at work zones, the specific issues relating to short-term work sites on urban freeways were not fully addressed.

ACCIDENT STUDY

Data Collection

Accident data for this study were provided by the Illinois Department of Transportation Division of Highway Safety. They were available in summary form on a magnetic tape that contained a 6-year accident history (1980–1985) of the Chicago Area Expressway System (CAES). Work zone accidents were identified by matching the locations and activity dates of a selected number of construction projects (three long-term and 23 short-term projects) to the data on the accident tape. This required a thorough examination of the construction logs for each project, including the precise hours during which the work activity was under way and the mileposts or other identifying information for the location of the work. If a project involved a point location rather than a segment of highway, then a segment 1/2 mile upstream and 1/2 mile downstream of the work location was assumed.

One crucial parameter in analyzing accidents is a measure of exposure that is routinely taken as the million vehicle miles (MVM) of travel through the work site in different time periods. This represented a major obstacle in this study for several reasons:

- Average daily traffic (ADT) data for CAES are summarized in odd years only, hence no record of annual ADT was available for this study;
- Communications with IDOT surveillance project personnel indicated that many of the surveillance detectors installed on CAES were not functioning while roadway work was in progress; and
- The nature of intermittent or short-term sites makes it quite difficult to estimate the hourly flow rates during construction so that the vehicle miles of travel can be computed.

Nevertheless, it is precisely because of the nature of these sites that the flow rates should not be expected to vary considerably during construction. The reason is that motorists are typically not advised of the presence of the work in time to alter the course of their trip. In addition, a review of ADT data on CAES revealed that during 1981–1985 the vehicle miles of travel on the system increased by less than 10 percent, while the overall systemwide accident rate was virtually unchanged in the same period (3.43 accidents/MVM in 1981, 3.79 in 1983, and 3.45 in 1985) (*9–13*). As will be explained in the next section, a different exposure measure that takes into account both the duration and length of the work zone was utilized in this study.

Data Analysis

An absolute accident rate is based on the exposure of vehicle miles of travel (equal to ADT * hours * miles). Because the objective of this study was to perform a comparative analysis, it was assumed that, as described previously, ADT will not vary for short-term sites. It is also assumed that ADT will not vary for long-term sites. This is a conservative assumption because ADTs may actually drop in the presence of long-term work zones.

The common (relative) exposure measure derived in this study is expressed as the product of the project duration and length. For long-term projects this procedure is self-explanatory. For intermittent locations (i.e., less than 24 hours), the cumulative hours in which construction was under way were recorded and converted into an equivalent number of full days. For those projects that overlap 2 years, the exposure was computed separately in each year. Thus mile days of exposure/year = $1/24 \, \Sigma \, L * H_c$, in which L equals the project length in miles and H_c equals the hours of construction in which the roadway is occupied.

Because there is only one "during" period and several "before and after" periods, each corresponding to 1 year, the total mile days of travel were multiplied by the number of years in the corresponding analysis period. In essence, a comparison is made of the accident performance of each roadway segment under construction over 6 years (1 year during and 5 years before and after). The designation mentioned previously yielded a sample of four long-term project periods and 25 intermittent or weekend project periods for further analysis. All these projects were undertaken between 1981 and 1983.

Long-Term Sites

Of the four long-term cases, only one experienced a significant increase in accidents during construction. That site involved extensive pavement resurfacing on the I-55 (Stevenson) Expressway between Wolf Road and California Avenue, a distance of roughly 13 mi. This project was by far the most extensive in terms of exposure (1,372 mile days versus 249 combined for the other three cases) as well as accident frequencies (1,147 accidents versus 198 combined for the other three cases). Accident rate summaries for each case are given in Table 1. The results also indicate that in one case no accidents occurred during the 6-year period for the segment under study. It is also evident from Table 1 that the after accident rates are consistently higher than the before rates. When it is noted that the vehicle miles of travel have increased by 10 percent between 1981 and 1985, it appears that this increase in accident rates is in part a reflection of the higher exposure rate during that period. Because Project 34916 is dominant in terms of accidents and exposure, the following analysis will be limited to this project as representative of long-term construction sites.

Initially, mean daily accident occurrences were computed for the "during construction" and "no construction" periods. These were estimated at 3.0 and 1.694 accidents per day, respectively, with a standard deviation of 0.473. A Z-test on the difference between the daily number of accidents under each situation was conducted for the purpose of testing the hypothesis that both values were essentially derived from the same distribution. Simply stated, it was desirable to determine the probability of observing three or more accidents per day on the roadway segment, given that the long-term average (in this case a 5-year average) is 1.694. Mathematically, this can be stated as follows. Find α, such that

$$\text{Prob}\,[Z > Z_\alpha] = \text{Prob}\,[Z > (3 - 1.694)/0.473]$$
$$= \text{Prob}\,[Z > 2.75]$$

yielding $\alpha = 0.003$. Hence it can be stated that the during accident rate is significantly higher than the rates experienced without construction at a 99.97 percent confidence level.

The next series of tests involved specific accident categories. In each, a Z-test on proportions is constructed as follows (*14*):

$$Z = (P_d - P_b)/[p(1-p)\ (1/N_d + 1/N_b)]^{1/2}$$

in which

$$P_d = X_d/N_d$$

$$P_b = X_b/N_b$$

$$p = (X_d + X_b)/(N_b + N_d)$$

where X_d and X_b equal the number of accidents in a specific category (e.g., injuries, rear ends, etc.) in the during and before periods, respectively, and N_d and N_b are the total number of accidents in the during and before periods, respectively.

Similar tests were conducted for the during versus after periods as well. The results are summarized in Table 2 and discussed next.

- Accident severity decreased significantly during construction. For this project, the decrease in fatal and injury accident proportions was over 20 percent, quite consistent with the findings from the literature.
- Rear end accidents increased significantly during construction. For this project, the proportional increase was almost 50 percent, and this result again corroborates the findings from previous studies.
- The presence of construction had a marginal effect on the proportion of object-on-road accidents. This reflects good site management on this project in terms of improving the visibility of TCDs, clearing debris, and so on.
- The proportion of multiple-vehicle accidents increased significantly during construction (by about 15 percent). This is very consistent with the higher occurrence of rear end collisions and points to the problem of increased speed variations between the lane closure and upstream segments.
- Accident categories that were not significantly altered by the presence of construction included sideswipe accidents, heavy vehicle accidents, and those caused by roadway defects (holes, bumps, and low shoulders).
- The proportion of ramp-related accidents increased significantly during construction. In this project, the increase was 45 percent compared to the before period and 142 percent compared to the after period. This was a very important finding that warranted further investigation. A review of the project logs revealed that in the conduct of this project, two specific

TABLE 1 ACCIDENT SUMMARIES FOR LONG-TERM PROJECTS

	Time in Relation to Construction					
	Before		During		After	
Project	Total	Rate	Total	Rate	Total	Rate
34916	282	0.103	300	0.219	565	0.137
35912	0	0	0	0	0	0
35359[a]	27	0.132	13	0.127	47	0.151
35359[b]	60	0.145	8	0.058	43	0.152

NOTE: Rates measured in accident/mile-day of construction.
[a] 1982 construction period.
[b] 1983 construction period.

TABLE 2 ANALYSIS OF ACCIDENT CATEGORIES FOR LONG-TERM PROJECTS

Category	Proportions by Period Before	During	After	Z	Significance Level
Fatal and injury	0.29	0.22		−1.93	0.027[a]
		0.22	0.27	−1.67	0.048[a]
Rear end on road	0.387	0.58		+4.65	0.000[a]
		0.58	0.37	+4.32	0.000[a]
Object on road	0.014	0.03		+1.31	0.095
		0.03	0.026	+0.36	0.36
Sideswipe	0.29	0.27		−0.53	0.300
		0.27	0.28	−0.41	0.350
Holes, bumps, low and soft shoulder	0.018	0.013		+0.49	0.31
		0.013	0.018	+0.50	0.31
Coded repair work	0.018	0.52		+13.52	0.000[a]
		0.52	0.018	+13.50	0.000[a]
Involving multiple vehicles	0.71	0.83		+3.43	0.0003[a]
		0.83	0.74	+2.99	0.0014[a]
Involving heavy vehicles	0.195	0.137		−1.88	0.03[a]
		0.137	0.17	−1.27	0.102
Ramp-related	0.117	0.17		+1.82	0.035[a]
		0.17	0.07	+4.56	0.000[a]

[a]Significant at the 5 percent level.

traffic control procedures were implemented at different points in time:

(a) In the earlier part of the project, the median lane and left shoulder were closed for construction, and traffic was allowed to use the two remaining lanes.

(b) In the latter part of the project, the two right lanes were closed to traffic, and traffic was allowed to only use the left lane and shoulder, a procedure known as traffic shifting (*15*). In this case, traffic entering or leaving the freeway must cross two lanes of traffic with little room for acceleration or deceleration, as shown in the schematic in Figure 1.

A comparison of accident categories under these two procedures is summarized in Table 3. The results indicate the following:

- There was a slight, albeit nonsignificant, increase in accident frequency and severity when the traffic shifting procedure was in effect.
- There is an evident shift in the distribution of rear end accidents. In case a, fewer rear end accidents involved stopped vehicles than in case b. This may be a reflection of the capacity constraints evident in case b and the absence of adequate acceleration-deceleration space at the ramp junctions.

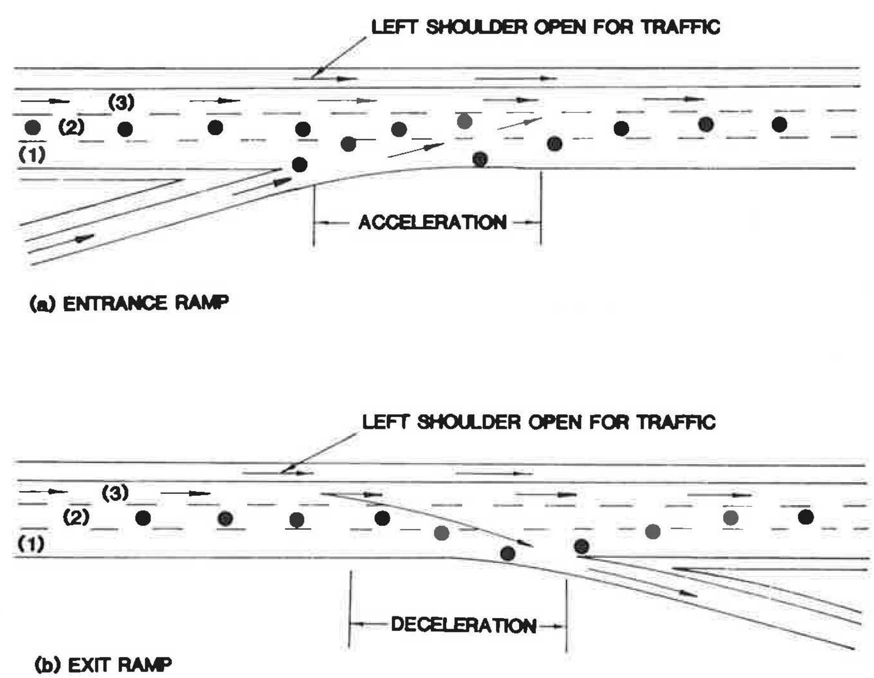

FIGURE 1 Right lane(s) closure procedures near ramp junctions.

TABLE 3 COMPARATIVE ANALYSIS OF CONSTRUCTION ACCIDENT FREQUENCIES BY LOCATION OF LANE CLOSURE(S)

	Case a	Case b	Z	Significance Level
Total accidents	102	169	NA	NA
Daily accidents	2.9	3.38	NA	NA
Proportion of fatal and injury	0.187	0.231	+0.856	0.195[a]
Proportion of rear end, both vehicles moving	0.314	0.266	−0.856	0.195[a]
Proportion of rear end, one vehicle stopped	0.245	0.349	+1.8	0.036[a]
Proportion of ramp-related accidents	0.078	0.231	+3.17	0.0008[b]

NOTE: In Case a, lanes 1 and 2 were open to traffic. In Case b, lane 3 and the left shoulder were open to traffic (see Figure 1).
[a]Significant at 20 percent level.
[b]Significant at 5 percent level.

- The effect of closing the right two lanes is dramatically evident in the occurrence of ramp-related accidents. About 25 percent of all accidents in case b took place in the vicinity of the ramps, in comparison to only 7.8 percent in case a. Thus it appears that the preponderance of ramp-related accidents evident in Table 2 can be attributed primarily to the weaving problem that is encountered at the ramp junction by merging and diverging vehicles. The implications for the layout of TCDs in the ramp vicinity, especially for the rather heavy volumes serviced by this facility (over 100,000 ADT) are evident but fall beyond the scope of this study.

Intermittent or Weekend Projects

A total of 25 cases were incorporated in this analysis. Average accident rates per mile-day were 0.538, 0.78, and 0.67 in the before, during, and after periods, respectively. The rates indicate an increase in accident rates during construction and maintenance. In six cases, no accidents occurred during the 6-year period under study. Preliminary investigation revealed a linear relationship between construction accident frequency and mile-days. Because ADT is approximately constant throughout the before, during, and after periods, the use of mile-days as an exposure measure yields the same results that would have been derived if vehicle miles had been used.

With the exception of the two outliers, accident frequency shows an increase at a rate of 0.80 accident/mile day. A plot of construction accident rate versus mile-days is shown in Figure 2. As expected, no distinct pattern emerges. The implication from this sample is that a fixed accident rate occurs during short-term construction work and that this rate is independent of the length and duration of the work activity. It can also be inferred from the information given previously that in cases for which the control (i.e., before and after) accident rates are low (less than 0.80 accident/mile day), the presence of construction will generally result in an increase in accident rate and vice versa. This phenomenon is best represented graphically, as in Figure 3, in which a set of hypothetical during and control accident rates is plotted. The dashed line represents the "ideal" condition for which the presence of construction has no effect on the accident rate. The solid line represents a best fit to the data, and the two curves depict the upper and lower 95th-percentile estimates of the regression line. Thus the shaded area to the left of point A represents cases that experience a

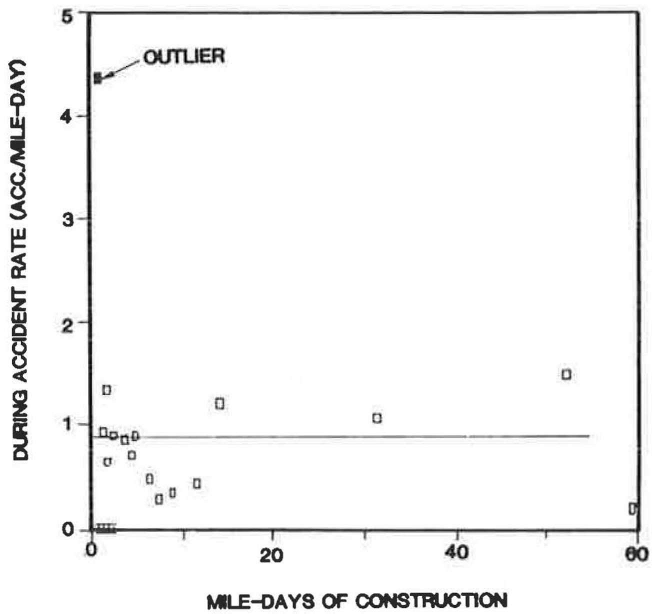

FIGURE 2 Work zone accident rate versus mile-days: short-term sites.

significant increase in accident rates during construction, whereas the shaded area to the right of point B represents cases that experience a significant decrease in accident rate during construction. These tests were carried out for several subsets of accidents (all accidents, rear end accidents, property damage, etc.). The results are summarized in Table 4, which presents the various subsets analyzed, the regression line parameters, and the range of control variables for cases in which significant deviations in work zone accident rates occurred.

- In general, the models yielded a poor fit to the data. Work zone accident rates were significantly higher than the before rates at sites experiencing less than 0.50 accident/mile-day in the before period and less than 0.20 accident/mile-day in the after period. The during accident rates were significantly lower at sites that experienced more than 0.80 accident/mile-day in the after period.
- Property damage only (PDO) accident rates during construction were significantly higher than the before rate when the latter is less than 0.30 accident/mile-day and the corresponding after rate is less than 0.10 accident/mile-day. The

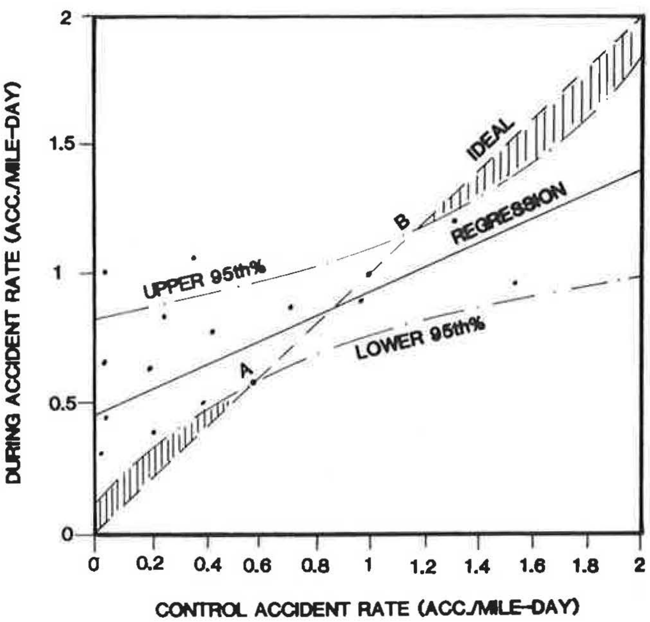

FIGURE 3 During accident rates versus control accident rates at short-term sites: hypothetical model.

during PDO accident rates were lower than the corresponding after accident rates when the latter were above 0.50 accident/mile-day.

• Highway accident rates yielded a moderate fit to the data, showing a similar pattern to the total accident rate. The during highway rates were higher than the corresponding before rates when the latter were less than 0.40 accident/mile-day and higher than the corresponding after rates when the latter were less than 0.10 accident/mile-day.

• Modest correlations were found only between daytime during and before accident rates. The during accident rates were higher then the corresponding before rates when the latter were less than 0.40 accident/mile-day. This is not unexpected because all construction activities in this category were carried out primarily during daytime off-peak hours.

• The other accident rates presented in Table 4 gave very poor fit to the data and did not produce any meaningful results for work zone accident rates.

Summary

In summary, the study indicated that there is evidence at long-term lane closures of an increase in accident rates during construction. On urban freeways, which were the focus of the study, most accidents appear to be attributable to the capacity constraints imposed by the lane closure and the resulting speed variations between the bottleneck and the approach areas to the zone. This is demonstrated by the preponderance of rear end and multiple vehicle accidents, which also tended to reduce the overall accident severity during construction. The problem was even more acute at ramp junctions, where the capacity problems are compounded by the merging and diverging traffic in relatively short distances (as is evident in Figure 1).

At short and intermittent construction sites, an average accident rate of 0.80 accident/mile-day was observed; the rate appears to be independent of the length or duration of the work

TABLE 4 SUMMARY OF ACCIDENT RATES FOR SHORT-TERM PROJECTS

	Accident Rate Categories			Regression Line		Range of Control Variables for Significant Deviations During Acceleration	
Model	During	Control	R^2	Slope	Intercept	Increase	Decrease
1	Total	Total, before	0.32	0.29	0.63	0–0.5	NS
2	Total	Total, after	0.14	0.36	0.27	0–0.2	0.8–1.8
3	PDO	PDO, before	0.36	0.21	0.69	0–0.3	NS
4	PDO	PDO, after	0.29	0.19	0.47	0–0.1	0.8–1.4
5	Rear end	Rear end, before	0.29	0.15	0.64	0–0.2	NS
6	Rear end	Rear end, after	0.16	0.14	0.39	0–0.1	0.5–1.4
7	Object on road	Object on road, before	NA	NA	NA	NA	NA
8	Object on road	Object on road, after	NA	NA	NA	NA	NA
9	Wet/other	Wet/other, before	0.05	0.10	−0.27	0–0.03	0.15–0.6
10	Wet/other	Wet/other, after	0.01	0.09	−0.04	0–0.1	0.2–1.8
11	Repair work	Repair work, before	0.00	0.05	−0.37	NS	NS
12	Repair work	Repair work, after	0.02	0.05	−0.08	NS	0.1–1.4
13	Road defects	Road defects, before	0.00	0.02	0.12	NS	NS
14	Road defects	Road defects, after	0.00	0.02	0.06	NS	NS
15	Highway	Highway, before	0.33	0.30	0.73	0–0.4	NS
16	Highway	Highway, after	0.33	0.23	0.57	0–0.1	NS
17	Daytime	Daytime, before	0.21	0.30	0.57	0–0.4	NS
18	Daytime	Daytime, after	0.08	0.37	0.21	0–0.2	0.8–1.8
19	Rain/other	Rain/other, before	0.05	0.11	−0.35	0–0.03	0.18–0.48
20	Rain/other	Rain/other, after	0.01	0.09	−0.03	NS	0.2–1.8
21	First car	First car, before	0.28	0.26	0.65	0–0.4	NS
22	First car	First car, after	0.20	0.24	0.43	0–0.1	1.0–1.6
23	First tractor-trailer	First tractor-trailer, before	0.00	0.04	0.01	NS	>0.1
24	First tractor-trailer	First tractor-trailer, after	0.04	0.03	0.19	NS	0.16–0.48

NOTE: R^2 is a coefficient of determination. NS signifies "not significant"; NA, "no accidents."

zone. Again, accident severity appears to be lower during construction, but rear end collisions are higher. Thus the effect of short-term construction on accidents is in fact dependent on the accident history of the segment during other periods.

TRAFFIC FLOW AND CONTROL DEVICES STUDY

The purpose of this study was to provide a comparative evaluation of traffic control layouts for short- and long-term urban freeway lane closures. Evaluation criteria were derived from IDOT standards, which apply to both types of closure. Part of the analysis also focused on the problem of speed variations in the work zones, as determined from the accident study, and attempts were made to investigate correlations between TCD layouts and speed variance at the approach, transition, and closure areas.

Data Collection

An instrumented data collection system was specifically devised for this study. The system utilizes the floating automobile concept for speed measurements and is supplemented with a video recording system to gather information on the presence, location, and indication of TCDs and construction or work activity. The hardware consisted of the following:

- Two on-board video cameras, one aimed at the roadway and the other at the vehicle dashboard. The first recorded the placement and indication of traffic control devices while the second recorded speed observations.
- One ½-in. portable VCR with real-time display of 1/30-sec accuracy. A schematic of the system is shown in Figure 4. An audio channel over which information regarding site description is recorded supplements the video input. This audio link it was also used to identify sign locations that were obscured by traffic.

In all, 150 construction sites were visited. Because of time constraints, only 46 sites were coded for further analysis. The analysis group included both short- and long-term sites during the day and at night. A FORTRAN code program was developed to produce summary reports of the TCD layout, as well as the speed profile of the instrumented vehicle. A review of IDOT standards indicated there were three designations of work zones for TCD layout requirements:

- Short-term sites, at which construction lasts less than 6 hr in daytime (35 sites comprising 14 single-lane closures and 21 two-lane closures);
- Intermediate sites, at which construction lasts over 24 hours but less than 4 days (seven sites comprising four single-lane closures and three two-lane closures); and
- Long-term sites, at which construction lasts more than 4 days (four sites, all involving single-lane closures).

TCD Layout Results

Because the primary purpose of the warning signs is to provide adequate response time to the lane closures for approaching traffic, all the standards and actual locations of these devices

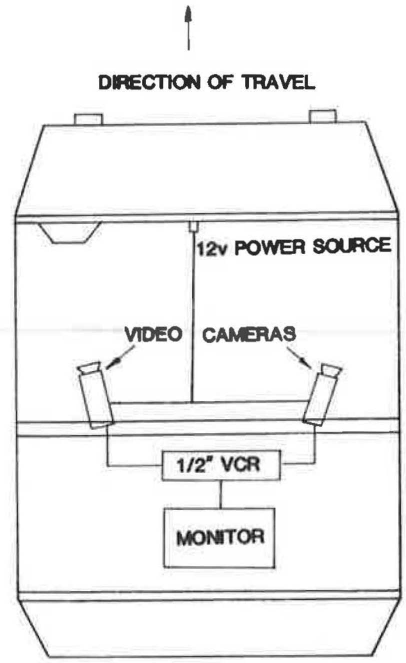

FIGURE 4 Schematic of instrumented vehicle for traffic study.

were computed from the start of the transition taper. Table 5 summarizes the results of the four long-term sites. Except for one site, all the required devices were present at all sites. The positions of the devices, however, varied considerably from the standards, especially the Road Construction Ahead sign, which on the average was placed 1,400 ft closer to the taper than required by standards. The position of the sign also varied considerably from site to site, with a standard deviation of almost 1,200 ft. The same variation was found for all other warning signs, which were also placed closer to the taper than allowed by standards. Their locations, however, were far more consistent than that of the first sign, as is evident from the much smaller standard deviation. Finally, the actual taper lengths appear to conform very closely to standards. A similar pattern emerged for sign locations at intermediate work sites with single-lane closures. In addition, the Right/Left Lane Closed ½ Mile was missing in three of the four sites. Interestingly, the standard deviations of sign positions were consistently higher at these sites than those at the long-term sites. Table 6 summarizes the results for the short-term sites. In this case, several required devices were missing at a number of sites, including the arrow board. The first two signs were again placed closer to the taper, while the next two signs were placed further from the taper compared with the standards. Deviations of sign positions between sites were extremely high, varying by as much as 70 percent of the mean in some cases. The data for intermediate and short-term sites with two-lane closures were consistent with previous findings in that (a) signs were placed closer to the taper than required by standards, (b) many devices were missing at the short-term sites, averaging 2.5 missing devices per site, and (c) sign positioning appears to be very inconsistent at the short-term sites, as evidenced by the high standard deviations.

TABLE 5 TCD LAYOUT FOR LONG-TERM (> 4 DAYS) SITES, SINGLE-LANE CLOSURE

Device	Requirement Status	Number of Sites Not Present	Device Position (ft)[a] IDOT Standard Location	Observed Mean	Observed Standard Deviation of Location (ft)
Road Construction Ahead (RCA)	Required	0	7,800	6,483	1,191
Right/Left Lane(s) Closed 1 miles (LC1)	Required	0	5,200	4,706	145
Right/Left Lane(s) Closed 1/2 mile (LC2)	Required	1	2,600	1,762	42
Right/Left Lane(s) Closed Ahead (LC3)	Required	0	1,500	1,293	103
First Symbolic Lane Drop Sign (SLC1)	Required	0	500	645	105
Second Symbolic Lane Drop Sign (SLC2)	Not required	NA[b]	NA	NA	NA
First Arrow Board (AB1)	Required	0	220	403	62
Second Arrow Board (AB2)	Not required	NA	NA	NA	NA
Taper Length (TAPER1)	Required	0	660	655	115
Tangent Between Tapers (TANG)	Not required	NA	NA	NA	NA
Second Taper Length (TAPER2)	Not required	NA	NA	NA	NA

NOTE: N = four sites.
[a]Measured from the start of the first taper.
[b]Not applicable.

TABLE 6 TCD LAYOUT FOR SHORT-TERM (≤ 6 DAYS) SITES, SINGLE-LANE CLOSURE

Device	Requirement Status	Number of Sites Not Present	Device Position (ft)[a] IDOT Standard Location	Observed Mean	Observed Standard Deviation of Location (ft)
Road Construction Ahead (RCA)	Not required	7	7,800	6,000	2,063
Right/Left Lane(s) Closed 1 miles (LC1)	Required	6	5,200	3,979	1,612
Right/Left Lane(s) Closed 1/2 mile (LC2)	Required	5	2,600	3,196	542
Right/Left Lane(s) Closed Ahead (LC3)	Required	2	1,500	1,980	722
First Symbolic Lane Drop Sign (SLC1)	Required	2	500	1,236	818
Second Symbolic Lane Drop Sign (SLC2)	Not required	NA[b]	NA	NA	NA
First Arrow Board (AB1)	Required	3	220	380	296
Second Arrow Board (AB2)	Not required	NA	NA	NA	NA
Taper Length (TAPER1)	Required	0	660	499	243
Tangent Between Tapers (TANG)	Not required	NA	NA	NA	NA
Second Taper Length (TAPER2)	Not required	NA	NA	NA	NA

NOTE: N = 14 sites.
[a]Measured from the start of the first taper.
[b]Not applicable.

Speed Distribution Results

At each of the visited sites the mean and standard deviation of the test vehicle speed was computed along three subsections:

- Approach (from the location of the first construction sign to the start of the lane taper),
- Transition (in the taper zone), and
- Closure (along the closed portion of the work zone).

The driver of the test vehicle was instructed to travel in the closed lane of traffic, merging at or near the taper when appropriate. The following flow descriptors were then derived:

Mean Speed Differences Between Zones

The amount of speed reduction between the approach and transition and between transition and closure areas is indicative of potential rear-end collisions because drivers must adjust their speed ahead of the lane restrictions.

Speed Coefficient of Variation (CV) by Zone

This is defined as

CV = (Speed Standard Deviation/Zone Mean Speed) * 100

In essence, CV normalizes the amount of speed variation by the average speed to distinguish between free flow conditions in

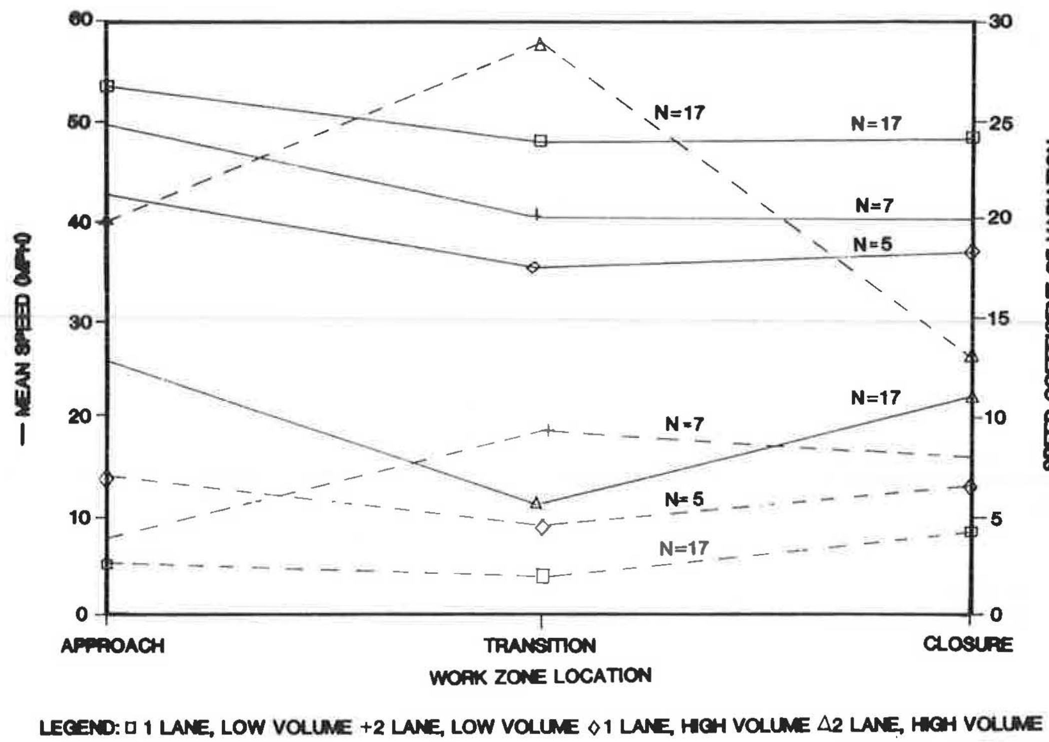

FIGURE 5 Composite speed profile versus number of lane closures and traffic volumes.

which speed variation is merely reflective of the driver's desired speed (i.e., low CV) and those due to friction between vehicles in the traffic stream (i.e., high CV). Because speeds are intimately tied to traffic volumes, all subsequent analyses were performed separately for high- and low-volume sites.

Mean speed profiles at the three zones are presented in Figure 5. The average drop in speed between the approach and transition zones increased with volume and the number of closed lanes. The observed values for single lane closures were 5.45 and 7.19 mph under low- and high-volume conditions, respectively. The corresponding values for two lane closures were 9.64 and 14.58 mph. Between the transition and closure zones the average speed did not vary considerably under light volume, increasing by 0.40 mph for single-lane closures and decreasing by 0.10 mph for two-lane closures. Speed recovery was more significant under high-volume conditions, increasing by 1.50 mph and 10.80 mph for single- and two-lane closures, respectively. Thus the amount of speed recovery in the closure area appears to be dependent on the loss of speed in the transition area. In combination with the preponderance of rear end accidents during construction, the results indicate that speed control on the approach and transition areas is of paramount importance in reducing the frequency of these accidents. It is also evident that the transition area, and not the closure area, governs the capacity of the work zone because of the number of lane change maneuvers taking place. Smooth merging thus becomes a prerequisite for enhancing the overall zone capacity.

Speed coefficients of variations are also depicted in Figure 5. For sites with two-lane closures, CV increased in the transition zone. The amount of increase was much higher in congested traffic. This confirms the results obtained previously, which are indicative of the disturbances caused by the multiple lane changes that occur in the transition area. On the other hand, CV decreased in the transition zone when single-lane closures (i.e., fewer merges) were in effect. Again, the magnitude of CV is higher under high-volume conditions.

Finally, an attempt was made to analyze zonal speed characteristics (specifically CV) with regard to the TCD layouts described in the previous section. So that the effect of traffic volumes could be excluded, observations from heavy volume sites were not considered in this analysis. The results are summarized in Table 7. In all three zones, short-term sites exhibited higher coefficient of speed variations in comparison to the intermediate and long-term sites. This was especially true in the transition zone, where the increase was over 400 percent. In addition, when either some of the warning signs or the arrow board were missing, the coefficient of speed variation increased. Finally, when the taper lengths were shorter than allowed by standards, the speed coefficient of variation increased by an average of 35 percent. One unexpected result is that during the conduct of work activity, there were fewer speed variations than when no activity was under way.

To summarize, the field observations of TCD layout and speed measurements at the 46 construction zones indicate inconsistent TCD positioning according to IDOT standards. This is much more so at short and intermittent sites. The resulting speed analyses revealed that such inconsistencies are concomitant with larger speed variations in the approach and transition zones, thus corroborating the accident findings with regard to the increased frequency of rear end collisions.

CONCLUSIONS AND RECOMMENDATIONS

This paper documents the findings of a comprehensive study aimed at investigating and comparing the safety and operational aspects of urban freeway work zones in the Chicago Metropolitan area. The following conclusions are drawn:

• At long-term lane closures (longer than 4 days), accident frequency increased and accident severity decreased during construction. The predominant accident types were rear end collisions and ramp-related accidents, especially when the lane closures involved the two right lanes adjacent to the entrance and exit ramps.

TABLE 7 COEFFICIENTS OF SPEED VARIATIONS VERSUS TCD LAYOUT AT LOW-VOLUME SITES

Factors	Value	Sample Size
Approach Area (CV1)		
1a Short-term sites	3.68	15
1b Intermediate and long-term sites	1.83	9
2a Arrow board present	2.87	22
2b Arrow board missing	4.27	2
3a All warning signs present	1.99	5
3b One or more warning signs missing	3.20	19
Transition Area (CV2)		
1a Short-term sites	5.65	15
1b Intermediate and long-term sites	1.11	9
2a Taper length less than standard	4.36	18
2b Taper length greater than or equal to standard	3.22	6
Closure Area (CV3)		
1a Short-term sites	5.67	15
1b Intermediate and long-term sites	4.67	9
2a No work activity	6.24	13
2b Work activity	4.13	11

NOTE: $N = 24$ sites.

• At intermittent or weekend closures, the accident rate during construction appears to be constant at about 0.80 accident/mile-day of work activity. Thus road segments that had a typical (i.e., without construction) accident rate of less than 0.80 accident/mile-day indeed experienced an increase in accidents during construction, and vice versa. This conclusion would still be valid if traffic volumes were introduced in the computation of accident rates: the ADT on a highway segment will not vary significantly due to the presence of a short-term construction site.

• At the 46 sites analyzed by the research team, there were discrepancies between observed and standard positioning of TCDs. In general, signs were placed much closer to the taper than allowed by standards. The deviations were of higher magnitude at short-term lane closures, as were the occurrences of missing TCDs. Speed profiles indicated that sites that had short tapers, missing arrow boards, and missing signs or that were of short duration exhibited significantly higher speed variations than other sites. This points to the importance of adhering to standards, even though the overall exposure to traffic may be quite limited.

It is evident from these findings that much work remains to be done in evaluating the safety and operational characteristics of highway work zones adequately. Continuing research needs are as follows:

Development of Uniform Standards for Reporting Work Zone Accidents

This study indicates that except for long-term activities, many accidents cannot be routinely identified from the accident tape. Indeed, only about 10 percent of accidents known to have occurred during construction were identified as such from the accident tape. Suggested modifications to current reporting procedures should be explored. The possibility of developing a supplemental work zone accident information sheet is one such modification (*16*).

Further Evaluation of the Safety Impacts of the Traffic Shifting Procedure

In this study, shifting traffic to the left lane and shoulder has resulted in doubling the proportion of ramp-related accidents from the before condition. The implications regarding the layout of the traffic control devices near the ramp area must be explored in greater detail.

Development of a User-Based Cost Model for Freeway Work Zones

The comprehensive data collected at 150 sites in the course of this study can be used to derive representative vehicle operating costs due to excess travel time, speed changes, and fuel consumption. The outcome of this study will be a model that is sensitive to the type of construction activity, number of lanes closed, traffic conditions, and visibility conditions. This information can be used in planning future work zone activities and may be supplemented with accident costs to evaluate the costs of alternative traffic control plans.

Work Zone Capacity Estimation

In contrast to the findings from the literature, it was observed that vehicle speeds (and, in essence, the work zone capacity) are primarily governed by traffic flow in the transition area rather than in the closure area itself. Therefore merging capacity appears to be a better indicator of the bottleneck capacity, especially at high approach volumes. A future study will utilize the collected operational data at construction zones to refine the capacity estimates in existing literature.

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Safety Effects of Two-Lane Two-Way Segment Length Through Work Zones on Normally Four-Lane Divided Highways

PATRICK T. MCCOY AND DAVID J. PETERSON

In 1979, an equation was derived for the optimum length of two-lane two-way (TLTW) segments through work zones on normally four-lane divided highways. Solution of this equation for the prevailing conditions on I-80 in Nebraska in 1986 yielded optimum segment lengths that were about 60 percent longer than those used previously in Nebraska. Because of concern expressed about the safety of longer TLTW segments, the applicability of the longer optimum lengths was questioned. The objective of this study was to determine the effects of longer TLTW segments on the safety of traffic operations. The study examined the relationships between segment length and five speed distribution parameters used as indicators of traffic safety. The relationship between segment length and TLTW accident rate was also examined. No relationships were found between TLTW segment length and accident rate or any of the speed distribution parameters. It was concluded that there is no relationship between TLTW segment length and the safety of TLTW operations for the conditions studied.

In 1979, McCoy et al. (*1*) derived an equation for the optimum length of two-lane two-way (TLTW) traffic operations in construction zones on rural four-lane divided highways. The equation, which was derived by using the methods of calculus, defined the optimum length as the length that would minimize the sum of the additional road user and traffic control costs resulting from the construction project. The equation for optimum segment length (l_o) was expressed as follows:

$$l_o = \left[\frac{LC_x/(\text{ADT}.D)}{(10^{-8})\ (c_{al}\, a_l - c_{an}\, a_n) + c_T\,(1/v_l - 1/v_o) + c_{ol}} \right]^{1/2} \quad (1)$$

where

L = project length (mi);
C_x = average cost of constructing a crossover system, which consists of two median crossovers, and providing traffic control devices on the two crossovers and their approaches (dollars per crossover system);
ADT = average daily traffic volume (vehicles/day);
D = project duration (days under TLTW operations);
c_{al} = average cost per segment accident during TLTW operations (dollars);
a_l = segment accident rate during TLTW operations (accidents/100 million vehicle miles, or MVM);
c_{an} = average cost per segment accident during four-lane divided operations (dollars);
a_n = segment accident rate during four-lane divided operations (accidents/100 MVM);
c_T = unit value of time (dollars/vehicle hour);
v_l = average overall speed of TLTW operations (mph);
v_o = average overall speed of four-lane divided operations (mph); and
c_{ol} = increase in average vehicle operating costs due to TLTW operations (dollars/vehicle mile).

Although the functional relationship expressed by this equation remains valid, the values of the unit cost factors used in its solution have changed considerably since the equation was first derived. In addition, improved traffic control measures implemented by the Nebraska Department of Roads (NDOR) have reduced the frequency and severity of accidents that occur with TLTW operations in construction zones. Therefore optimum segment lengths computed with Equation 1 using 1979 unit costs and accident rates are not applicable to current conditions. To compute segment lengths that are appropriate for current conditions, current values for the unit costs and accident rates must be used in Equation 1.

In 1986, McCoy (*2*) conducted a study to update the unit cost factors and accident rates used in Equation 1 to reflect current conditions on I-80 in Nebraska. Accident, delay, and vehicle operating cost analyses were performed to update the road user cost factors in Equation 1. For prevailing roadway and traffic conditions representative of those on I-80 in Nebraska, optimum segment lengths computed using the 1986 values in Equation 1 were found to be about 60 percent longer than those computed using the 1979 values. Optimum segment lengths were generally found to be in the range 3.0 to 5.0 mi with the 1979 values and in the range 4.8 to 8.0 mi with the 1986 values.

However, concern has been expressed about the safety of longer segment lengths. On the basis of intuition and limited data, Pang (*3*) concluded that longer segments tended to

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experience higher accident rates. The Traffic Control Devices Handbook (*4*) states that:

> When segments of TLTWO exceed 3 to 5 mi, there may be operational problems due to lower speed vehicles which can make a section more prone to rear end accidents . . . . Where separation devices are other than positive barrier, there is a high probability of illegal passing maneuvers on longer sections due to frustration and impatience . . . . Therefore, from a safety standpoint short segments are desirable.

This concern about the safety effects of longer segments brings into question the applicability of the optimum lengths computed with Equation 1 using the 1986 values. First of all, in Equation 1, the segment accident rate during TLTW operations (a_l) is a constant value and not a function of segment length. Therefore, if in fact a_l is not a constant and does vary with segment length, the optimum segment lengths computed with Equation 1 would not be valid, and Equation 1 would have to be revised to account for the relationship between a_l and segment length.

Second, the 1986 value used for a_l (30.8 accidents/100 MVM) was the average of the accident rates experienced on 24 TLTW segments on I-80 in Nebraska between 1978 and 1984. These rates, which are presented in Table 1, range from 0 to 178.1 accidents/100 MVM. Although the results of simple linear regression analysis, presented in Table 2, indicate that there is not a statistically significant linear relationship between a_l and segment length, only 2 of the 24 TLTW segments were longer than 5 mi. If segments longer than 5 mi would happen to have average accident rates greater than 30.8 accidents/100 MVM, then the optimum segment lengths computed with Equation 1 using the 1986 values would not be correct. Instead, they would be longer than the "true" optimum segment lengths.

OBJECTIVE

The objective of the study presented in this paper was to determine the effects of longer TLTW segments on the safety of traffic operations. The study examined the relationships between segment length and five speed distribution parameters that have been used as indicators of traffic flow safety. In addition, the relationship between TLTW segment accident rate and segment length was examined. The procedure, findings, and conclusion of the study are presented in this paper.

PROCEDURE

The study procedure involved the conduct of spot speed studies and the analysis of accident experience on four TLTW segments on I-80 in Nebraska in 1986. The segments that were studied are presented in Table 3. They were selected because their lengths, which ranged from 6.68 to 7.22 mi, were longer than the usual maximum segment length of 5 mi. All of the segments were located on level terrain in western Nebraska.

The pavement markings used on each segment are illustrated in Figure 1. The segment cross section consisted of two traffic lanes about 12 ft wide, a 3-ft median, a 3-ft paved shoulder on one side, and a 6.5-ft paved shoulder on the other side. This cross section was possible because the pavement structure of the shoulders on I-80 was of the same strength as the traffic lanes. The median used to separate the opposing traffic lanes consisted of two bidirectional yellow raised pavement markers centered 3 ft apart and installed at 10-ft intervals, plus 18-in. tubular posts installed at 200-ft intervals. Two-way traffic warning signs and "do not pass" regulatory signs were installed on each side of the roadway at ½-mi intervals.

Spot Speed Studies

Spot speed studies were conducted at five locations on each TLTW segment. As shown in Figure 2, each segment was divided into thirds. In the direction in which traffic did not have to cross over the median, spot speed data were collected at three points (the one-third point, two-thirds point, and at the end of the segment). Data were not collected at the beginning of the segment in this direction because of the influence of the transition from four-lane divided to TLTW operations. In the other direction, in which traffic had to cross over the median, spot speed data were collected at only two locations (the one-third and two-third points). Data were not collected at the beginning and end of segment in this direction because of the influence of the crossovers.

The spot speed data were collected by means of radar during daytime hours. All observations made were of free-flowing vehicles during free-flowing conditions (level of service B or better).

Previous research has determined that certain parameters of speed distributions can be used as indicators of the safety of traffic operations. A number of studies have concluded that speed variance and accident frequency are directly related (*5*). From a study of accident data for rural highway sections, Solomon (*6*) found a relationship between accident rate and speed variation. According to this relationship, the accident involvement rate of a vehicle increases as its speed varies from the average speed of traffic. In an AASHTO study of accident experience on Interstate highways (*7*), it was determined that accident rates decreased as the percentage of traffic traveling in the 10-mph pace increased. In another study, Taylor (*8*) found that a relationship exists between the accident rate and the speed distribution on rural highways. He determined that the safest traffic operations occur when speeds are normally distributed and that the best parameter to use as an indicator of the safety of traffic operations is the skewness of the speed distribution.

Therefore, for the purpose of this study, the following speed distribution parameters were used as indicators of the safety of traffic operations:

- standard deviation,
- range,
- percentage in the 10-mph pace,
- skewness, and
- expected accident involvement rate.

It was assumed that the safety of traffic operations improved with higher values of the percentage in the 10-mph pace. Conversely, it was assumed that the safety of traffic operations worsened with higher values of the other four parameters. The expected accident involvement rate was computed by applying the observed speed distribution to the relationship between

TABLE 1 ACCIDENT RATES UNDER TLTW OPERATIONS

Project	Year	Segment	Length (miles)	ADT	Days of Operation	No. of Accidents	Accident Rate (acc/100 MVM)
IR-80-6(37)	1978	1	2.63	11,800	30	0	0
		2	3.48	9,800	33	2	178.1
IR-80-7(55)	1978	1	4.83	15,700	72	2	36.7
IR-80-3(71)	1979	1	3.85	10,100	46	0	0
		2	3.71	13,500	38	0	0
		3	3.66	12,400	43	0	0
IR-80-5(31)	1979	1	3.43	10,900	40	0	0
		2	3.81	9,100	71	2	81.5
IR-80-7(56)	1979	1	4.49	9,700	47	0	0
IR-80-4(60)	1980	1	3.82	9,100	36	0	0
		2	3.60	11,900	27	0	0
IR-80-4(64)	1980	1	3.24	9,500	36	0	0
		2	2.43	11,100	27	0	0
IR-80-4(58)	1980	1	4.57	6,600	68	1	48.4
		2	4.80	8,000	41	1	63.8
IR-80-5(33)	1980	1	3.10	10,900	57	1	51.7
		2	3.26	12,500	40	1	61.2
		3	3.81	10,900	56	0	0
IR-80-4(66)	1981	1	3.29	11,300	49	1	54.9
		2	4.23	8,100	71	0	0
IR-80-7(68)	1983	1	4.91	14,500	114	5	61.6
IR-80-7(72)	1984	1	4.91	10,500	93	0	0
IR-80-6(48)	1984	1	6.46	9,100	65	3	78.5
IR-80-7(73)	1984	1	6.78	10,700	174	3	23.7
						Average	30.8

TABLE 2 ANALYSIS OF VARIANCE FOR REGRESSION ANALYSIS TLTW ACCIDENT RATES FROM PREVIOUS STUDY

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F	Prob > F
Due to b_0	1	22,822.83	22,822.83		
Due to b_1/b_0	1	908.86	908.86	0.472	0.499
Residual	22	42,340.89	1,924.59		
Total	24	66,072.58			

NOTE: $\hat{Y} = b_0 + b_1X$, where $\hat{Y}$ is the segment accident rate during TLTW operations (accidents/100 MVM), b_0 is the y intercept, b_1 is the slope, and X is the segment length (mi).

TABLE 3 TLTW SEGMENTS STUDIED

Project	Segment Length (mi)
IR-80-3(81), Sutherland West	6.78
IR-80-3(88), Hershey East	6.86
IR-80-4(82), Brady East	7.22
IR-80-5(44), Elm Creek–Odessa westbound	6.68

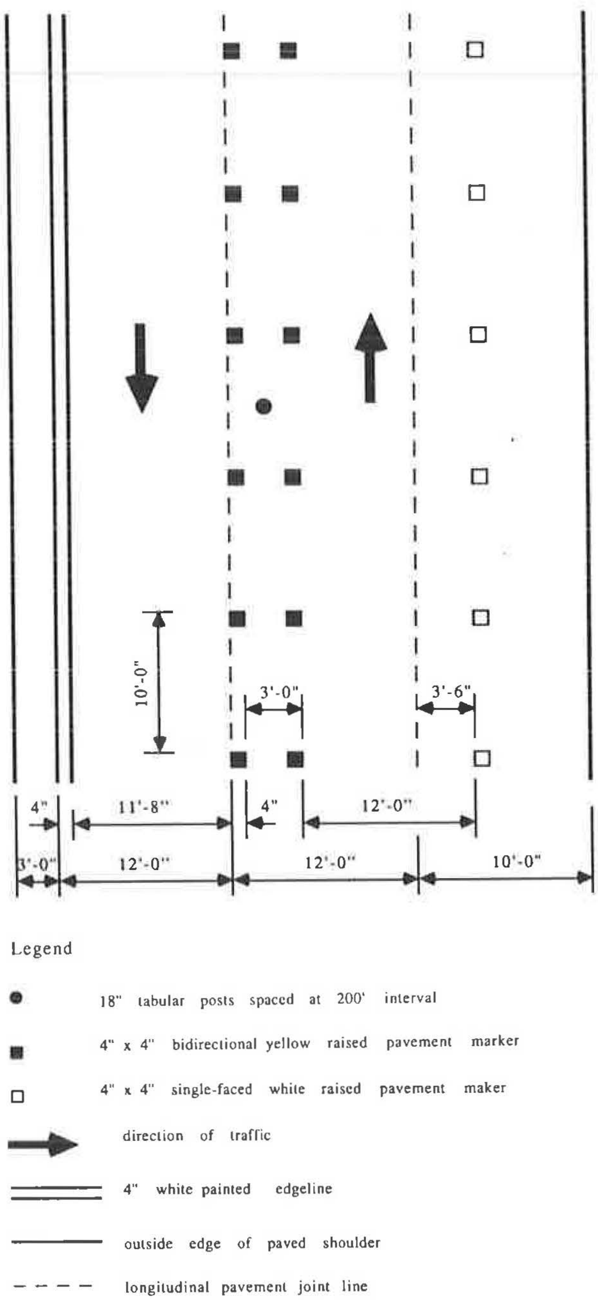

FIGURE 1 TLTW segment pavement markings.

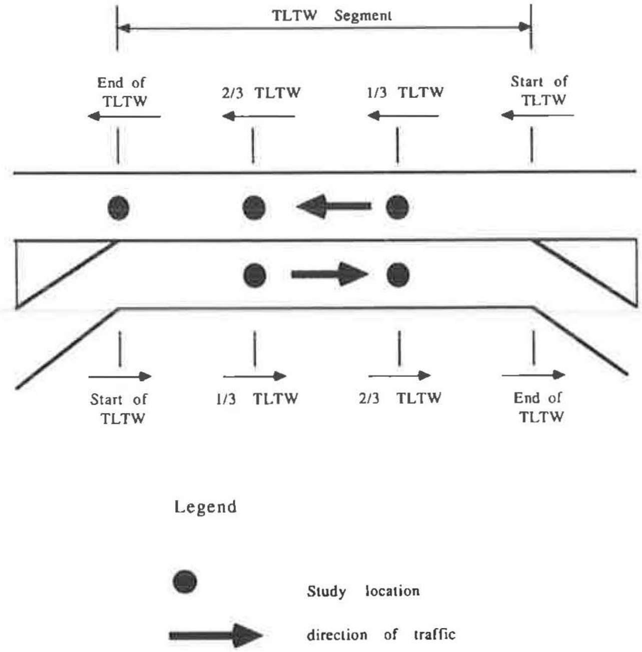

FIGURE 2 Spot speed study locations.

accident involvement rate and speed variation, which was determined by Solomon (*6*).

The five speed distribution parameters were computed from the spot speed data collected at each study location in the TLTW segments. Spot speed studies were conducted at 20 locations (five locations in each of the four TLTW segments). Thus 20 sets of speed distribution parameters were computed. In addition, the distance that each study location was from the beginning of TLTW operations was determined.

Next, a series of simple linear regression analyses was performed to determine the relationship between each speed distribution parameter and the distance from the beginning of TLTW operations. The regression model used was

$$Y_i = \beta_0 + \beta_1 X_i + \varepsilon_i \tag{2}$$

where

Y_i = value of parameter at the *i*th study location;
β_0 = *y*-intercept regression parameter;
β_1 = slope regression parameter;
X_i = distance of the *i*th study location from beginning of TLTW operations (mi); and
ε_i = random error at the *i*th study location.

The regression parameter of particular interest in this study was β_1. If β_1 were equal to zero, then there would have been no linear relationship between the speed distribution parameter and the distance from the beginning of TLTW operations. This would indicate that on the basis of the particular speed distribution parameter involved, the safety of traffic operations was not related to the lengths of TLTW segments. If β_1 were not equal to zero, it would indicate that the safety of traffic operations was related to the lengths of TLTW segments.

Accident Analysis

The accident reports for all reported accidents that occurred in each segment during TLTW operations were obtained from the Nebraska Department of Roads. These reports were reviewed to ensure that all of the reported accidents did in fact occur on the TLTW segment and not on the median crossovers or their approaches. Accident rates were computed for the TLTW segments. These rates, together with the TLTW segment accident rates from the previous study (2), were analyzed by using simple linear regression to determine the relationship between the accident rates on TLTW segments and the lengths of TLTW segments. The regression model used was

$$Y_i = \beta_0 + \beta_1 X_i + \varepsilon_i \tag{3}$$

where

Y_i = accident rate for the ith TLTW segment (accidents/100 MVM);
β_0 = y-intercept regression parameter;
β_1 = slope regression parameter;
X_i = length of ith TLTW segment (mi); and
ε_i = random error for ith TLTW segment.

As was the case in the analysis of the speed distributions, β_1 was the regression parameter of interest. If β_1 were equal to zero, then there would be no linear relationship indicated between the TLTW accident rate and segment length. On the other hand, if β_1 were not equal to zero, a linear relationship between these two variables would be indicated.

SPEED DISTRIBUTION PARAMETERS

Over 100 spot speeds were observed at each of the 20 study locations in the four TLTW segments. The number of observations made at each location, as well as the speed distribution parameters computed from these data, are shown in Tables 4–7. Each of these tables contains the values of the speed distribution parameters computed from the spot speed data collected at the five study locations in one of the four TLTW segments. Also presented in these tables are the mileposts at the ends of the TLTW segments and those at the two intermediate data collection points. The distances from the beginning of TLTW operations, which are also shown in these tables, were computed from these mileposts. As mentioned previously, data were not collected at the beginning of TLTW operations in either direction or at the end of TLTW operations in the direction of the traffic that had to cross over the median. Data were not collected at these locations because of the influence of the crossovers, the transition from four-lane divided to TLTW operations, or both.

The results of the analysis of variance for the simple linear regression analysis of each speed distribution parameter are shown in Table 8. In each case, no statistically significant linear relationship was found between the speed distribution parameter and the distance from the beginning of TLTW operations. Therefore these results indicate that the safety of TLTW operations, as measured by these parameters, was not related to the length of the TLTW operations for the conditions studied.

ACCIDENT RATES

A total of 15 accidents was reported on the four TLTW segments. Only five of these reports, however, were for accidents that actually occurred on the TLTW segments. The other 10 were for accidents that occurred on the median crossovers or on the approaches of the TLTW segments. Of the five accidents that occurred on the TLTW segments, two involved collisions with deer. To be consistent with the procedures of the previous study (2), these two accidents were eliminated from the analysis because the occurrence of this type of accident depends primarily on the population of deer along I-80 and not on the design and operation of the TLTW segments. Therefore only three accidents of interest occurred on the four TLTW segments.

None of the three TLTW accidents resulted in a fatality. Two of them were property damage–only accidents, one a rear end collision and the other a vehicle running over an object lying in the roadway. The third accident was a nonfatal injury accident in which a vehicle attempting to make a U-turn from one

TABLE 4 SPEED DISTRIBUTION PARAMETERS FOR PROJECT IR-80-3(81), SUTHERLAND WEST

	Mile Post			
	150.96	153.00	155.51	157.74
Eastbound				
Distance (mi)[a]	0.00	2.04	4.55	6.78
Number of observations		113	156	160
Standard deviation (mph)		5.0	4.6	4.0
Range (mph)		24	21	21
Percentage in 10-mph pace		72	72	84
Skewness		+0.14	–0.33	+0.20
Accident involvement rate (accidents/100 MVM)		140	100	140
Westbound				
Distance (mi)[a]	6.78	4.74	2.23	0.00
Number of observations		193	154	
Standard deviation (mph)		6.3	4.7	
Range (mph)			35	25
Percentage in 10-mph pace		65	73	
Skewness		–0.14	–0.14	
Accident involvement rate (accidents/100 MVM)		170	140	

[a]Distance from beginning of TLTW operations.

TABLE 5 SPEED DISTRIBUTION PARAMETERS FOR PROJECT IR-80-3(88), HERSHEY EAST

	Mile Post			
	164.08	167.00	169.62	170.94
Eastbound				
Distance (mi)[a]	0.00	2.92	5.54	6.86
Number of observations		142	109	
Standard deviation (mph)		4.4	4.3	
Range (mph)		20	22	
Percentage in 10-mph pace		73	82	
Skewness		–0.14	0.00	
Accident involvement rate (accidents/100 MVM)		140	140	
Westbound				
Distance (mi)[a]	6.86	3.94	1.32	0.00
Number of observations	155	164	200	
Standard deviation (mph)	4.8	3.6	4.0	
Range (mph)	28	24	21	
Percentage in 10-mph pace	80	85	82	
Skewness	0.00	0.00	–0.20	
Accident involvement rate (accidents/100 MVM)	150	140	140	

[a]Distance from beginning of TLTW operations.

TABLE 6 SPEED DISTRIBUTION PARAMETERS FOR PROJECT IR-80-4(82), BRADY EAST

	Mile Post			
	198.40	201.00	203.10	205.62
Eastbound				
Distance (mi)[a]	0.00	2.60	4.70	7.22
Number of observations		155	153	
Standard deviation (mph)		5.3	5.3	
Range (mph)		25	24	
Percentage in 10-mph pace		62	65	
Skewness		+0.25	0.00	
Accident involvement rate (accidents/100 MVM)		150	130	
Westbound				
Distance (mi)[a]	7.22	4.62	2.52	0.00
Number of observations	159	154	156	
Standard deviation (mph)	4.7	5.3	4.5	
Range (mph)	22	24	26	
Percentage in 10-mph pace	70	71	76	
Skewness	0.00	−0.14	0.00	
Accident involvement rate (accidents/100 MVM)	140	140	140	

[a]Distance from beginning of TLTW operations.

TABLE 7 SPEED DISTRIBUTION PARAMETERS FOR PROJECT IR-80-5(44), ELM CREEK–ODESSA WESTBOUND

	Mile Post			
	256.64	259.08	261.93	263.32
Eastbound				
Distance (mi)[a]	0.00	2.44	5.29	6.68
Number of observations		169	153	161
Standard deviation (mph)		4.2	3.9	3.9
Range (mph)		21	23	19
Percentage in 10-mph pace		76	84	81
Skewness		0.00	+0.20	0.00
Accident involvement rate (accidents/100 MVM)		140	140	140
Westbound				
Distance (mi)[a]	6.68	4.24	1.39	0.00
Number of observations		146	157	
Standard deviation (mph)		4.2	4.6	
Range (mph)		22	25	
Percentage in 10-mph pace		80	75	
Skewness		0.00	−0.33	
Accident involvement rate (accidents/100 MVM)		140	140	

[a]Distance from beginning of TLTW operations.

shoulder to the opposing lane was struck by an oncoming vehicle in the near lane. The vehicle making the U-turn was a construction vehicle that was turning around so that its occupants could replace one of the 18-in. tubular posts that was missing from the centerline of the TLTW segment.

The accident rates computed for the TLTW segments are shown in Table 9. The segment lengths, average daily traffic rates (ADTs), days of operation, and numbers of accidents that were used to compute these rates are also included in this table. Two of the four segments did not experience any accidents, therefore they had zero accident rates. The accident rates on the other two segments, which did experience accidents, were 11.4 and 28.6 accidents/100 MVM.

The analysis of variance for the simple linear regression analysis of these TLTW accident rates is presented in Table 10, together with that from the previous study (*2*), which is also presented in Table 2. This analysis shows that there was no statistically significant linear relationship between the accident rates on the TLTW segments and the lengths of the segments. Therefore the result of this analysis indicates that the safety of TLTW operations was not related to the length of the TLTW segment for the conditions studied.

CONCLUSION

In this study, no relationships were found between TLTW segment length and either the accident rate or any of the speed distribution parameters that were used as indicators of the safety of TLTW operations. Therefore it was concluded that there is no relationship between TLTW segment length and the safety of TLTW operations for the conditions studied. It was also concluded that the longer optimum segment lengths computed with Equation 1 using the 1986 cost factors are applicable from the standpoint of safety.

Of course, it must be noted that the maximum TLTW segment length in this study was 7.22 mi. Consequently, the findings and conclusion of this study are limited to TLTW segments no longer than 7.22 mi. Similar studies of longer TLTW segments are needed to determine the safety effects of segment lengths longer than 7.22 mi. Also, it should be noted that the TLTW segments in this study were on level terrain and that they had the paving markings shown in Figure 1, which featured a 3-ft median composed of raised pavement markers and 18-in. tubular posts. The findings and conclusion of this study may not be applicable to TLTW segments in rolling or mountainous terrain or with other types of centerline treatments.

TABLE 8 RESULTS OF ANALYSIS OF VARIANCE FOR SPEED DISTRIBUTION PARAMETER REGRESSIONS

Parameter	Degrees of Freedom	F	Prob > F	Conclusion[a]
Standard deviation	1,18	0.047	0.832	$\beta_1 = 0$
Range	1,18	0.038	0.847	$\beta_1 = 0$
Percentage in 10-mph pace	1,18	1.455	0.243	$\beta_1 = 0$
Skewness	1,18	1.757	0.302	$\beta_1 = 0$
Accident involvement rate	1,18	0.102	0.753	$\beta_1 = 0$

[a]On the basis of 0.05 level of significance.

TABLE 9 TLTW SEGMENT ACCIDENT RATES

Project	Length (mi)	ADT[a]	Days of Operation	Number of Accidents	Accident Rate (accidents/100 MVM)
IR-80-3(81), Sutherland West	6.78	9,220	93	0	0
IR-80-3(88), Hershey East	6.86	13,075	78	2	28.6
IR-80-4(82), Brady East	7.22	11,510	106	1	11.4
Elm Creek–Odessa westbound	6.68	11,425	54	0	0

[a]During TLTW operation.

TABLE 10 ANALYSIS OF VARIANCE FOR REGRESSION ANALYSIS OF ACCIDENT RATES ON ALL TLTW SEGMENTS

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F	Prob > F
Due to b_0	1	21,734.14	21,734.14		
Due to b_1/b_0	1	44.22	44.22	0.025	0.875
Residual	26	45,242.15	1,740.08		
Total	28	67,020.51			

NOTE: $\hat{Y} = b_0 + b_1X$, where $\hat{Y}$ is the TLTW segment accident rate (accidents/100 MVM), b_0 is the y intercept, b_1 is the slope, and X is the segment length (mi).

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Evaluation of I-75 Lane Closures

JERRY G. PIGMAN AND KENNETH R. AGENT

A spot pavement replacement and joint sealing project on I-75 in southern Kentucky during the 1986 construction season involved numerous lane closures. Traffic congestion caused by heavy volumes and late merges resulted in the use of the following traffic control devices to supplement standard lane closure devices: variable message signs, supplemental lane closure warning signs, and rumble strips placed in the lane to be closed in advance of the taper. Because the devices were not typical applications for work zones and because of the potential for applications at other sites, an evaluation study was conducted. Results showed a decrease in the percentage of traffic in the lane to be closed with each successive traffic control device in addition to the standard lane closure devices. There was a general decrease in speeds as traffic approached the taper. The percentage of trucks in the lane to be closed was lower than the percentage in the open lane when the closure was a left lane. Hourly traffic volumes observed in this study (800 to 1,300 vph) did not appear to influence the percentage of traffic in the lane to be closed. The percentage of trucks in both lanes (8.5 to 14.7 percent) did not influence the percentage of traffic in the lane to be closed either. Recommendations from the study included the following: (a) supplemental signs for all long-term closures on high-volume, high-speed four-lane roadways, (b) variable message signs when one-way hourly volumes exceed 1,000 (ADT exceeds 20,000), and (c) application of rumble strips if other devices do not reduce late merges and there is excessive congestion.

Construction and maintenance work zones have traditionally been hazardous locations within the highway environment. Safety in work zones has been recognized as a significant problem for several years, and the subject has received additional attention with the shift from construction of new facilities to the improvement or rehabilitation of existing facilities. With recent increases in the volume of traffic and changes in the compositions of the traffic streams, however, congestion on some highway sections has increased, and there is a greater potential for accidents. Several studies have shown that accident rates within construction or maintenance work zones are higher than for similar periods before work zones were established (*1–3*). Among the many factors cited as reasons for the increase in accident rates are inappropriate use of traffic control devices, poor traffic management, inadequate layout of the overall work zone, and a general misunderstanding of the unique problems associated with construction or maintenance work zones.

The closure of a lane on a four-lane high-speed facility during construction or maintenance activity presents potential safety problems. Lane closure problems are related to changes in the driving environment that require the driver to make adjustments in order to travel through a work zone safety. On high-volume four-lane facilities, problems occur when two lanes of traffic must be warned sufficiently in advance so that motorists may travel safety through the transition zone of merging two lanes into one lane at the work site. Frequently, there are vehicles that fail to merge to the open lane, at situation that leads to congestion and erratic maneuvers at the beginning of the taper.

The *Manual on Uniform Traffic Control Devices* (*4*) provides details on standard applications for lane closures, and those applications appear to be adequate for most situations. However, as volumes increase and geometric conditions place additional constraints on the flow of traffic at a lane closure, consideration should be given to additional traffic control devices. The effectiveness of variable message signs has been evaluated previously, and the results were increased advance lane change activity, smoother lane change profiles, and significantly fewer lane changes near the taper (*5*). As a result of that study, the following suggestions for additional research were made: use of arrows in barricade design, multiple variable message signs, audible signals such as rumble strips, and combined use of symbols and words on variable message signs (*5*).

A project on I-75 in Whitley and Laurel counties, Kentucky, during the 1986 construction season involved numerous lane closures to accommodate spot pavement replacement and joint sealing. Traffic congestion caused by heavy volumes and failure to adhere to the traffic control messages resulted in a decision by the Kentucky Department of Highways to use additional traffic control devices to encourage proper merging for a smoother flow of traffic through the lane closures. Additional traffic control devices used in this case included supplemental signs, variable message signs, and rumble strips. Because these additional devices were not typical applications for work zones, it was decided that their effectiveness should be evaluated.

DATA COLLECTION

Data were collected at lane closures on I-75 in Whitley and Laurel counties between June 6, 1986, and August 8, 1986. Data collection periods of 6 hr each on five Friday afternoons and four Sunday afternoons were included. Table 1 is a summary of data collection dates, locations, and traffic control conditions for each of the data collection periods. All data were collected from 11:00 a.m. to 5:00 p.m. on Fridays and from 12:00 noon to 6:00 p.m. on Sundays.

Because the objective of this study was to determine whether supplemental traffic control devices could be used at work zones to improve the flow of traffic through lane closures, it was necessary to add devices to the standard control devices by

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TABLE 1 DATA COLLECTION DATES, LOCATIONS, AND TRAFFIC CONTROL CONDITIONS

Date	Lane Closure Location	Traffic Control Conditions
6/06/86	I-75 SB, MP 42.2	Standard left-lane closure traffic control devices
6/08/86	I-75 NB, MP 27.2	Standard right-lane closure traffic control devices
6/13/86	I-75 SB, MP 42.2	Standard left-lane closure traffic control devices and variable message sign placed 1.8 mi before lane closure
6/15/86	I-75 NB, MP 27.2	Standard right-lane closure traffic control devices and variable message sign placed 1.25 mi before lane closure
6/20/86	I-75 SB, MP 42.2	Standard left-lane closure traffic control devices, variable message sign placed 2 mi before lane closure, and supplemental construction zone signs placed 5, 4, 3, and 2 mi before lane closure
6/22/86	I-75 NB, MP 30.1	Standard left-lane closure traffic control devices, variable message sign placed 0.9 mi before lane closure, and supplemental construction zone signs placed 5, 4, 3, and 2 mi before lane closure
7/11/86	I-75 SB, MP 46.4	Standard left-lane closure traffic control devices, variable message sign placed 2 mi before lane closure, supplemental construction zone signs placed 5, 4, 3, and 2 mi in advance of lane closure, and rumble strips placed 1.5, 1.0, 0.6, 0.3, and 0.1 mi before lane closure
7/27/86	I-75 NB, MP 17.9	Standard left-lane closure traffic control devices, variable message sign placed 1.9 mi before lane closure, and supplemental construction zone signs placed 5, 4, 3, and 2 mi before lane closure
8/08/86	I-75 NB, MP 14.2	Standard left-lane closure traffic control devices, variable message sign placed 1.9 mi before lane closure, supplemental construction zone signs placed 5, 4, 3, and 2 mi in advance of lane closure, and rumble strips placed 1.5, 1.0, 0.6, 0.3, and 0.1 mi before the lane closure

NOTE: NB = northbound, SB = southbound, MP = milepoint.

an incremental process. This required selection of sites at which the lane closure would exist for a long enough time to permit addition of the supplemental devices and data collection before the closure had to be moved. Obvious constraints on these requirements were construction schedules and holiday periods. It was undesirable to extend the time of lane closures from the standpoint of prolonged congestion and increased accident potential. Therefore some variability in the data was expected because of the inability to evaluate all increments of supplemental traffic control at the same location. Geometric constraints included vertical curves, horizontal curves, and interchange ramps.

As presented in Table 1, data collection included four days for southbound traffic and five days for northbound traffic. For southbound traffic, one site (I-75 at milepoint 42.2) was used for the first three lane closure traffic control conditions, and another site (I-75 at milepoint 46.4) was used for the fourth lane closure condition. The first lane closure condition consisted of the standard left-lane closure traffic control devices, as shown in Figure 1. To evaluate the effect of lane closure advance warning devices, it was necessary to station observers at four positions in advance of the closed lane. For the standard lane closure control condition, observers were positioned at the following points with respect to data collection needs:

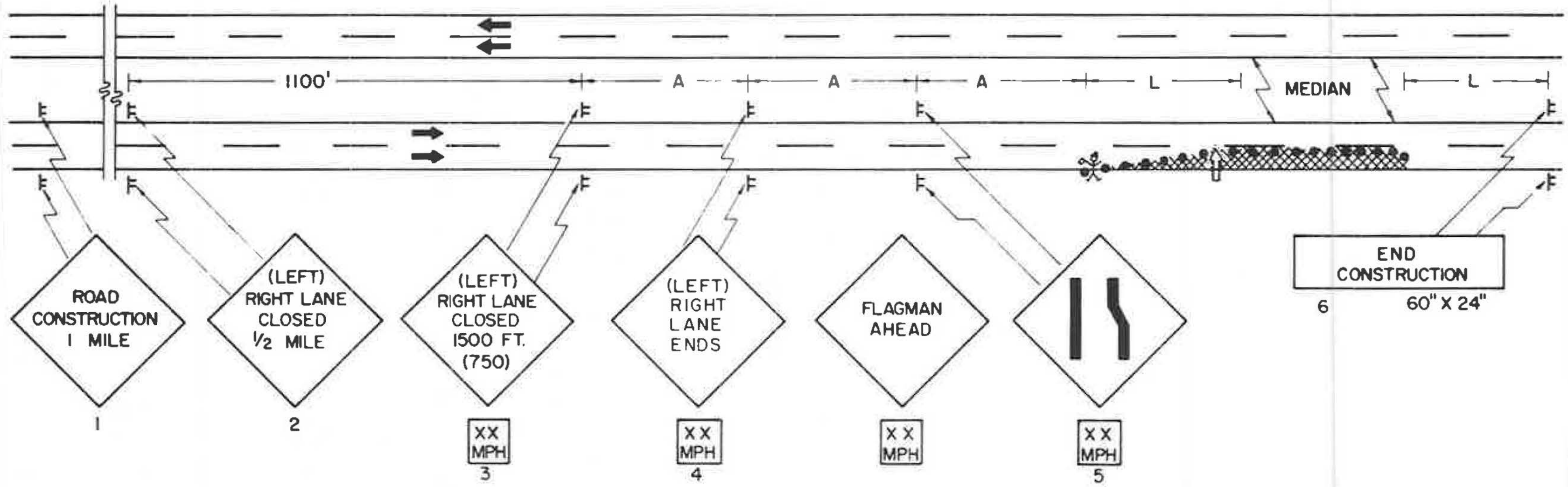
- Before construction zone signs, where free-flowing traffic could be observed;
- At the point where the variable message sign was to be placed;
- Between the variable message sign position and the beginning of the taper; and
- At the beginning of the taper.

Several observation points were necessary to monitor the effect of various traffic control conditions on lane distributions and speeds. Data also were collected to represent total volumes and percent trucks.

Data were collected on June 6, 1986, to document lane distribution and speed conditions for standard lane closure traffic control devices. On the following Friday (June 13, 1986), data were collected at the same observation points with the variable message sign (message was Merge Right, with an arrow progressively moving to the right) placed 1.8 mi before the lane closure. Sight distance requirements made it necessary to place the variable message sign either 1.8 mi before or very near to the beginning of the taper and the standard arrow board. The third data collection date was June 20, 1986, and this same lane closure at milepoint 42.2 on I-75 was modified by adding signs 5, 4, 3, and 2 mi before the closure indicating that the left lane was closed. These supplementary signs were in addition to standard lane closure devices and the variable message sign 1.8 mi before the taper. The fourth traffic control condition was the addition of sets of rumble strips 1.5, 1.0, 0.6, 0.3, and 0.1 mi before the beginning of the taper. Because the lane closure had been moved before the fourth day of data collection, it was necessary to delay additional data collection in the southbound direction until July 11, 1986. The site for evaluation of the rumble strips was located at milepoint 46.4 on I-75, and there were geometric constraints in the form of both vertical and horizontal curvature that may have influenced the lane distribution and speed data.

Rumble strips used before the lane closure consisted of eight strips per set, placed with 24 in. between strips. As noted previously, the strips were installed 1.5, 1.0, 0.6, 0.3, and 0.1 mi before the taper in the lane to be closed. The strips were made of a hard plastic-vinyl material, with dimensions of 1/2 in. × 4 in. × 23 3/4 in. Each set required 48 strips, or 240 strips for five sets. The installation process included the following steps:

- Preparation of the surface by brushing,
- Application of solvent cement to the back of the strip,
- Placement of the strip on the pavement,
- Application of pressure to the strip so that a coating of cement was deposited on the pavement,
- Removed of the strip from the pavement for approximately 30 sec so that the cement was exposed to the air to dry, and



APPLICATION

This drawing applies to lane closure of shoulder or median lanes on two direction multi-lane highways with medians over six (6) feet in width.

SIGNING AND SPACING TABLE

NORMAL POSTED SPEED LIMIT	"L" FT.	SIGNS REQUIRED	SIGN SIZE (1 THRU 5)	SPEED ADVISORY	ADVISORY SPEED PLATE SIZE	"A" FT.
55 and above	900	1 thru 6	48"X 48"	45 ①	24"X 24"	500
45 to 50	600	3thru 6	48"X 48"	35 ①	24"X 24"	500
35 to 40	440	3thru 6	48"X 48"	25 ①	24"X 24"	250
Less than 35	360	3thru 6	48"X 48"or 36"X 36"	None Req.①	24"X 24"	250

① Or as directed by the Engineer

LEGEND

- Portable Flashing Arrow
- Flagman
- Sign
- L Length of Transition
- ● CHANNELIZATION DEVICES
  - Cones
  - Drums
  - Type II Barricades
  - Tubular Markers

Traffic Cones, Tubular Markers, Drums or Type II Barricades shall be maintained throughout the entire length of the immediate construction area. Spacing of Channelization Devices shall not exceed forty(40) ft. throughout the work site and the transition zone. (Note: Skip lines on pavement are normally forty (40) feet from begining of line to begining of line.)

The Engineer may require the use of Drums or Type II Barricades in lieu of Cones or Tubular Markers if the closure time exceeds four (4) days.

The signs shall be moved behind the ditch line, and made inaccessible to the view of traffic or covered, at any time the lane is not physically closed.

The portable flashing arrow shall be required when the normal posted speed limit is greater than 45 MPH. At speeds less than 45 MPH, a flagman may be used in lieu of the portable flashing arrow as specified in the plans or as directed by the Engineer. When a flagman is used, the "Flagman Ahead" sign shall be substituted for the "(Left) Right Lane Ends Sign." All traffic control devices required on this drawing shall conform to the requirements shown on the current edition of Standard Drawing No. TSC 260, TSC 270, and to the Department's "Manual On Uniform Traffic Control Devices". Use "750 Ft." on sign No. 3 when the normal posted speed limit is 40 miles per hour or less.

Sign spacing may be modified or adjusted slightly to fit the physical conditions encountered such as driveways, approach roads, etc. Such modifications shall have the approval of the Engineer.

When approach roads and intersecting streets are encountered, some additional signing may be required on these roads and streets which is not shown on this drawing. Such signing shall conform to the requirements of the Department's "Manual On Uniform Traffic Control Devices" as directed by the Engineer.

KENTUCKY
DEPARTMENT OF HIGHWAYS

LANE CLOSURE
CASE III

STANDARD DRAWING No. TSC 210-03

SUBMITTED ____ DATE ____
APPROVED ____ STATE HIGHWAY ENGINEER ____ DATE

FIGURE 1 Standard lane closure traffic control devices.

• Application of pressure to the strip again to bond it to the pavement.

The rumble strips used in the installation were purchased from Astro Optics, and the solvent cement (Type SC-1958) was produced by the H. B. Fuller Company.

After installation, traffic was allowed to pass over the strips after about 2 hr, even though the solvent cement remained soft and flexible. The solvent cement proved to be a very good adhesive for application of the rumble strips. The cement was relatively easy to apply with caulking guns. It remained somewhat flexible for several hours yet sufficiently bonded the strips to the pavement, and the strips were relatively easy to remove. After removal of the strips, the cement remained on the pavement, but it was thin enough so that there was no noticeable noise when vehicles passed over it. At the northbound installation of rumble strips, the number missing after 9 days was 12 of the 240 (5 percent). This loss of a few strips did not appear to diminish the effectiveness of the installation.

The first data collection at a northbound lane closure was on June 8, 1986, at milepoint 27.2. Traffic control at this location was a standard right lane closure. That same location was used again on June 15, 1986, with the addition of the variable message sign 1.25 mi before the closure. However, on the third Sunday of data collection, it was necessary to move to a new site at milepoint 30.1 because the closure at milepoint 27.2 had been removed. This resulted in data collection at a different location with supplemental signs at 5, 4, 3, and 2 mi before the closure. The difficulty of evaluating those signs was complicated because the observation points had to be located very near an interchange ramp.

Because roadway geometrics had complicated the evaluation process for determining the potential impact of adding rumble strips to the three previous control conditions, it was decided that data should be collected at a site with and without rumble strips. That required two additional data collection periods with all other traffic control conditions in place on the first date and rumble strips added to the existing control devices on the second date. The final two data collection dates were July 27 and August 8, 1986. Again because of unanticipated construction scheduling problems, data could not be collected at the same site for two consecutive weeks at a northbound site. However, there did not appear to be major geometric differences that would prevent a comparison of those two lane closures with and without rumble strips.

RESULTS

The primary measure of effectiveness for evaluating the various traffic control alternatives was percentage of traffic remaining in the lane to be closed. As noted previously, data were collected at the following points before the lane closure:

• Before the construction zone signs,
• At the variable message sign or where it was to be placed,
• Between the variable message sign and the beginning of the taper, and
• At the beginning of the taper.

Other data collected included average speeds at the observation points in advance of the construction zone and at the observation point between the variable message sign and the beginning of the taper. Percent trucks and average hourly traffic volumes were also tabulated for each of the observation points. Summaries of the various types of data collected at southbound and northbound sites are presented in Tables 2 and 3, respectively.

TABLE 2 SUMMARY OF DATA COLLECTED AT SOUTHBOUND LANE CLOSURES

Date	Location	Traffic Control Conditions	Data Collection Point	Distance From Taper (mi)	Average Speed (mph)	Percent of Traffic in Lane To Be Closed	Percent Trucks in Lane To Be Closed	Average Percent Trucks (both lanes)	Average Hourly Traffic (both lanes)
6/06/86	I-75 SB, MP 42.2	Standard left-lane closure	Free-flowing	3.6	60.1	35.8	4.9	12.1	913
			Free-flowing	1.8		29.0	5.8	12.8	808
			Intermediate	0.9	58.2	35.7	7.9	12.2	953
			500 ft before taper	0.1		14.9	9.6	11.3	967
			At taper	0		3.7	9.2	11.3	967
6/13/86	I-75 SB, MP 42.2	Standard left-lane closure and variable message sign	Free-flowing	3.6	60.8	50.8	4.9	11.7	1,042
			Free-flowing	1.8		20.3	5.2	11.2	1,096
			Intermediate	0.9	60.3	23.6	7.4	11.2	1,018
			500 ft before taper	0.1		11.6	17.3	11.9	1,068
			At taper	0		3.2	9.0	11.9	1,068
6/20/86	I-75 SB, MP 42.2	Standard left-lane closure, variable message sign, and supplemental signs	Free-flowing	3.6	62.4	37.0	4.1	10.6	1,095
			Free-flowing	1.8		17.7	5.8	10.6	1,104
			Intermediate	0.9	61.1	21.7	8.0	10.5	1,076
			500 ft before taper	0.1		10.4	5.3	9.6	1,096
			At taper	0		3.0	1.5	9.6	1,096
7/11/86	I-75 SB, MP 46.4	Standard left-lane closure, variable message sign, supplemental signs, and rumble strips	Free-flowing	8.1	62.6	37.7	6.2	11.2	1,082
			Free-flowing	2.1		24.0	7.2	10.9	1,075
			Intermediate (at rumble strips)	1.25	57.2	26.2	7.8	11.6	1,030
				0.8	55.5	22.8	3.9	10.6	1,013
				0.45	48.4	24.9	2.5	11.2	952
				0.2	51.6	11.4	5.3	10.4	1,114
			500 ft before taper	0.1		7.8	3.6	9.2	1,063
			At taper	0		2.1	3.0	9.2	1,063

NOTE: SB = southbound, MP = milepoint.

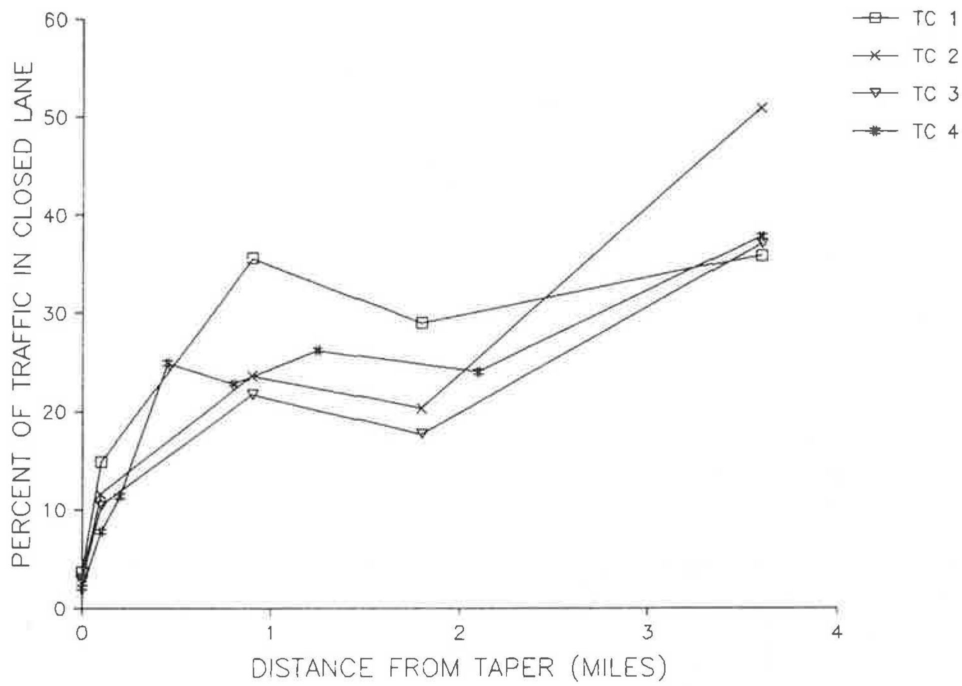
TABLE 3 SUMMARY OF DATA COLLECTED AT NORTHBOUND LANE CLOSURES

Date	Location	Traffic Control Conditions	Data Collection Point	Distance From Taper (mi)	Average Speed (mph)	Percent of Traffic in Lane to be Closed	Percent Trucks in Lane to be Closed	Average Percent Trucks (both lanes)	Average Hourly Traffic (both lanes)
6/06/86	I-75 NB, MP 27.2	Standard right-lane closure	Free-flowing	1.8	64.8	59.8	14.5	11.7	1,005
			Free-flowing	1.25		61.9	12.4	10.2	1,083
			Intermediate	0.5	61.1	59.1	12.9	12.0	1,075
			500 ft before taper	0.1		21.9	9.1	11.3	1,047
			At taper	0		6.7	5.9	11.3	1,047
6/15/86	I-75 NB, MP 27.2	Standard right-lane closure and variable message sign	Free-flowing	1.8	61.0	55.8	12.2	9.4	1,133
			Free-flowing	1.25		19.3	14.6	9.6	1,085
			Intermediate	0.5	54.2	19.3	15.6	9.6	1,139
			500 ft before taper	0.1		10.9	13.1	8.9	1,117
			At taper	0		6.9	12.4	8.9	1,117
6/22/86	I-75 NB, MP 30.1	Standard left-lane closure, variable message sign, and supplemental signs	Free-flowing	3.5	61.9	35.6	8.7	9.8	1,253
			Free-flowing	0.9		20.3	7.8	10.2	1,224
			Intermediate	0.5	54.0	9.9	10.7	10.2	1,299
			500 ft before taper	0.1		5.6	12.9	8.5	1,273
			At taper	0		3.3	11.6	8.5	1,273
7/27/86	I-75 NB, MP 17.9	Standard left-lane closure, variable message sign, supplemental signs	Free-flowing	5.9	57.6	38.3	4.3	10.0	1,018
			Free-flowing	1.9		33.0	3.6	9.5	1,059
			Intermediate	1.1	57.1	25.6	6.0	9.7	1,064
			500 ft before taper	0.1		11.0	4.3	8.7	1,070
			At taper	0		3.0	6.3	8.7	1,070
8/08/86	I-75 NB, MP 14.2	Standard left-lane closure, variable message sign, supplemental signs, and rumble strips	Free-flowing	5.8	63.7	33.6	8.2	12.7	882
			Free-flowing	2.2		30.3	5.3	11.8	1,015
			Intermediate (at rumble strips)	1.4	58.6	22.3	9.2	14.7	975
				0.8	57.4	23.2	8.2	13.8	1,006
				0.4	61.0	18.9	7.6	11.1	995
				0.2	57.6	8.9	11.4	13.2	889
			500 ft before taper	0.1		4.1	9.0	10.9	950
			At taper	0		0.1	6.4	10.9	950

NOTE: NB = northbound, MP = milepoint.

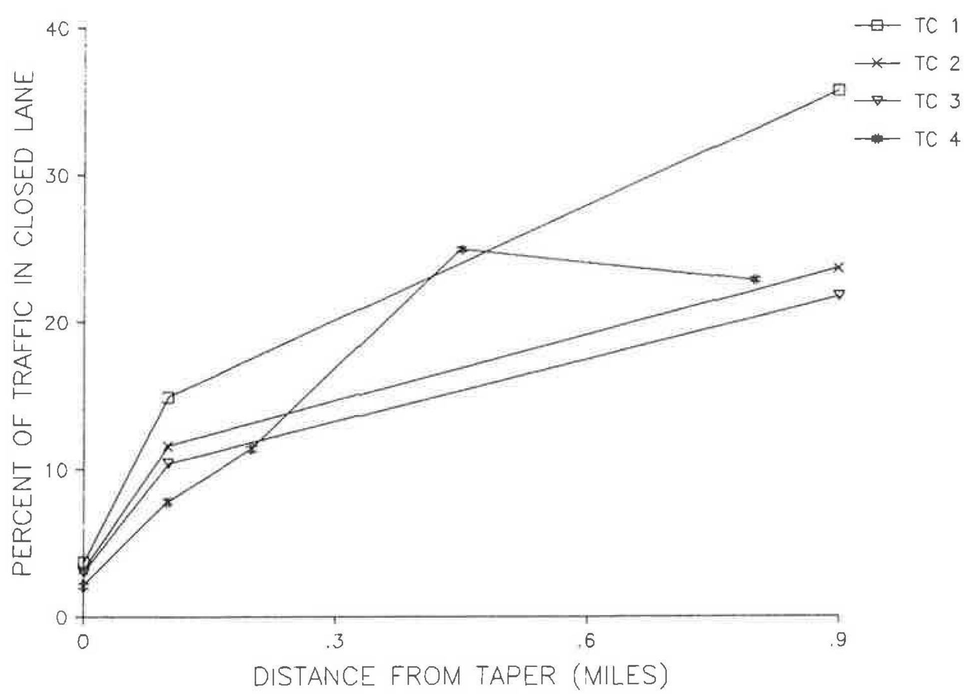
For southbound sites (Table 2), the data generally indicated a decreasing percentage of traffic in the lane to be closed as the distance to the taper decreased. When the various traffic control conditions were compared, a decrease was also seen in the percentage of traffic in the closed lane with the addition of traffic control devices beyond the standard lane closure devices. The data that show the relationship between percent of traffic in the lane to be closed and the distance from the taper are presented graphically in Figure 2. The general trend over approximately 3.5 mi before the taper indicated the effectiveness of various traffic control devices. Specifically, it may be noted that the addition of a variable message sign (Merge Right or Left with arrow) has a positive effect on decreasing the percentage of traffic in the lane to be closed. For example, the percentage of traffic in the lane to be closed decreased from 14.9 percent to 11.6 percent at 0.1 mi before the taper. If the data in Table 2 are examined further, it may be seen that the addition of supplemental advance warning signs reduced the percentage of traffic in the closed lane to 10.4 percent at 0.1 mi in advance. The effect of adding a variable message sign and then supplemental construction zone warning signs to the standard lane closure signs could be evaluated without questioning the results because data were collected at the same lane closure site. The addition of rumble strips to the standard lane closure signs, variable message sign, and supplemental signs, however, was complicated because data had to be collected at a new lane closure site. The site at which rumble strips were installed included both horizontal and vertical curvatures, which may have resulted in the greater probability of a higher percentage of vehicles being in the closed lane. The results presented in Table 2 show 7.8 percent of the traffic in the closed lane (at 0.1 mi before the taper) with rumble strips added as compared to 10.4 percent without rumble strips but with the other three traffic control conditions. To better show the effect of various traffic control conditions within 1 mi of the taper, Figure 3 was prepared. That figure allows comparisons to be made within 0.9 mi of the beginning of the taper.

Similar data summaries were prepared for northbound lane closure data, and the results included in Table 3 indicate a pattern similar to the data presented for southbound lane closures. Again, there were factors that complicated evaluations of the differences in traffic control devices. The first 2 days of data collection for northbound traffic involved right-lane closure at the same location. A standard right-lane closure was in operation on the first day, and 21.9 percent of the traffic was in the lane to be closed 0.1 mi in advance of the taper, compared to 10.9 percent in the lane to be closed with a variable message sign added 1.25 mi before the taper. This clearly shows the effectiveness of the variable message sign as a device to promote earlier merging and a smoother flow of traffic through the lane closure. The third traffic control condition of supplemental signs added to the standard left-lane closure and variable message sign was at milepoint 30.1 on I-75 northbound. This was a left-lane closure rather than a right-lane closure, as used for the first and second data collection sites northbound, and the results indicate a much lower



TC 1 - Standard left lane closure traffic control devices
TC 2 - Standard devices with variable message sign
TC 3 - Standard devices, variable message, and supplemental signs
TC 4 - Standard devices, variable message, supplemental signs and rumble strips

FIGURE 2 Distribution of traffic from 3.5 mi in advance to beginning of taper (southbound lane closures).



TC 1 - Standard left lane closure traffic control devices
TC 2 - Standard devices with variable message sign
TC 3 - Standard devices, variable message, and supplemental signs
TC 4 - Standard devices, variable message, supplemental signs and rumble strips

FIGURE 3 Distribution of traffic from 0.9 mi in advance to beginning of taper (southbound lane closures).

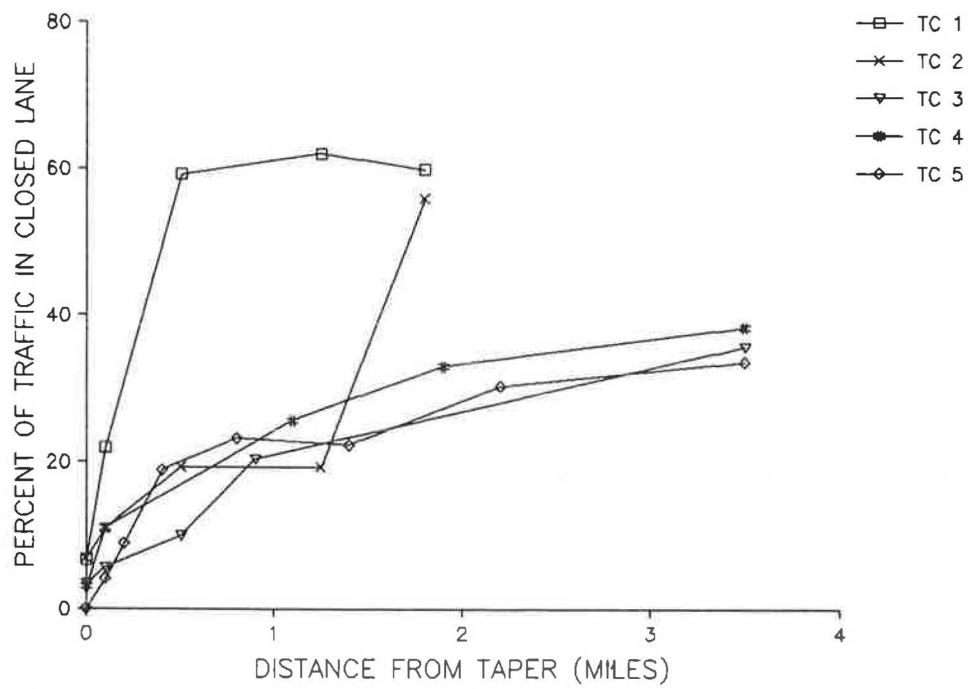
percentage (5.6 percent) of traffic in the lane to be closed 0.1 mi in advance of the taper.

In an effort to determine the impact of rumble strips used in addition to the other traffic control devices, another site was selected at a northbound closure (milepoint 17.9), where data would be collected with all devices except rumble strips and then at that same location with the addition of rumble strips. However, there was an unanticipated change in the construction schedule, and the left-lane closure was not in place for two consecutive weeks. Instead, data were collected at milepoint 17.9 without rumble strips and at milepoint 14.2 with rumble strips added to the other types of traffic control. The results, presented in Table 3, show that the percentage of traffic in the lane to be closed at 0.1 mi in advance of taper decreased from 11.0 percent with all traffic control devices in place except rumble strips to 4.1 percent with rumble strips added at distances in advance of the taper of 1.5, 1.0, 0.6, 0.3, and 0.1 mi. Even with the change in locations for evaluation of rumble strips, there were relatively minor differences in geometrics that may have affected the results. It appears the rumble strips were effective in decreasing the percentage of traffic in the lane to be closed 0.1 mi in advance and at the beginning of the taper. The relationship between percentage of traffic in the lane to be closed and distance from the taper is presented for northbound lane closures in Figure 4. The effects of various traffic control measures within 1 mi of the taper are presented in Figure 5. Data presented in Figure 5 allow a more detailed comparison of percent traffic in the lane to be closed at 1.0 mi, 0.1 mi, and at the taper.

Additional data documenting speeds, percent trucks, and average hourly traffic are presented in Tables 2 and 3. Speed data were collected before the construction zone signs and at a point between the variable message sign and the beginning of the taper. Results indicate a general decrease in speeds as traffic approached the taper; however, speeds still averaged more than 55 mph in the range of 1 mi to 1/2 mi before the taper.

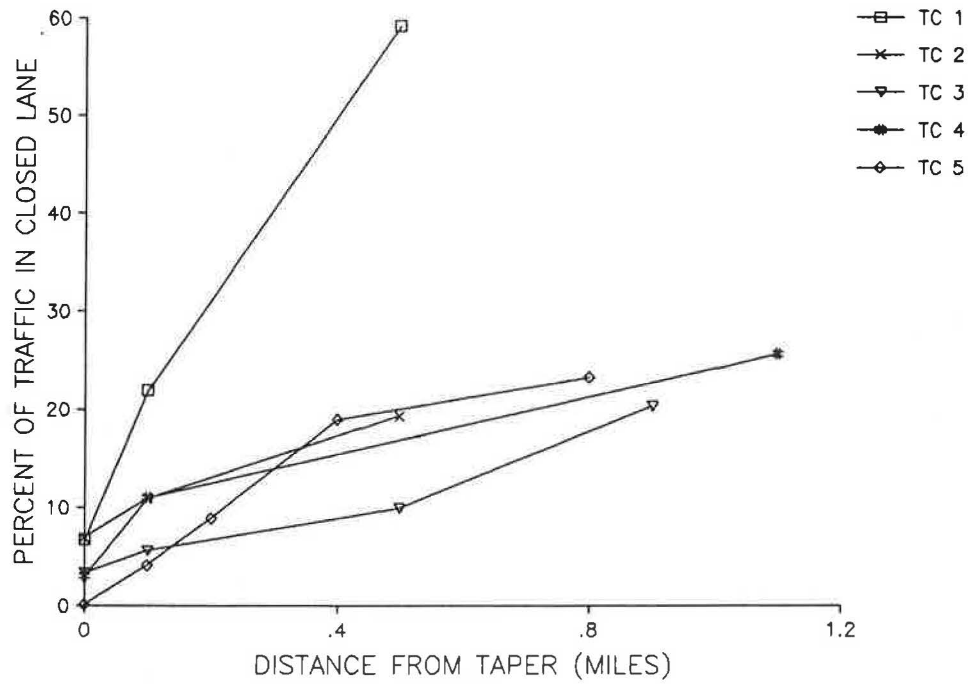
The percentage of trucks was determined for all data collection points, and the results are presented as percentage of trucks in both lanes and the percentage of trucks in the lane to be closed. When the percentage of trucks was averaged for both lanes, it ranged from 8.5 to 14.7 percent. There were generally more trucks on Fridays than on Sundays. Another measure of compliance with the traffic control devices was the percentage of trucks in the lane to be closed. For almost all data collection points, the percentage of trucks in the lane to be closed was lower than the percentage in the open lane when the closure was a left lane. For a right-lane closure, there were more occurrences of a higher percentage of trucks in the lane to be closed than in the open lane. This was obviously affected by the higher percentage of trucks that typically travel in the right lane on four-lane roadways.

Average hourly traffic data, as summarized in Tables 2 and 3, show a range from approximately 800 to 1,300. As stated previously, data were collected on Fridays and Sundays, and the highest volumes were generally on Sundays. For the 6-hr data collection period, the lowest average volumes were on Friday, June 6, and the highest volumes were on Sunday, June 22.



TC 1 - Standard lane closure traffic control devices
TC 2 - Standard devices with variable message sign
TC 3 - Standard devices, variable message, and supplemental signs
TC 4 - Standard devices, variable message, and supplemental signs
TC 5 - Standard devices, variable message, supplemental signs, and rumble strips

FIGURE 4 Distribution of traffic from 3.5 mi in advance to beginning of taper (northbound lane closures).



TC 1 - Standard lane closure traffic control devices
TC 2 - Standard devices with variable message sign
TC 3 - Standard devices, variable message, and supplemental signs
TC 4 - Standard devices, variable message, and supplemental signs
TC 5 - Standard devices, variable message, supplemental signs, and rumble strips

FIGURE 5 Distribution of traffic from 1.1 mi in advance to beginning of taper (northbound lane closures).

Because the average hourly volume was expected to have an impact on the percent of traffic in the lane to be closed, the relationships between these variables 0.1 mi before the taper and at the taper were investigated. The general perception had been that there was a higher percentage of late merges and interrupted traffic flow at the taper with increasing volumes. The data presented in Table 4 indicate relatively little change in volume for the southbound lane closures; it was therefore assumed that decreasing percentages of traffic in the lane to be closed were related to the effectiveness of traffic control devices. For northbound lane closures, the volumes were generally higher but did not appear to influence the percentage of traffic in the lane to be closed.

The interrelationship between percent trucks, hourly traffic volumes, and percent traffic in the lane to be closed was also analyzed (Table 4). It does not appear that a higher percentage of trucks resulted in a higher percentage of traffic in the lane to be closed. For example, at northbound lane closures on July 27 and August 8, there was a decrease in the percent traffic in the lane to be closed (11.0 to 4.1 percent) even though the percent trucks increased from 8.7 to 10.9 percent. It should be noted that average hourly traffic volumes decreased from 1,070 to 950, which also may have contributed to the reduced traffic in the closed lane. In addition, there does not appear to be a relationship between percent trucks in both lanes and the percent trucks in the lane to be closed.

TABLE 4 RELATIONSHIPS BETWEEN HOURLY VOLUMES, PERCENT TRUCKS, AND PERCENT TRAFFIC IN LANE TO BE CLOSED

		Percent Traffic in Lane To Be Closed		Percent Trucks in Lane To Be Closed			
Date	Location	0.1 mi Before Taper	At Taper	0.1 mi Before Taper	At Taper	Percent Trucks (both lanes)	Average Hourly Traffic
6/06/86	I-75 SB, MP 42.2	14.9	3.7	9.6	9.2	11.3	967
6/13/86	I-75 SB, MP 42.2	11.6	3.2	17.3	9.0	11.9	1,068
6/20/86	I-75 SB, MP 42.2	10.4	3.0	5.3	1.5	9.6	1,096
7/11/86	I-75 SB, MP 46.4	7.8	2.1	3.6	3.0	9.2	1,063
6/08/86	I-75 NB, MP 27.2	21.9	6.7	9.1	5.9	11.3	1,047
6/15/86	I-75 NB, MP 27.2	10.9	6.9	13.1	12.4	8.9	1,117
6/22/86	I-75 NB, MP 30.1	5.6	3.3	12.9	11.6	8.5	1,273
7/27/86	I-75 NB, MP 17.9	11.0	3.0	4.3	6.3	8.7	1,070
8/08/86	I-75 NB, MP 14.2	4.1	0.8	9.0	6.4	10.9	950

NOTE: NB = northbound, SB = southbound, MP = milepoint.

SUMMARY

As mentioned previously, guidelines for standard applications of lane closures are detailed in the *Manual on Uniform Traffic Control Devices* (*4*). However, at work zones on some high-volume, high-speed Interstate-type facilities, there may be a need for traffic control devices in addition to those specified as standard applications. For the I-75 pavement restoration project in southern Kentucky during summer 1986, a decision was made by Department of Highways personnel to use the following traffic control devices to supplement standard lane closure devices: variable message sign placed 1 to 2 mi before the taper; supplemental lane closure warning signs 5, 4, 3, and 2 mi before the taper; and rumble strips 1.5, 1.0, 0.6, 0.3, and 0.1 mi before the taper. A summary of primary findings from the evaluation of traffic control devices used in addition to standard lane closure devices follows.

- For all southbound and northbound sites evaluated, there was a decrease in the percentage of traffic in the lane to be closed with the addition of traffic control devices beyond the requirements for devices at standard lane closures.
- There was a decrease in the percentage of traffic in the lane to be closed for southbound sites with each successive traffic control device in addition to the standard devices. The order in which devices were added to the standard lane closure devices was as follows: variable message sign, supplemental lane closure warning signs, and rumble strips placed before the taper.
- Geometric constraints reduced the reliability of data collected at the southbound site when rumble strips were installed in addition to standard lane closure devices, variable message sign, and supplemental signs.
- For one northbound site, the effectiveness of adding the variable message sign to the standard lane closure devices was clearly shown, with a decrease from 21.9 percent to 10.9 percent of the traffic in the lane to be closed 0.1 mi before the taper.
- The effectiveness of rumble strips was demonstrated at the northbound sites when the percentage of traffic in the lane to be closed 0.1 mi before the taper decreased from 11.0 percent with all devices except rumble strips in place to 4.1 percent with rumble strips added.
- Results indicate a general decrease in speeds as traffic approached the taper. However, speed still averaged slightly more than 55 mph in the range 1 mi to 1/2 mi before the taper.
- For almost all data collection points, the percentage of trucks in the lane to be closed was lower than the percentage in the open lane when the closure was a left lane. Overall, the average percentages of trucks for both lanes of traffic ranged from 8.5 to 14.7 percent.
- Average hourly traffic for all sites ranged from 800 to 1,300. Hourly traffic volumes in the range observed in this evaluation did not appear to influence the percentage of traffic in the lane to be closed.
- The percentage of trucks in both lanes did not influence the percentage of traffic in the lane to be closed.
- There does not appear to be a relationship between percentage of trucks in both lanes and percentage of trucks in the lane to be closed.

RECOMMENDATIONS

In general, results of this evaluation indicate that variable message signs, supplemental signs, and rumble strips are effective devices for reducing late merges and provide smoother flow of traffic through lane closures. However, application of these devices in addition to standard lane closure devices should be reserved for special locations where volumes are high and geometric constraints suggest a higher probability for late merges or erratic maneuvers at the closure. Supplemental signs indicating a lane closure 5, 4, 3, and 2 miles ahead should be considered for all long-term closures on high-speed, high-volume four-lane roadways. Variable message signs should be considered at long-term lane closures (in addition to supplemental signs) when one-directional hourly volumes exceed 1,000 (or AADT exceeds 20,000). Application of rumble strips should be reserved for locations where supplemental signs and variable message signs do not reduce late merges and there is excessive congestion due to late merges or other erratic maneuvers at the lane closure.

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Speed Control Through Freeway Work Zones: Techniques Evaluation

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In this paper, the implementation and evaluation of four techniques for improving the effectiveness of speed zoning in construction areas on multilane freeways are presented. The techniques are (a) the flagging procedure of the *Manual On Uniform Traffic Control Devices* (MUTCD), (b) the use of the MUTCD flagging procedure plus having the flagger point at a nearby speed limit sign with the free hand after motioning motorists to slow, (c) a marked police car with cruiser lights and radar active, and (d) a uniformed police officer to control traffic. Each of the techniques was applied continuously on a six-lane freeway for a period of 10 to 15 days. The results of the analysis indicate that all four techniques can provide significant reduction in traffic speed through highway construction zones. The flagging methods were effective in construction areas where one lane remained open to traffic. The law enforcement methods demonstrated a stronger speed reduction capability, particularly when the lane closures result in two or more lanes open. The construction projects used for the collection of field data collection required speed reduction from the regulatory 55 mph to an advisory 45 mph. Although the law enforcement techniques were determined to be effective, their implementation requires a high degree of administrative coordination and cooperation involving police departments, highway officials, and construction contractors.

Use of excessive speed for existing conditions reduces the effectiveness of corrective navigational maneuvers made by motorists as they travel through highway construction zones. The safety of motorists and work crews in construction zones remains an unresolved issue, in spite of numerous techniques for speed control. Traffic accidents in construction sites are a continuing problem. Several studies have concluded that highway construction zones have a propensity for increasing accidents. In a 1965 California accident study (*1*) of 10 randomly selected construction projects, a 21.4 percent increase in the accident rate was observed, with a 132 percent increase in the fatality component. In a study of 207 highway resurfacing projects on two-lane highways, Graham et al. (*2*) reported a 61 percent increase in total accidents, 67 percent increase in injuries, and 68 percent increase in fatalities during construction. The Virginia Highway Research Council (*3*) reported a 119 percent increase in accident frequency in construction zones on I-495 in northern Virginia. The National Safety Council surveys (*4*) show that over 500 people working on the roadway are reported killed by traffic accidents each year.

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There is no doubt that highway construction and maintenance zones increase the potential for traffic accidents. Attention must be focused on innovative traffic control measures that are more responsive to drivers in highway construction zones. This paper examines the long-term effectiveness of two flagging and two law enforcement techniques in reducing speeds in freeway construction zones. These techniques were previously determined to have reasonable promise for reducing speeds during 1–2-hr applications (*5*). The four treatments were

MUTCD Flagging. This is the flagging procedure described in the 1978 edition of the *Manual on Uniform Traffic Control Devices* (MUTCD) (*6*). The flagger, equipped with a red flag and orange vest, performs the "alert and slow" signal detailed in Part IV of the MUTCD.

Innovative Flagging. This flagging technique combines the MUTCD procedure with having the flagger use the other hand (without the flag) to motion traffic to slow and then to point at a nearby speed limit sign (Figure 1).

Stationary Police Cruiser with Lights and Radar on. This technique requires a marked patrol car with cruiser lights and radar in operation to be stationed at the site.

Uniformed Police Traffic Controller. A uniformed officer standing on the side of the road near a speed limit sign manually motions the traffic to slow down.

Two applications of each of the above techniques were studied on a six-lane Interstate freeway in Delaware.

BACKGROUND

The safety of motorists and workers in highway construction zones has been the subject of many research studies (*7–12*). The results of these studies, as well as others, have contributed to major improvements in the way traffic is controlled to improve safety in highway construction zones. The 1978 version of the *Manual on Uniform Traffic Control Devices* (*6*) and its periodical revisions represent the results of years of experimentation and are the national engineering standard for highway traffic control, including traffic control in maintenance and construction zones. In spite of great progress in reducing the accident rates in construction zones, safety remains a continuing issue, primarily because of the tragic nature of accidents in construction zones. The fundamental hypothesis of this research is that further reduction in the rate, frequency, and severity of accidents in construction zones could be obtained

FIGURE 1 "Innovative" MUTCD flagging.

through the use of improved techniques for causing drivers to reduce speeds.

Traffic accidents in highway work zones are caused by a combination of factors, including driver error, inadequate visibility, poor road surface condition, construction obstructions, inadequate traffic control and information, and improper management of material, equipment, and personnel in construction zones. Liberty Mutual Insurance Company (*13*) noted that more than one half of the accidents in the vicinity of road closures are caused by driver error and negligence. Unsafe operating speeds for existing conditions is a frequent driver error. In the review of work zone accidents on rural highways in Ohio, Nemeth (*14*) concluded that in comparison to other causative factors, excessive speed is 5.5 times more frequently cited as the reason for traffic accidents in highway construction areas. Humphries (*15*) studied 103 work zones located in several states and concluded that both unsafe operating speed and inadequate speed control can be blamed for many traffic accidents in highway construction zones. Richards and Faulkner (*16*) studied accidents in Texas and observed that speed violation contributed to 27 percent of work zone accidents, compared to 15 percent for non-work zone accidents. More effective ways are needed to cause motorists to reduce speed in highway construction zones where slower operating speeds are required. The standard practice of using signs to control speeding in work zones is not working. Drivers are generally not responsive to purely advisory and regulatory speed signing in construction zones.

Graham et al. (*2*) conducted experiments to evaluate several speed reduction techniques for highway work zones in the Kansas City metropolitan area. The researchers observed speed, erratic maneuvers, and conflicts at three sites: an urban freeway, a rural freeway, and an urban street. Data collection was limited to 2 to 3.5 hr per technique. The study did not address the long-term speed reduction potential of each technique.

Richards et al. (*5*) studied the short-term effectiveness of a number of work zone speed reduction methods. The flagging technique described in the MUTCD (*6*), an innovative flagging modification of the MUTCD method, police controller, and police car with activated radar on site were among the techniques studied. The study examined the short-term speed-reduction response of motorists to each technique. Observations for each treatment were made over 1 to 2 hr. In comparison to the standard MUTCD flagging method, the innovative flagging treatment resulted in larger speed reduction at five of the six study sites. On the urban freeway site, the innovative flagging treatment reduced speeds by 4 mph (7 percent) and the MUTCD flagging reduced speed by 3 mph (5 percent). These reductions, from a traffic operational standpoint, are not significant. Richards et al. state that the police controller technique was not evaluated at any of the freeway sites because of the reluctance of the police to stand at the roadside. The stationary patrol car reduced speeds by 4 to 12 mph (6 to 22 percent). This method was determined to be most successful on urban arterials and apparently less so on urban freeways. These four reduction techniques were determined to have modest promise on the basis of short-term observations of 1 to 2-hr durations. The unanswered question is whether the potential demonstrated for the short-term application of the four techniques can be reached during long-term application on freeways. Construction activities that last more than 2 weeks are common occurrences on freeways. Thus, the experiments initiated by Richards et al. need to be expanded to cover long-term conditions.

IMPLEMENTATION

Study Sites

Eight study sites were selected on Interstate 495 in the suburbs of Wilmington, Delaware. I-495 is a six-lane divided freeway, with three lanes in each direction. The construction activity was performed in two phases for each bridge. The left and center lanes were closed in phase 1, and the right lane was closed in phase 2. Figure 2 shows the typical two-lane closure used on all sites. The typical one-lane closure is depicted in Figure 3. Figures 2 and 3 also provide information on the location of treatment stations in relation to the sensors at speed

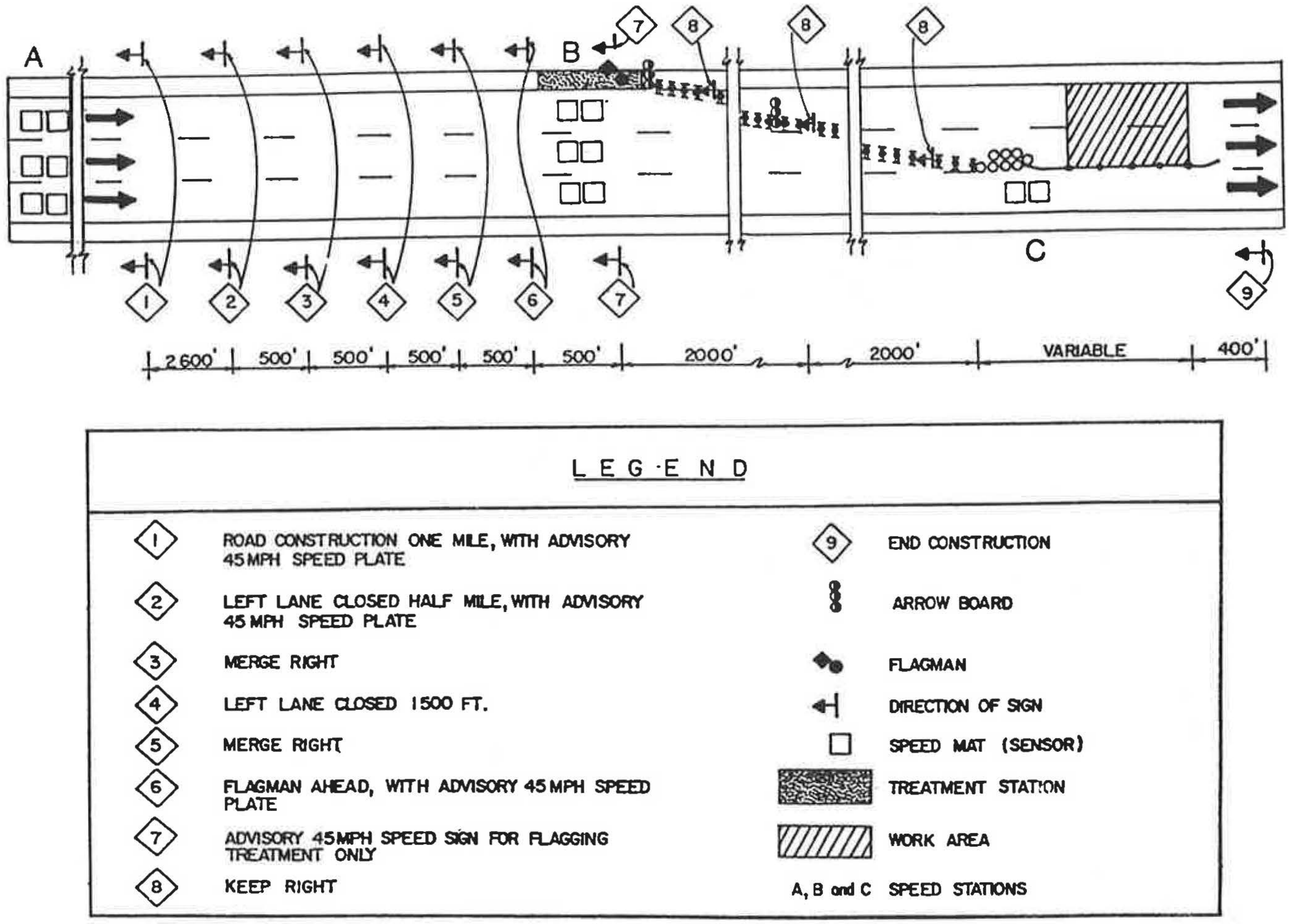

FIGURE 2 Schematic of typical left and center lane closure.

stations A, B, and C. Station A was placed about 5,000 ft upstream of Station B. The regulatory speed limit at Station A was 55 mph. An advisory speed of 45 mph was posted throughout the construction area. All study sites had the same geometrical, topographical, and traffic operating conditions. The distance between B and C was either 2,500 or 4,500 ft, depending on the number of lanes closed. Table 1 provides a listing of the treatments and the spatial separation between speed stations. Traffic control devices in the construction area were not visible from Station A.

Application of Treatments

Each of the four treatments was applied during the two-lane closure phase and then during the one-lane closure phase on the same bridge. No treatment was repeated on any other bridges. For example, the MUTCD flagging was applied only to bridge 802 during phase 1 and phase 2 construction for that bridge (see Table 1 for treatments and lane closures applied to other bridges). The treatment applied to each lane-closure situation remained in place for 10 to 15 days, depending on the schedule of the construction contractor.

The data collection periods were on weekdays only, lasted for approximately 3 hr, involved good weather and dry pavement, and were carefully selected to avoid night conditions and peak traffic periods. VC 1900 traffic analyzers with loop detectors were used to obtain speed, volume, and vehicle classification. Two portable electromagnetic loop detectors mounted on rubber mats (see Figure 4) were placed in each through lane. One VC 1900 traffic analyzer was used at each speed station. Use of the analyzer aided concealment of the experiment and removed the need for the field team to remain on site while data were being automatically collected.

Data Collection Procedure

For each treatment, speed observations were made at the three speed stations (A, B, and C) before test procedures were implemented, within the first three days of implementation, and about 10 to 15 days after implementation. The exact duration of the 10 to 15–day exposure period depended on construction progress. No treatment received less than a 10-day exposure. For each treatment and speed station, at least 100 speed observations per lane were made, except for speed stations that preceded the tapered one- or two-lane closure. Occasionally, the fast lanes were less frequently used than the other two lanes, and this factor resulted in less than 100 speed observations for some time periods.

All the lanes that were open to traffic at the three speed stations were equipped with sensors to detect speed and classify vehicles in two categories (cars and trucks.) The VC 1900 traffic analyzer was programmed to detect the speed and

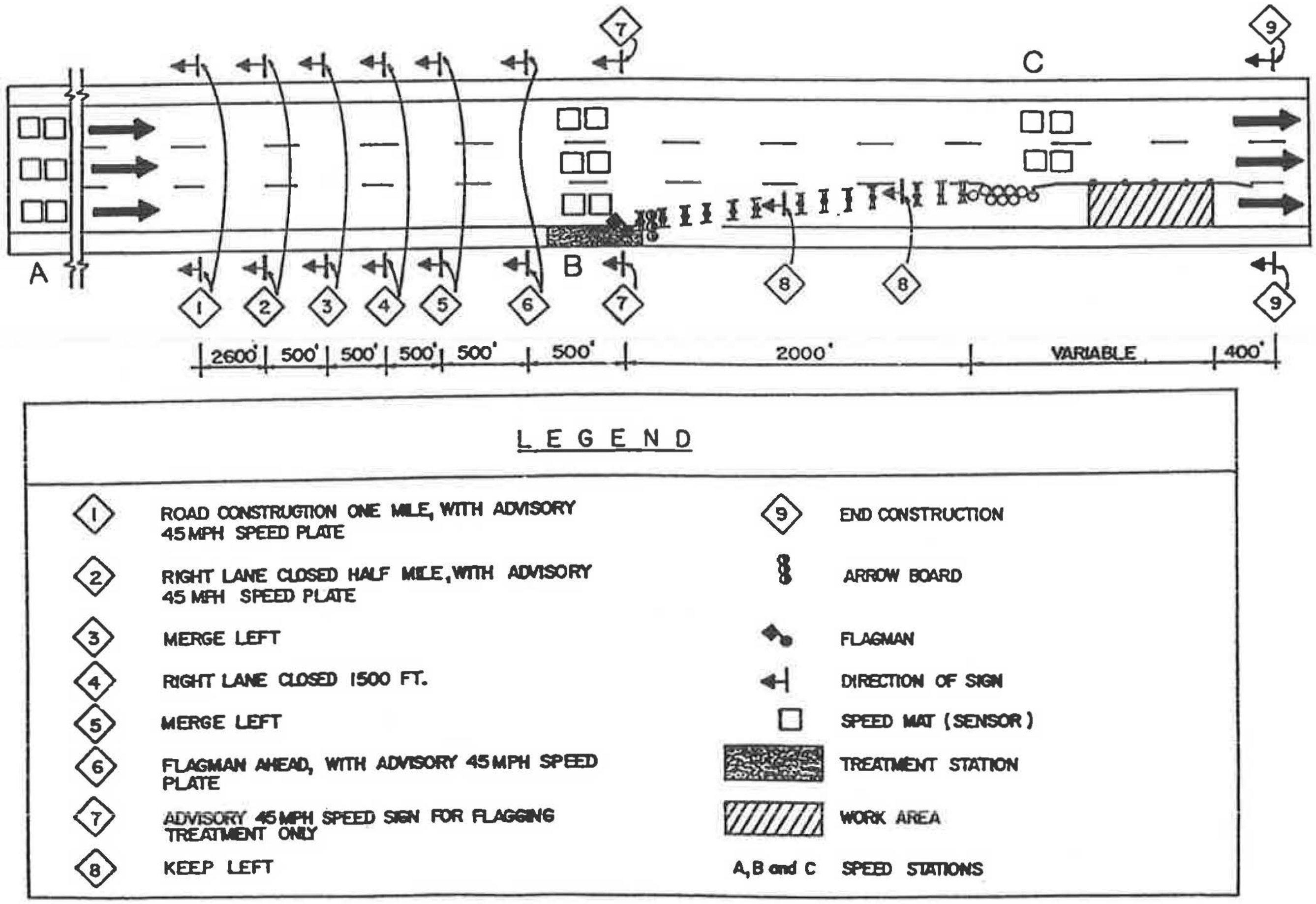

FIGURE 3 Schematic of typical right lane closure.

TABLE 1 LANE CLOSURES AND DISTANCES BETWEEN STATIONS

Treatment Type	Site No.	Freeway	Unidirectional Lanes	Lanes Closed	Distance Between Stations (ft) A–B	B–C
MUTCD	802	I-495S	3	Center, left	5,000	4,500
MUTCD	802	I-495S	3	Right	5,000	2,500
Police Car & Radar	805	I-495N	3	Center, left	5,000	4,500
Police Car & Radar	805	I-495N	3	Right	5,000	2,500
Police Controller	813	I-495N	3	Center, left	5,000	4,500
Police Controller	813	I-495N	3	Right	5,000	2,500
Innovative Flagging	826A	I-495N	3	Center, left	5,000	4,500
Innovative Flagging	826A	I-495N	3	Right	5,000	2,500

NOTE: MUTCD is the flagging procedure in the *Manual of Uniform Traffic Control Devices* (*6*). All sites located in Wilmington, Delaware.

type of vehicles separated by a selected headway of 4 sec. A Husky Hunter portable microcomputer was used to program the traffic analyzers placed at each speed station. Vehicle data were electronically stored in the memory of the traffic analyzer and were retrieved periodically with a Kaypro 2000 portable microcomputer, which is compatible with the IBM Personal Computer. Once the equipment at all speed stations was programmed for data collection, the field team left the stations and took on a supervisory role, periodically observing the equipment.

Data Reduction

The means and standard deviations of speed for each treatment are presented in Table 2. Although the long-term speed reduction capability of some treatments is already indicated by the tabulation of unadjusted data (for example, the police car and radar treatment shows a consistent decrease in speed from base to long-term periods), consideration must be given the speed changes due to differences in driver population across the periods.

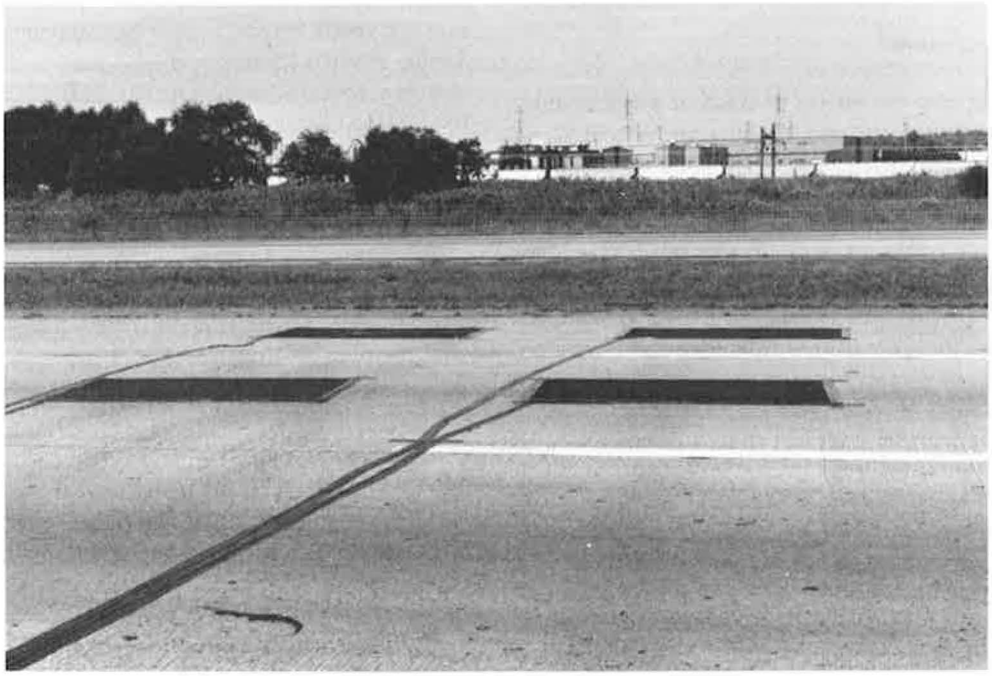

FIGURE 4 Mat-mounted electromagnetic loop detectors.

TABLE 2 MEANS AND STANDARD DEVIATION OF SPEEDS OF ALL VEHICLES

Treatment	Lanes Closed	STATION A Base $\bar{X}$	STATION A Base S	STATION A Short-Term $\bar{X}$	STATION A Short-Term S	STATION A Long-Term $\bar{X}$	STATION A Long-Term S	STATION B Base $\bar{X}$	STATION B Base S	STATION B Short-Term $\bar{X}$	STATION B Short-Term S	STATION B Long-Term $\bar{X}$	STATION B Long-Term S	STATION C Base $\bar{X}$	STATION C Base S	STATION C Short-Term $\bar{X}$	STATION C Short-Term S	STATION C Long-Term $\bar{X}$	STATION C Long-Term S
MUTCD Flag	CL & LL	60.8	5.7	57.7	6.5	58.4	8.3	57.3	7.8	60.9	7.2	58.2	7.1	60.4	6.1	52.6	6.6	54.2	7.7
MUTCD Flag	RL	62.2	7.1	58.3	7.8	59.0	8.0	58.8	6.0	55.1	6.0	60.0	5.8	60.5	7.1	59.2	7.1	58.1	4.9
INN. Flag	CL & LL	55.6	6.8	57.9	7.1	59.0	7.9	61.8	7.7	57.7	7.8	58.5	7.1	60.8	7.2	59.5	7.8	61.2	7.0
INN. Flag	RL	58.8	5.7	56.0	5.5	57.2	5.5	57.5	6.5	56.1	6.6	55.6	6.6	63.8	6.3	59.3	6.2	63.6	6.2
Police Car and Radar	CL & LL	55.6	5.4	56.3	6.5	58.3	5.9	56.6	5.6	60.9	7.0	53.9	5.6	63.6	6.2	60.3	7.6	59.9	6.1
Police Car and Radar	RL	59.0	8.2	57.9	6.0	57.6	5.7	60.2	6.6	56.7	5.6	59.3	6.0	66.7	5.5	61.6	5.2	56.9	8.1
Police Controller	CL & LL	56.7	6.8	57.7	6.5	58.4	6.6	58.9	7.0	53.6	6.6	53.8	6.4	62.0	7.6	57.7	7.4	60.4	6.7
Police Controller	RL	54.9	7.0	57.8	7.1	57.2	6.9	55.5	6.9	53.3	6.8	55.5	6.2	59.9	8.0	59.3	7.1	58.9	6.6

*located in active construction area $\bar{X}$ = means speed (mph) S = standard deviation

Statistical Method

All statistical analyses were done on an AT&T microcomputer using the Statistical Analysis System for Personal Computers (PC-SAS). The experimental design provided statistical controls for site differences and driver populations within sites by incorporating speed data from a base station (Station A) and a base period across all stations. A one-way analysis of variance procedure was used to compare mean driver speeds among the treatments. The driver speeds were adjusted for potential differences in the driving population before the analysis by subtracting the mean driver speeds at the base station (Station A). This adjustment assumes that the driver speeds at Station A adequately reflect the speeds of the population of drivers and that this population of drivers has the same variability at all stations. This assumption of equal variability was statistically tested and found to be valid at the .05 level of significance. The mean driver speeds among the stations were ranked and compared using the Scheffé method of multiple comparison (*17*). The individual levels of significance for these multiple comparison tests were adjusted so that the overall conclusions drawn are reliable at the .05 level of significance.

EVALUATION OF SPEED CONTROL TREATMENTS

Measure of Effectiveness

This analysis compared the effects of the four speed control treatments (Police Radar, Police Controller, Innovative Flagging, MUTCD Flagging) during the base (reference condition without any treatment) and short-term (within a few days after implementation of the treatment) time periods and during the base and long-term (10 to 15 days of continuous exposure) periods. The effect of the treatment was evaluated on the basis of the estimated expected speed change at Station C, adjusted for the actual speed change at the upstream base station, Station A. When the effect of the treatments at the point of application was assessed, Station B was used in place of Station C. However, the most dramatic treatment effect was anticipated at Station C.

The unadjusted speed change at Station C due to a speed control treatment was estimated by subtracting the average speed at Station C during the short-term period from the average speed during the base time period. This average speed change at Station C was then adjusted for differences in speeds that might be anticipated under no speed control treatment conditions (i.e., differences due to changes in the driver populations between the base and early periods). The net speed change was estimated by subtracting the average speed at Station A during the base time period. Because traffic at the upstream Station A was not influenced by the speed control treatment implemented near Station B, changes in average speed at Station A (between the base and short-term periods) could be assumed to be the result of differences in driver populations. Thus average speed differences at Station C were adjusted accordingly. The same procedure was used in estimating the net speed change at Station B.

Table 3 summarizes the estimated average (net) speed changes at stations A and C and the expected net average speed changes at Station C after adjusting for Station A speed differences. For example, for the MUTCD Flagging one-lane closure for all vehicles (cars and trucks), the difference in average speed between the base and long-term periods at Station A was −3.9 mph and −1.3 mph at Station C, for a net change at Station C of +2.6 mph [(−1.3) − (−3.9)]. Note that for the MUTCD Flagging speed control treatment to be effective, the average speed at Station C should have decreased by more than 3.9 mph. However, there was actually a net increase in speed at Station C.

A two-way analysis of variance model was applied to the base and long-term data of Station C for all treatments, adjusted for Station A speeds for each respective treatment period. The factors in the analysis of variance were site, treatment period (base or short-term), and site by treatment interaction. The interaction hypothesis in these two-way analysis of variance tables was equivalent to testing equality among the speed changes in columns labeled "net change" in Table 3. The adjusted (net change) estimates were tested using a modified interaction test.

If a significant overall difference in net speed change was found, the next step was to determine which treatments were different. This was done by using the Scheffé test for multiple comparisons at the overall level of significance of .05 for three contrasts.

The results of these statistical tests are summarized in Table 4. These results were interpreted separately for one- and two-lane closure conditions.

Short-Term Exposure at Station C

One-Lane Closure

Statistically, the Police Radar and Police Controller treatments were equally effective, with net average speed changes of −4.0 and −3.5 mph. Both were significantly more effective in reducing speeds than the Innovative Flagging and the MUTCD Flagging treatments, with net average speed changes of −1.7 and +2.6 mph, respectively, during the field studies. However,

TABLE 3 AVERAGE SPEED CHANGES (MPH), BASE VERSUS SHORT-TERM PERIODS

	One-Lane Closure			Two-Lane Closure		
	Station A	Station C	Net Change Station C	Station A	Station C	Net Change Station C
All Vehicles						
MUTCD	-3.9	-1.3*	+2.6*	-3.1	-7.8*	-4.7*
Police with Radar	-1.1	-5.1*	-4.0	+0.7	-3.3*	-4.0*
Police Controller	+2.9	-0.6*	-3.5*	+1.0	-4.3*	-5.3*
Innovative Flagging	-2.8	-4.5*	-1.7*	+2.3	-1.3*	-3.6*
Cars						
MUTCD	-3.5	-0.7	+2.8	-2.7	-7.1	-4.4
Police with Radar	-1.3	-5.0	-3.7	+0.5	-3.0	-3.5
Police Controller	+2.3	0.0	-2.3	+0.7	-3.9	-4.6
Innovative Flagging	-2.1	-4.5	-2.4	+2.6	-1.3	-3.9
Trucks						
MUTCD	-6.8	-3.0	+3.8	-2.1	-9.3	-7.2
Police with Radar	-2.0	-5.1	-3.1	+1.7	-4.2	-5.9
Police Controller	+3.8	-0.6	-4.4	-0.1	-4.9	-4.8
Innovative Flagging	-1.4	-4.8	-3.4	+1.4	-1.5	-2.9

M.P.H = Miles per hour

TABLE 4 RANKING WITHIN ONE- OR TWO-LANE CLOSURES AT STATION C: SHORT-TERM TREATMENT EFFECT

One-Lane Closure			Two-Lane Closure		
Rank	Treatment	mph	Rank	Treatment	mph
All Vehicles					
1	Police Controller	−4.0	1	Police Controller	−5.3
2	Police Controller	−3.5	2	MUTCD Flagging	−4.7
3	Innovative Flagging	−1.7	3	Police Radar	−4.0
4	MUTCD Flagging	+2.6	4	Innovative Flagging	−3.6
Cars					
1	Police Radar	−3.7	1	Police Controller	−4.6
2	Innovative Flagging	−2.4	2	MUTCD Flagging	−4.4
3.	Police Controller	−2.3	3	Innovative Flagging	−3.9
4	MUTCD Flagging	+2.8	4	Police Radar	−3.5
Trucks					
1	Police Controller	−4.4	1	MUTCD Flagging	−7.2
2	Innovative Flagging	−3.4	2	Police Radar	−5.9
3	Police Radar	−3.1	3	Police Controller	−4.8
4	MUTCD Flagging	+3.8	4	Innovative Flagging	−2.9

because the differences in speed reductions for the Police Radar and the Police Controller were at most 2.3 mph greater [(−4.0) − (−1.7)] than the Innovative Flagging, from a practical standpoint, it cannot be said that the Police Radar and Police Controller treatments were better than the Innovative Flagging.

The net average speed increase of 2.6 mph for the MUTCD Flagging was significantly different from any of the other treatment effects. It should be noted that the site at which the MUTCD Flagging was studied was the first site for data collection and analysis.

Analysis of cars only indicated that the Police Radar treatment, with an average speed change of −3.7 mph, was better at a statistically significant level than the other treatments. The Innovative Flagging treatment was found to be as effective as the Police Controller treatment in reducing average speeds. There was no statistically significant difference in average speed reductions between the Innovative Flagging and Police Controller treatments (−2.4 versus −2.3 mph). From a practical standpoint, there was no difference between the Police Radar, Police Controller and Innovative Flagging treatments.

The net average speed of cars during the MUTCD Flagging treatment increased by 2.8 mph. The results of the analysis of truck-only data were similar to those for cars only. The Police Controller, Innovative Flagging, and Police Controller resulted in statistically significant reductions in net average truck speeds of 4.4, 3.4, and 3.1 mph. The MUTCD Flagging resulted in a net increase of 3.8 mph in average truck speed.

Two-Lane Closure

For the two-lane closure condition, the net average vehicle speeds for all four speed control treatments were both statistically and practically lower than the speeds during the base conditions. The average speeds were reduced by a net of 5.3, 4.7, 4.0, and 3.6 mph for the Police Controller, MUTCD Flagging, Police Radar, and Innovative Flagging treatments. There were no statistically significant differences among the four treatments for all vehicles, cars only, or trucks only.

The results of the data for the two-lane closure are somewhat surprising in comparison to the one-lane closure. For example, it is difficult to understand why the MUTCD Flagging treatment would be effective for two-lane closures and not effective for one-lane closures. One proposed theory is that an experimental artifact may have biased the results at Station C during the two-lane closures. For example, it is possible that drivers were forced to reduce speeds to merge into one open lane during the two-lane closures. Thus the speed reductions may have been tempered by things other than the speed control treatments, in spite of efforts to collect data in free-flowing traffic.

Short-Term Exposure at Station B

Station B was analyzed with the same procedures that were used at Station C. Both short-term and long-term speed control treatment effects were evaluated, with adjustments for differences in Station A speeds. Tables 5 and 6 summarize the speed changes at stations A and B, the net adjusted speed change, and the ranking and results of statistical tests of significance of these net changes.

One-Lane Closure

For the one-lane closure for all vehicles (cars and trucks), all treatments showed a statistically significant change in net speeds. The Police Controller and the Police Radar treatments both significantly reduced net speeds (−5.1 and −2.4 mph). The Police Controller treatment resulted in a significantly lower net speed than the Police Radar treatment. The MUTCD Flagging and the Innovative Flagging treatments resulted in increases in net speed (+0.2 and +1.4 mph). The changes in speed that resulted from the Police Radar, MUTCD Flagging, and Innovative Flagging treatments were not considered to be of practical significance (i.e., there were essentially no changes in net speeds with these treatments).

For cars and trucks as one group, the Police Controller treatment was effective in reducing speeds. The Police Radar

TABLE 5 AVERAGE SPEED CHANGES (MPH), BASE VERSUS EARLY PERIODS

	One-Lane Closure			Two-Lane Closure		
	Station A	Station B	Net Change Station B	Station A	Station B	Net Change Station B
All Vehicles						
MUTCD	-3.9	-3.7	+0.2	-3.1	+3.6	+6.7
Police with Radar	-1.1	-3.5	-2.4	+0.7	+4.3	+3.6
Police Controller	+2.9	-2.2	-5.1	+1.0	-5.3	-6.3
Innovative Flagging	-2.8	-1.4	+1.4	+2.3	-4.1	-6.4
Cars						
MUTCD	-3.5	-3.6	-0.1	-2.7	+3.2	+5.9
Police with Radar	-1.3	-3.9	-2.6	+0.5	+4.2	+3.7
Police Controller	+2.3	-2.0	-4.3	+0.7	-5.6	-6.3
Innovative Flagging	-2.1	-2.1	0.0	+2.6	-4.1	-6.7
Trucks						
MUTCD	-6.8	-2.7	+4.1	-2.1	+4.3	+6.4
Police with Radar	-2.0	-1.9	+0.1	+1.7	+5.2	+3.5
Police Controller	+3.8	-3.1	-6.9	-0.1	-5.6	-5.5
Innovative Flagging	-1.4	+1.9	+3.3	+1.4	-3.2	-4.6

M.P.H = Miles per hour

TABLE 6 RANKING WITHIN ONE- OR TWO-LANE CLOSURES AT STATION B: SHORT-TERM TREATMENT EFFECT

One-Lane Closure			Two-Lane Closure		
Rank	Treatment	mph	Rank	Treatment	mph
All Vehicles					
1	Police Controller	−5.1	1	Innovative Flagging	−6.4
2	Police Radar	−2.4	2	Police Controller	−6.3
3	MUTCD Flagging	+0.2	3	Police Radar	+3.6
4	Innovative Flagging	+1.4	4	MUTCD Flagging	+7.8
Cars					
1	Police Controller	−4.3	1	Innovative Flagging	−6.7
2	Police Radar	−2.6	2	Police Controller	−6.3
3.	MUTCD Flagging	−0.1	3	Police Radar	+3.7
4	Innovative Flagging	0.0	4	MUTCD Flagging	+5.9
Trucks					
1	Police Controller	−6.9	1	Police Controller	−5.5
2	Police Radar	+0.1	2	Innovative Flagging	−4.6
3	Innovative Flagging	+3.3	3	Police Radar	+3.5
4	MUTCD Flagging	+4.1	4	MUTCD Flagging	+6.4

treatment was effective in reducing car speeds but resulted in no effect for trucks. The Innovative Flagging and the MUTCD Flagging treatments were found to be equal in effect. No net speed change was found for cars, and speed increases were found for trucks.

Two-Lane Closure

For the two-lane closure, the Innovative Flagging and the Police Controller resulted in very significant net reductions in speed (−6.4 and −6.3 mph). The MUTCD Flagging and the Police Radar resulted in significant increases in net speeds (+6.7 and +3.6 mph). The increase in speed using the MUTCD Flagging treatment was significantly higher than the increase with the Police Radar treatment.

Long-Term Exposure at Station B

Results of the speed changes at Stations A and B, the net speed changes (adjusted speeds), and the ranking and results of statistical tests of significance are summarized in Tables 7 and 8.

One-Lane Closure

For the one-lane closure for all vehicles, only the Police Controller resulted in a statistically significant reduction in net speed. However, the −2.3 mph speed change was not of practical significance. The Innovative Flagging and the Police Radar treatments had no effect on net speeds. The MUTCD Flagging resulted in a statistically and practically significant increase in net speeds: the change was +4.4 mph. For cars,

TABLE 7 AVERAGE SPEED CHANGES (MPH), BASE VERSUS LONG-TERM PERIODS

	One-Lane Closure			Two-Lane Closure		
	Station A	Station B	Net Change Station B	Station A	Station B	Net Change Station B
All Vehicles						
MUTCD	-3.2	+1.2	+4.4	-2.4	+0.9	+3.3
Police with Radar	-1.4	-0.9	+0.5	+2.7	-2.7	-5.4
Police Controller	+2.3	0.0	-2.3	+1.7	-5.1	-6.8
Innovative Flagging	-1.6	-1.9	-0.3	+3.4	-3.3	-6.7
Cars						
MUTCD	-2.2	+1.1	+3.3	-3.4	+0.6	+4.0
Police with Radar	-1.5	-1.2	+0.3	+2.0	-3.0	-5.0
Police Controller	+1.7	+0.6	-1.1	+0.9	-5.1	-6.0
Innovative Flagging	-1.4	-2.7	-1.3	+3.2	-3.3	-6.5
Trucks						
MUTCD	-6.8	+2.7	+9.5	+2.4	+1.6	-0.8
Police with Radar	-1.8	+0.3	+2.1	+1.0	-0.5	-1.5
Police Controller	+3.9	-3.1	-7.0	+3.9	-5.4	-9.3
Innovative Flagging	-0.4	+1.4	+1.8	+1.4	-2.9	-4.3

M.P.H = Miles per hour

TABLE 8 RANKING WITHIN ONE- OR TWO-LANE CLOSURES AT STATION B: LONG-TERM TREATMENT EFFECT

One-Lane Closure			Two-Lane Closure		
Rank	Treatment	mph	Rank	Treatment	mph
All Vehicles					
1	Police Controller	−2.3	1	Police Controller	−6.8
2	Innovative Flagging	−0.3	2	Innovative Flagging	−6.7
3	Police Radar	+0.5	3	Police Radar	−5.4
4	MUTCD Flagging	+4.4	4	MUTCD Flagging	+3.3
Cars					
1	Innovative Flagging	−1.3	1	Innovative Flagging	−6.5
2	Police Controller	−1.1	2	Police Controller	−6.0
3.	Police Radar	+0.3	3	Police Radar	−5.0
4	MUTCD Flagging	+3.3	4	MUTCD Flagging	+4.0
Trucks					
1	Police Controller	−7.0	1	Police Controller	−9.3
2	Innovative Flagging	+1.8	2	Innovative Flagging	−4.3
3	Police Radar	+2.9	3	Police Radar	−1.5
4	MUTCD Flagging	+9.5	4	MUTCD Flagging	−0.8

none of the treatments resulted in any practical changes in net speed. However, for trucks, the Police Controller resulted in a −7.0 mph change in speed, whereas the MUTCD Flagging treatment resulted in a +9.5 mph change in speed.

Two-Lane Closure

For the two-lane closure, all treatments except the MUTCD Flagging treatment reduced net speeds significantly. The Police Controller, Innovative Flagging, and Police Radar resulted in net speed changes of −6.8, −6.7, and −5.4 mph. The MUTCD Flagging resulted in a 3.3 mph increase in speed. The results with respect to decreases and increases in net speeds were repeated when the car data alone were analyzed. However, for trucks only, the Police Radar and the MUTCD Flagging treatments resulted in no significant change in net speeds.

Long-Term Exposure at Station C

The speed changes at stations A and C between the base and long-term treatment periods and the net speed change for Station C adjusted for Station A speeds are listed in Table 9. Rankings of the speed control treatments and the results of statistical tests of significance among these treatments are presented in Table 10. If there was a long-term speed control treatment effect, the results of this analysis should agree with those of the short-term treatment effect at Station C.

One-Lane Closure

For the long-term period sample with all vehicles (cars and trucks) and one-lane closure, the rankings of the treatments agree with the short-term treatment analysis. However, the data indicated that the Police Radar treatment improved with time.

TABLE 9 RANKING WITHIN ONE- OR TWO-LANE CLOSURES AT STATION C: LONG-TERM TREATMENT EFFECT

One-Lane Closure			Two-Lane Closure		
Rank	Treatment	mph	Rank	Treatment	mph
All Vehicles					
1	Police Radar	−8.4	1	Police Radar	−6.4
2	Police Controller	−3.3	2	MUTCD Flagging	−3.8
3	MUTCD Flagging	+0.8	3	Police Controller	−3.3
4	Innovative Flagging	+1.4	4	Innovative Flagging	−3.0
Cars					
1	Police Radar	−8.7	1	Police Radar	−5.8
2	Police Controller	−2.4	2	Police Controller	−3.5
3.	MUTCD Flagging	−0.4	3	Innovative Flagging	−3.2
4	Innovative Flagging	+1.2	4	MUTCD Flagging	−2.3
Trucks					
1	Police Controller	−6.4	1	MUTCD Flagging	−10.6
2	Innovative Flagging	−4.9	2	Police Radar	−4.1
3	Police Radar	+0.1	3	Police Controller	−1.5
4	MUTCD Flagging	+6.2	4	Innovative Flagging	−0.8

TABLE 10 AVERAGE SPEED CHANGES (MPH), BASE VERSUS LATE PERIODS

	One-Lane Closure			Two-Lane Closure		
	Station A	Station C	Net Change Station C	Station A	Station C	Net Change Station C
All Vehicles						
MUTCD	-3.2	-2.4	+0.8	-2.4	-6.2	-3.8
Police with Radar	-1.4	-9.8	-8.4	+2.7	-3.7	-6.4
Police Controller	+2.3	-1.0	-3.3	+1.7	-1.6	-3.3
Innovative Flagging	-1.6	-0.2	+1.4	+3.4	+0.4	-3.0
Cars						
MUTCD	-2.2	-2.6	-0.4	-3.4	-5.7	-2.3
Police with Radar	-1.5	-10.2	-8.7	+2.0	-3.8	-5.8
Police Controller	+1.7	-0.7	-2.4	+0.9	-2.6	-3.5
Innovative Flagging	-1.4	-0.2	+1.2	+3.2	0.0	-3.2
Trucks						
MUTCD	-6.8	-0.6	+6.2	+2.4	-8.2	-10.6
Police with Radar	-1.8	-8.2	-6.4	+1.0	-3.1	-4.1
Police Controller	+3.9	-1.0	-4.9	+3.9	+2.4	-1.5
Innovative Flagging	-0.4	-0.3	+0.1	+1.4	+0.6	-0.8

M.P.H = Miles per hour

The net change in average speed with the Police Radar treatment was −8.4 mph. This reduction is also better at a statistically significant level than the Police Controller treatment, which caused a net speed change of −3.3 mph during the long-term period. Neither Innovative Flagging nor MUTCD Flagging were significant in reducing speeds, and although there were speed increases for both of these treatments, the increases were neither statistically nor practically significant. It should be noted that the net speed increase for the MUTCD Flagging during the short-term period was statistically significant.

For cars only, all treatments were significantly different from each other. For trucks only, the net speed changes for all treatments were significant but equal.

Two-Lane Closure

For the two-lane closure, all speed control treatments resulted in a net average speed reduction during the long-term period. However, the Police Radar treatment reduced net speeds by an even greater amount than in the short-term treatment period. When vehicle types were separated, however, this improvement was not statistically significant for cars. For trucks the new speed change for the MUTCD Flagging became significantly higher than it was during the short-term period. The sample sizes for trucks in this analysis were extremely low for some treatments, however, and the variability was higher (as evidenced in the results of statistical equality between the Police Radar and Innovative Flagging treatments, despite a 3.3-mph difference).

SUMMARY OF RESULTS

The basic theory is that the speed reduction treatments applied at Station B, where all the freeway lanes are opened to traffic, will result in reduced speed at Station C, located in the area of active construction. Lane closure refers to the reduction of the number of lanes opened to traffic at Station C only. A summary of results is presented next.

Station C with One-Lane Closure

The results indicate that the Police Radar and the Police Controller were effective in reducing vehicle speeds in both the short term (about 3 days) and the long term (more than 10 days) after the speed control treatments were implemented on the freeway work sites studied. The Innovative Flagging speed control treatment elicited a speed decrease of less than 2 mph in the short term. From a practical sense, however, it cannot be said that the Police Radar and Police Controller treatments were better than Innovative Flagging. In the long term, the Innovative Flagging did not result in speed reductions at Station C. The MUTCD Flagging treatment actually resulted in a small increase in speed in both the short and long term.

Station C with Two-Lane Closure

Significant reductions in speeds were experienced in both short term and long term for all four speed control treatments when two of the three freeway lanes were closed. The amount of speed reduction was the same statistically for each treatment. The exception was the Police Radar treatment, which resulted in a greater long-term speed reduction.

Station B with One-Lane Closure at C

The Police Controller was the only speed control treatment that resulted in a significant (both statistically and practically) short-term speed reduction at Station B. The Police Controller also resulted in a long-term speed reduction; however, the reduction was only 2.3 mph, which was not considered to be of practical significance. There was essentially no long-term speed reduction for the Police Radar or the Innovative Flagging treatments. In the long term, the MUTCD Flagging treatment resulted in an increase in speed.

Station B with Two-Lane Closure at C

Significant long-term speed reductions were found at Station B when the Police Controller, Police Radar, or Innovative Flagging treatments were used. There was a significant long-term speed increase during the MUTCD Flagging operations.

CONCLUSIONS

The results of this research indicate that the long-term (more than two weeks) application of all the tested speed control treatments can derive significant reduction in traffic speed through the work area in highway construction zones. However, the effectiveness of the treatments appears to depend on the number of lanes that remain open to traffic in the work area. The flagging techniques are effective in reducing speed in the work area of multilane freeways where one lane is open to traffic. It should be noted, however, that the entire data collection effort was conducted under ideal traffic conditions, with level of service A. It stands to reason that at lower levels of service (higher lane volume) the flagging methods could have increased effectiveness during one-lane closures.

The law enforcement methods demonstrated strong long-term speed reduction capability. This finding, however, must be evaluated with due consideration given to the normal level of law enforcement activity on the freeways. In this research, all the study sites were located on facilities where there was already an exceptionally high level of police patrol. Thus most motorists were already aware of the high probability of being ticketed and saw compliance with speed control as the convenient option. Jurisdictions in which the police force does not have a reputation for enforcing the speed limit may not obtain significant reductions in speed via law enforcement methods. Consistent enforcement of speed limits will facilitate the effectiveness of speed control techniques that use law enforcement.

RECOMMENDATION

When this research began, the study team contacted highway officials in several states, seeking their cooperation in implementing the data collection on construction sites. Every contacted state official said that speeding through highway construction zones was a serious continuing problem, and most were skeptical about any solution. This skepticism appears to be rooted in the scarcity of resources for effective implementation of speed control methods and the inability to establish an integrated administrative mechanism that would enable the speed reduction methods of this research to be included in construction specifications as part of the traffic control plan. The engineer responsible for developing the traffic control plan should select a safe operating speed for the work zone and determine the need for specific speed reduction measures. Because the effectiveness of using police officers for speed control was noteworthy in this study, state and local highway agencies are encouraged to make special contractual provisions for implementation of law enforcement treatments into the traffic control plans. These provisions should include procedures for obtaining off-duty police personnel for the work sites, compensation, lists of contact people, applicable union requirements, scheduling, dress, and equipment.

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Safety in Construction Zones Where Pavement Edges and Dropoffs Exist

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In this paper, the development of "Guidelines for Warning and Protective Devices for Pavement Dropoffs" are described. Included in this development are summaries of pertinent information from the literature, new analyses of vehicle stability, and the results of accident probability studies and benefit-cost studies. Four different safety-related vehicle-pavement dropoff interactions were analyzed and evaluated: nibbling, scrubbing, dragging, and rolling. These interactions are described in detail in the paper. A wide range of vehicle sizes was considered in developing the guidelines, from small automobiles to large tractor semi-trailers. Pavement edges and dropoffs can pose a significant hazard under some construction conditions and need to be carefully considered and dealt with appropriately. The guidelines presented here are now in use by the State Department of Highways and Public Transportation in Texas.

This paper is based on the senior author's published work in this area (*1–4*), on a review of other significant literature, on direct experience with the analysis of construction zone and pavement edge-related accidents, and on an understanding of vehicle dynamics for both automobiles and trucks. Bicycles and motorcycles were not considered in this work. Four levels of vehicle interaction with pavement edges were identified (Figure 1):

Level 1: Nibbling;
Level 2: Scrubbing;
Level 3: Drag; and
Level 4: Roll.

Nibbling is associated with pavement longitudinal edges not more than 1 in. in height. This interaction was not considered to have significance for safety but was included for analysis to ensure that understanding was accurate. When tires are traversing a nibbling edge, a force is imparted to them that may move the vehicle laterally a small distance. Although this is not a control problem for automobiles or stable truck configurations, it could initiate some degree of oscillation in double or triple bottoms at critical speeds.

Scrubbing is the classic edge phenomenon that has been recognized as a safety problem. It is a resistance to edge traversal that can result in loss of vehicle control once the vehicle has mounted the pavement edge. Scrubbing was considered to occur at edge height levels of 1 to 5 in. This interaction does not usually affect safety at 1-in. edge height, but such an effect is possible. Scrubbing loses safety significance for automobiles as edges exceed 5 in. because automobiles are rarely able to mount edges this high. For trucks, however, scrubbing will be important at larger edge heights.

Drag occurs when the edge height exceeds the clearance of the vehicle crossing the edge. As a safety problem, it was considered of lesser significance than scrubbing because in most cases the only problem is damage to vehicle undercarriage elements and possibly the hazard posed for other vehicles by the vehicle that is stopped by dragging. Because most vehicles have their fuel lines routed along the frame or lower monocoque structure, dragging could also result in rupture of these lines, as well as in damage to the brake lines. It was considered possible, under some drag conditions, that the eccentric, friction-type drag force could cause a vehicle spin-out. A spin-out at significant speed may roll the vehicle. It was determined that this possible phenomenon would be investigated.

Roll is a very significant safety consideration. If the edge drop is very high (initially considered to be more than 1 ft), the possibility of a vehicle roll was the final, or Level 4, consideration. Preliminary computations related to vehicle center of gravity (cg) height, track width, and ground clearance indicated that static rollover would not be likely if the edge height were less than half the track width of automobiles. This was true for automobiles but not for trucks because the ratio of truck cg height to track width is much smaller than the same ratio for automobiles. High-cg trucks may roll when the edge drop is no more than 1 ft. A dynamic analysis could give quite different results, however, including the problem of a vehicle digging into a soft shoulder surface when it runs off the edge. This consideration dictated that the edge drop distance to produce vehicle rolling should be assessed by using vehicle simulation models. This analysis will be described in the section of this paper that examines roll.

NIBBLING

"Nibbling" is a term that comes from the tire-manufacturing industry. It probably comes from the idea that a tire rolling immediately adjacent to a longitudinal pavement edge or "seam" of low height nibbles at the edge until it gets a good bite, and then the tire-edge interaction forces pull the tire up onto the higher-level pavement.

Marshal et al. (*5*) defines nibbling as "the process which occurs when a tire encounters a road seam of moderate height at an angle of attack of five degrees or less." The literature

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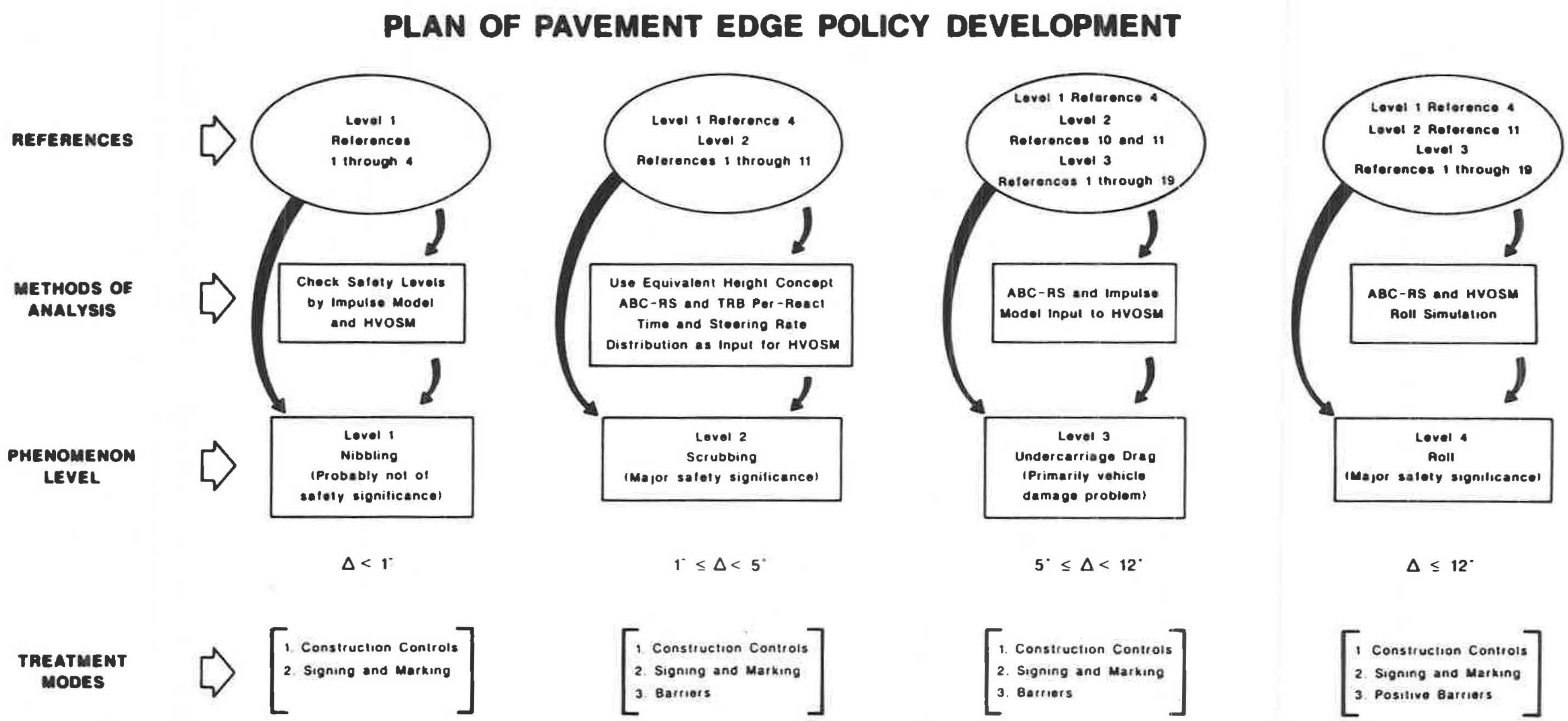


FIGURE 1 Categorization of pavement edges for study planning.

indicates that significant nibbling only occurs when there is a very sharp edge that is from 0.5 to 1 in. high.

Figure 2 shows the tire lateral forces that occur when a tire crosses a small edge or "seam," which is the British term. The "road data" curve is the most interesting one. Lateral forces of up to 160 lb are generated over the time period necessary to traverse the edge.

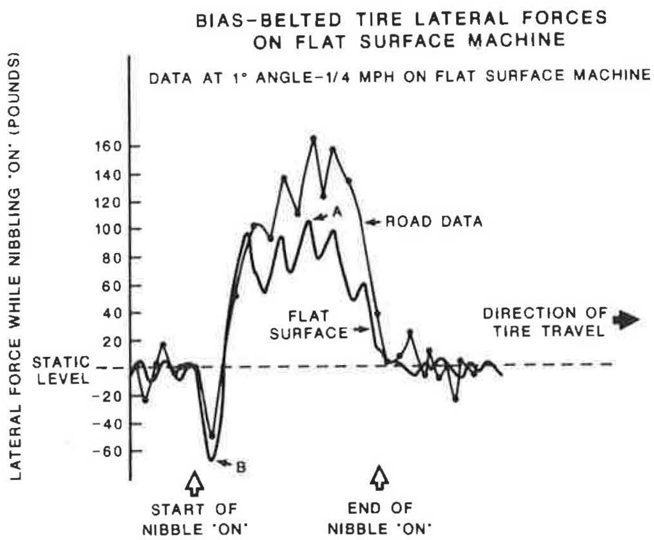


FIGURE 2 Edge-mounting forces involved in the nibbling phenomenon [after Pottinger (*6*)].

To check the way in which this pair of impulses, first on front and then on rear wheel, would change the path of a vehicle, the simulation HVOSM (*6*) was used. A mini-compact vehicle was selected in the belief that the path deviation of a small vehicle would be greater than that of a larger vehicle. The small influence of nibbling forces was illustrated by applying the impulse first to the right front wheel and then, 0.11 sec later, beginning the impulse to the right rear wheel. Each lateral force of 100 lb at the tire-pavement interface was applied for 0.33 sec. No steering was applied to the vehicle during these impulses and for 2.5 sec thereafter. The time 2.5 sec was chosen because it is a common value used by AASHTO for "design" perception-reaction time. The lateral movement of the simulated vehicle was less than 1 ft from the straight line path, confirming the belief of the current authors that nibbling was possibly a factor of irritation for an automobile driver but not one related to safety. This is illustrated in Figure 3.

One possible exception to that conclusion should be stated. If an edge capable of producing tire nibbling is located 9 to 4 ft laterally from a significant pavement edge (i.e., one that might produce scrubbing), a vehicle might be influenced adversely if the driver allowed it to cross the higher edge to avoid the irritation of the nibbling edge. It is also possible that if a "nibbling" edge occured within the 9 to 4 ft specified, it would move or influence the driver to inadvertently move the vehicle laterally into contact with a construction barrier or channeling device. The 9-ft distance was chosen as 1 ft greater than the track width of the largest typical highway truck or tractor-trailer. The 4-ft distance is slightly less than the track width of the smallest automobile.

SCRUBBING

Scrubbing is a factor that has been recognized as a significant safety problem since the term was defined by Klein et al. (*7*) in 1978. The phenomenon of control loss after the occurrence of edge scrubbing was described by Zimmer and Ivey (*2*) as follows:

- A vehicle is under control in a traffic lane adjacent to a pavement edge where an unpaved shoulder is lower than the pavement.
- Because of inattention, distraction, or some other reason the vehicle is allowed to move so that the right wheels are on the unpaved shoulder and just off the paved surface.
- The driver then carefully tries to steer the vehicle gently to bring the right wheels gradually back up onto the paved surface without reducing speed significantly.
- The right front wheel encounters the pavement edge at an extremely flat angle and is prevented from moving back onto the pavement. The driver further increases the steering angle to make the vehicle regain the pavement. However, the vehicle continues to scrub the pavement edge and does not respond. At this time there is equilibrium between the cornering force to the left and the edge force acting to the right, as shown in Figure 4 (1a).
- The driver continues to increase the steering input until the critical steering angle is reached and the right front wheel finally mounts the paved surface. Suddenly, in less than one wheel revolution, the pavement edge force has disappeared and the cornering force of the right front wheel may have doubled because of increases in the available friction on the pavement and the increases in the right front wheel load caused by cornering (see Figure 4, 1b).
- The vehicle yaws radically to the left, pivoting about the right rear tire, until that wheel can be dragged up onto the pavement surface. The excessive left turn and yaw continues, and it is too rapid in its development for the driver to prevent penetration into the oncoming traffic lane (Figure 4, 1c).
- A collision with oncoming vehicles or spin-out and possible vehicle roll may then occur.

An earlier research effort (*2*) developed Figure 5. This figure shows the potential of a given shape and height edge to cause a vehicle control loss. The pavement edge shapes are a relatively sharp 90-degree edge (Shape A), a rounded edge (Shape B), and a 45-degree sloped edge (Shape C). The "safety zones" that the curve for each shape goes through are defined as follows:

Safe: No matter how impaired the driver or defective the vehicle, the pavement edge will have nothing to do with a loss of control. This includes the influence of alcohol or other drugs and any other infirmity or lack of physical capability (includes subjective severity rating values 1 through 3).

Reasonably Safe: A prudent driver of a reasonably maintained vehicle would experience no significant problem in traversing the pavement edge (includes severity values 3 through 5).

Marginally Safe: A high percentage of drivers could traverse the pavement edge without significant difficulty. A small group of drivers may experience some difficulty in performing

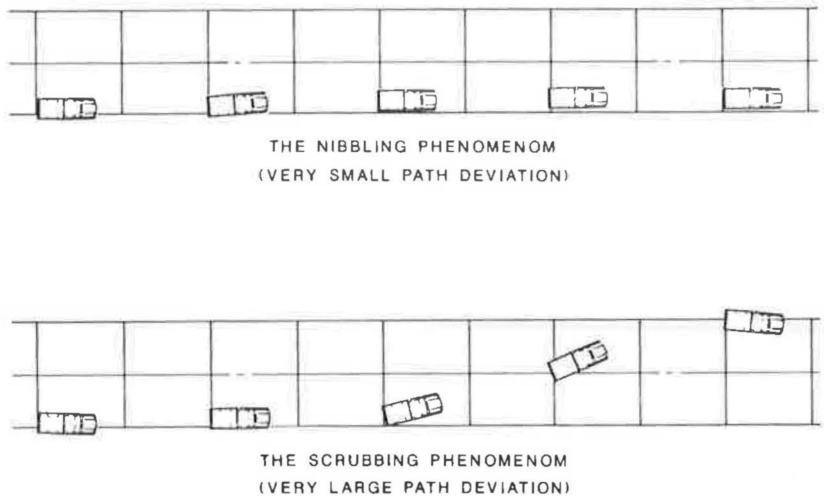


FIGURE 3 Automobile path disturbance by nibbling compared with a critical scrubbing maneuver.

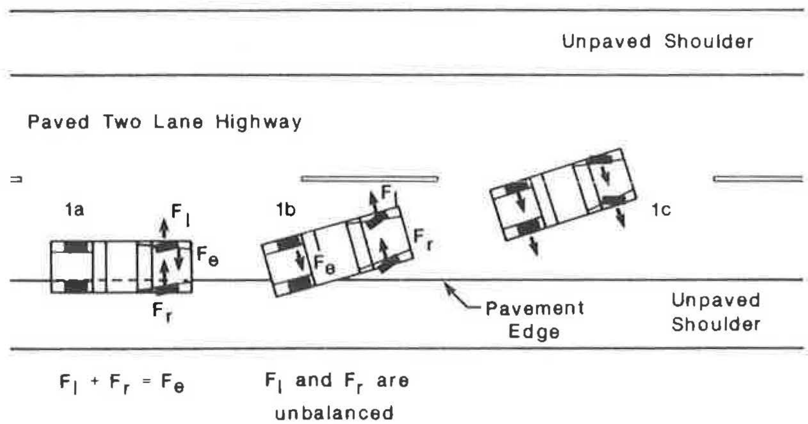


FIGURE 4 Pavement edge influence on vehicle stability.

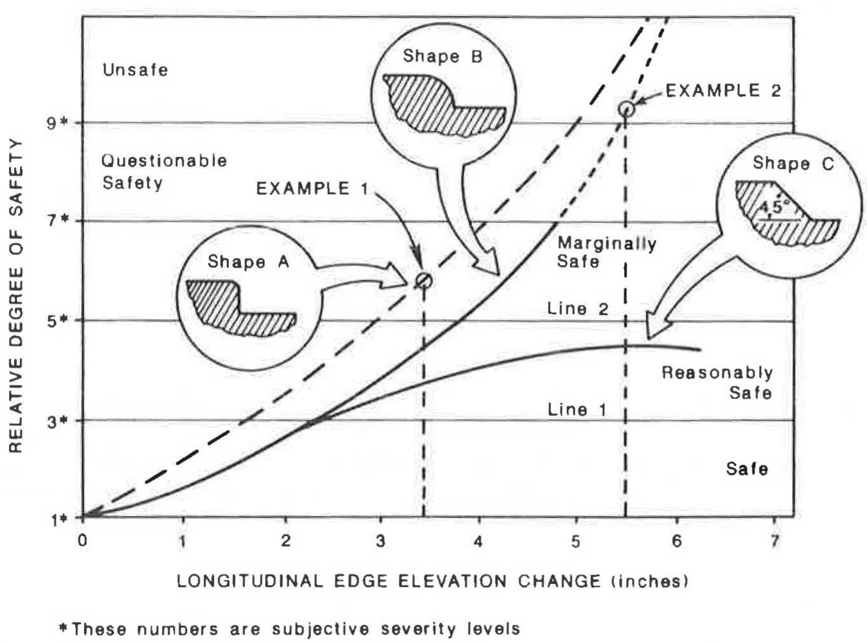


FIGURE 5 Relative degrees of safety for various edge conditions (3).

the scrubbing maneuver and remaining within the adjacent traffic lane (includes severity values 5 through 7).

Questionably Safe: A high percentage of drivers would experience significant difficulty in performing the scrubbing maneuver and remaining in the adjacent traffic lane. Full loss of control could occur under some circumstances (includes severity rating values 7 through 9).

Unsafe: Almost all drivers would experience great difficulty in returning from a pavement edge scrubbing condition. Loss of control would be likely (includes subjective severity values 9 and 10).

In interpreting the influence of different edge shapes, Ivey and Sicking (*4*) developed the concept of effective edge height and presented a theory for its determination. Figures 6 and 7 show a series of pavement edge profiles along with effective edge heights related to the cross section of a tire. This illustrates graphically how the effective edge height is dictated as the point at which the tire rubs on the edge to generate an edge-mounting force system. For other edge profiles, Table 1 gives the wheel steering angle necessary for the tire to mount the edge. These angles can be determined for any edge condition on the basis of the theory developed by Ivey (*4*) and can be used to determine the post-edge-mounting vehicle trajectory on the basis of the protocol for HVOSM developed by Sicking (*4*). It was this protocol, along with the driver response parameters developed by Olson et al. (*8*), that was used to examine the severity of several pavement edges and to develop the curves relating pavement total edge height (TEH) to pavement effective edge height (EEH) that were used in this study (Figure 8). The concept of effective edge heights was one of the most important considerations in the development of these construction zone guidelines.

DRAGGING

Dragging is an interaction with the pavement edge that can occur when edge heights are greater than the clearance underneath an automobile. In assessing this clearance value, publications of the Motor Vehicle Manufacturers Association were analyzed. The data base included 266 makes of automobiles, ranging in weight from 1,500 to 5,000 lb. Figure 9 shows clearance values of 2.4–8.0 in. About 75 percent of the automobiles analyzed had clearance values of 4.8–6.4 in. Figure 10 shows that only about 4 percent of the automobiles had clearance values less than 4.8 in. and about 15 percent had values above 6.4 in. Note that the frequencies given are not an accurate estimate of the exposure of each clearance value because the number of automobiles of each make was not included in the analysis.

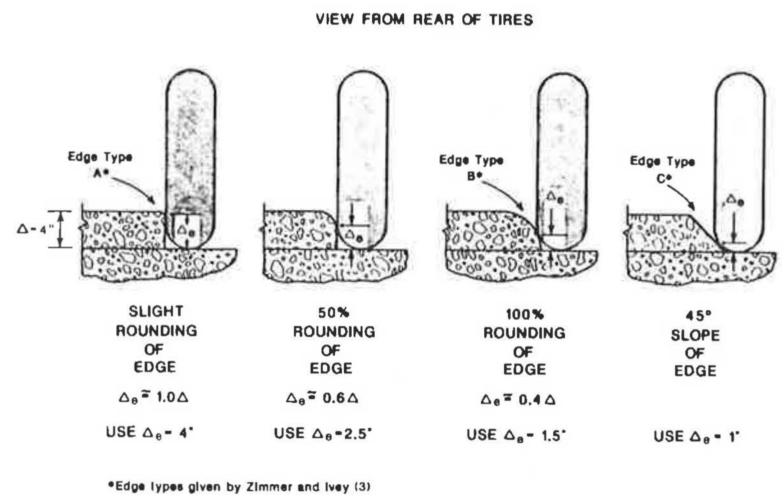


FIGURE 6 Effective edge heights for different edge shapes.

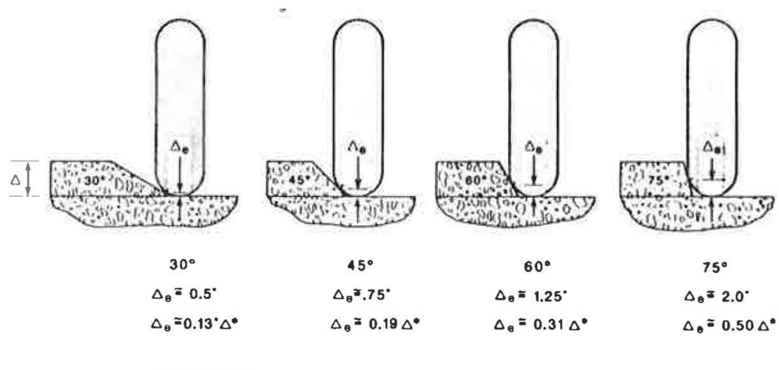
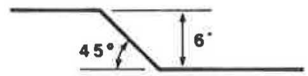
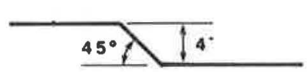
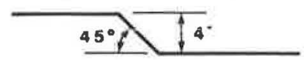
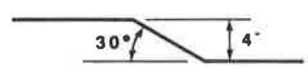
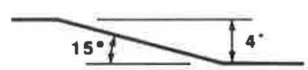


FIGURE 7 Effective edge heights for different edge slopes.

TABLE 1 PAVEMENT EDGE PROFILES, EFFECTIVE EDGE HEIGHTS, AND INITIAL STEERING ANGLES

Condition	Pavement Edge Profile	Effective Edge Height, Δ_e, inches	Initial Steer Angle, α_c degrees
1		4.0 (From Figure 6)	7.5*
2		2.5 (From Figure 6)	3.8*
3		1.5 (From Figure 6)	2.1*
4		0.75 (From Figure 7)	1.1*
5		0.75 (From Figure 7)	1.1*
6		0.75 (From Figure 7)	1.1*
7		0.75 (From Figure 7)	1.1*
8		0.50 (From Figure 7)	0.7*
9		0.20 (From Figure 7)	0.5*

*These values determined from the effective edge height and Figure 8.

It is clear that where pavement edge drops are within these ranges, a significant portion of automobiles will drag undercarriage elements on the pavement edge. This drag will generate a force proportional to the weight supported by the edge and the capacity for friction between the edge and undercarriage elements. Additional forces may be generated by edge gouging.

A friction value of 0.5 has frequently been used for contact between metal and pavement. If the various shapes of undercarriage elements and the lack of stability of a relatively sharp ACP edge are considered, however, that value might be somewhat low. In this work, to assure a conservative solution, that value will be increased by 20 percent to a level of 0.6.

Figure 11 shows two possible situations. The most common is probably the case in which the drag force is to the right of the cg if the vehicle runs off the edge at a shallow angle to the right. If the drag force is acting just inboard of the right front wheel, the maximum yaw moment is generated.

This maximum yaw moment can be calculated by the following equation:

$$M_y = F_d\left(\frac{T}{2} - m\right) = \frac{W}{2}\left(\frac{T}{T - m}\right) f \left(\frac{T}{2} - m\right)$$

where

F_d = drag force;
W = total vehicle weight;
T = track width;
m = distance inbound of the tire center where F_d acts; and
f = friction between undercarriage and edge.

The most critical case would be that in which there is no contact between the right-hand tires and the lower road (possibly shoulder) surface. If a 1,800-lb vehicle with a wheel base of 52 in. were under consideration, the value of M_y for the specific case considered would be

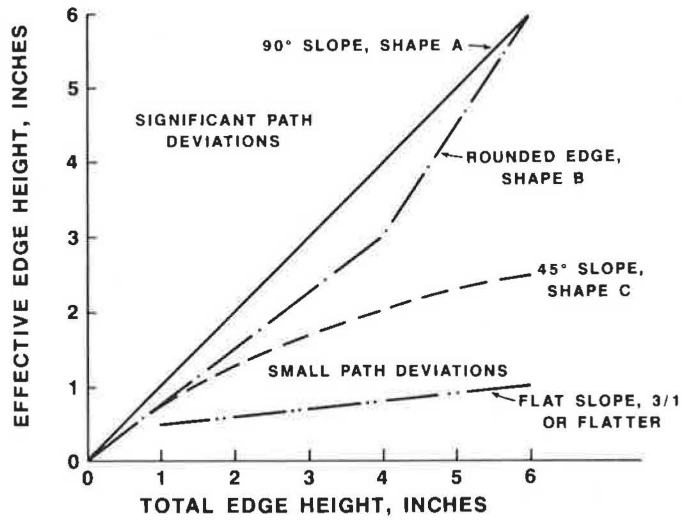


FIGURE 8 Effective edge height on the basis of edge shape and total edge height.

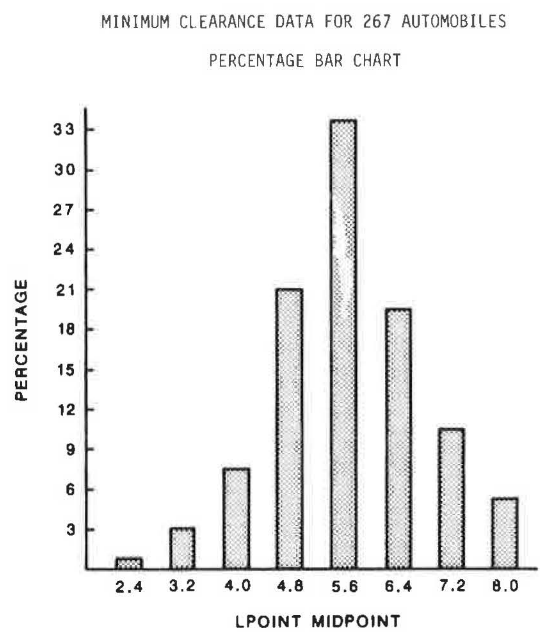


FIGURE 9 Frequency distribution of automobile clearance values.

$$M_y = \frac{1{,}800}{2}\left(\frac{52}{52-6}\right)0.6\left(\frac{52}{2}-6\right)$$

so that M_y = 12,204 in. lb or 1,017 ft lb. This is more than sufficient to cause a yaw in the vehicle that would bring the right rear tire into contact with the pavement edge and gradually move the vehicle to where the cg is coincident with the pavement edge. At that point the moment arm of the drag force becomes zero and the yaw moment becomes zero.

If the average drag force over the entire "fall off edge–drag to stop" maneuver is considered to be

$$\frac{1}{2}\left[fW + f\frac{W}{2}\left(\frac{T}{T-m}\right)\right] = f\frac{W}{2}\left[1 + \frac{1}{2}\left(\frac{52}{52-6}\right)\right]$$

$$= 0.78\,f\,W \text{ or } 0.47W$$

then the distance to stop for a vehicle moving 45 mph would be S, where

$$S = \frac{V^2}{2a} = \frac{66^2}{2\,(0.47)\,32.2} = 144 \text{ ft}$$

at a deceleration rate of 0.47 g's, or 15 ft/sec^2. This deceleration is tolerable for the occupants of the stopping vehicle but is an abrupt deceleration from the viewpoint of another driver following closely behind, because the vehicle would stop in about 4.4 sec. For this reason the drag interaction is considered a safety influence primarily because of the possibility of collision with following vehicles.

The other type of drag situation is shown by Figure 11*b*. Here the departure angle, speed, or combination of both would be sufficient to have the drag force act on a line to the left of the cg. The resultant vehicle rotation would be counterclockwise and would not be limited by edge-wheel interference, as in 11*a*. Other factors would tend to reduce the effect of the drag moment. First, as yaw progressed and the drag force moved toward the left front wheel, the load carried by the edge would decrease, going to zero as the left front wheel approached the edge brink. Second, the cornering force developed on the right rear tire, which must be in contact with the lower surface, would oppose the yaw of the vehicle. If the yaw developed quickly enough, possibly induced by major gouging into the edge, and if very high cornering forces were developed on the right-side tires, a vehicle roll might be induced. The result of these considerations is that the drag situation should be considered the primary control loss phenomenon for automobiles where edge drops of 5–20 in. occur.

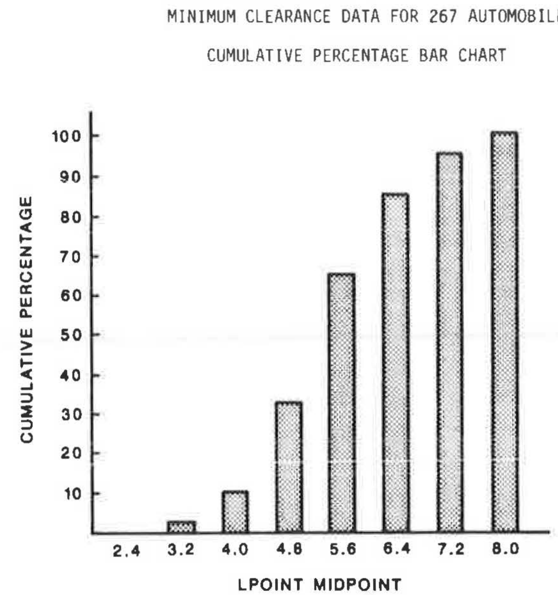


FIGURE 10 Cumulative percentage distribution of automobile clearance values.

ROLLING

Trucks

In deciding whether a truck will roll when it traverses an edge drop, several items must be considered. These are first, the fact that one side moves to a lower elevation and the center of gravity moves outboard with respect to the right-side wheels; second, the compression of right-side tires as the load shifts to the low-elevation side (causing larger axle rotation); third, the compression of right-side springs, which causes further rotation of the body (thus shifting the cg farther to the right); and finally, whether any cornering is induced by the driver's trying to steer back. If this cornering occurs, a lateral acceleration is generated. This resulting inertial force provides an additional overturning moment. Figure 12 shows a truck approaching the critical roll condition.

Ervin et al. (*9*) have shown that typical tractor-trailers have a threshold roll lateral acceleration of 0.24–0.34 gs (see Table 2). If the case of the geometrics alone is considered and if the lower threshold acceleration is chosen, the maximum edge drop for a trailer to remain upright would be given by

Max angle θ where $\sin \theta = 0.24$

$$\theta = 13.8 \text{ degrees}$$

If a trailer track width is 6 ft, then

$$\sin \theta = \frac{\Delta}{T}$$

or

$$\Delta = T \sin \theta = 6(0.24) = 1.44 \text{ ft} = 17.3 \text{ in.}$$

In this situation, the trailer is influenced by the overturning component of gravity, which is equal to the sine of the rotation angle, θ. Ross has recently shown by Phase IV simulation (*10*) that a van trailer subjected to this level of lateral acceleration would have a net body roll of about 3 degrees, including the effect of both tire deflection and suspension. If this roll is considered, then the critical edge height would be estimated by

$$\theta = 13.8 \text{ degrees} - 3 \text{ degrees} = 10.8 \text{ degrees}$$

$$\sin 10.8 \text{ degrees} = 0.187$$

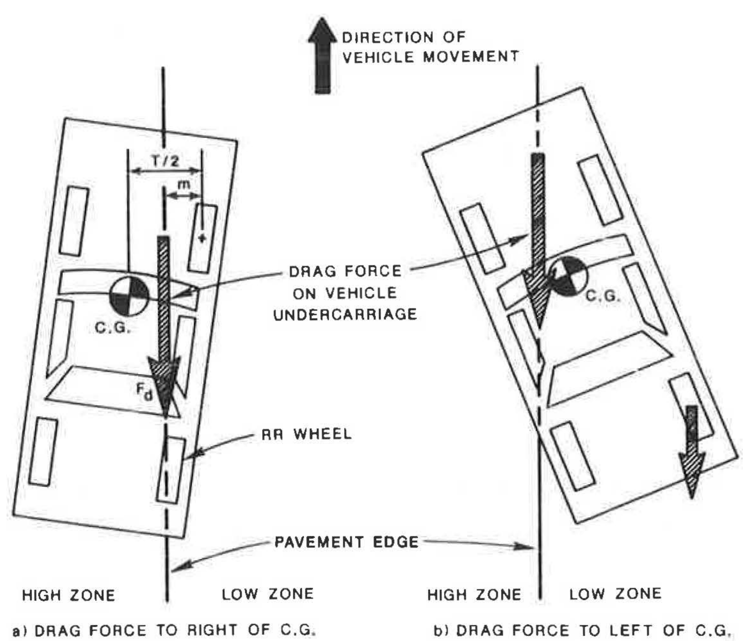


FIGURE 11 Vehicle movements in response to drag force.

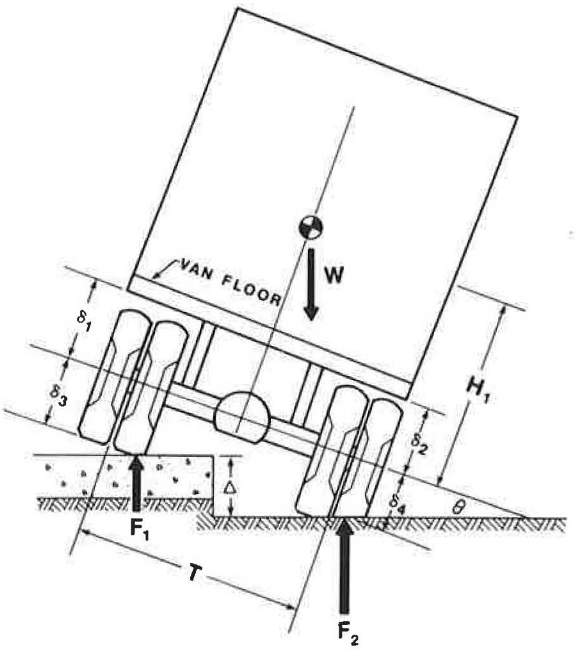


FIGURE 12 Trailer approaching critical roll condition.

$$\Delta = T \sin \theta = 6(0.187) = 1.12 \text{ ft} = 13.5 \text{ in.}$$

This would be the critical edge drop to cause rolling for a small segment of the truck population. If the driver's steering back to the left increased the roll moment, or if a soft soil condition increased the effective edge height, or finally, if load shift produced significant lateral cg movement, trucks with lower cg heights might also roll. There are other compromising conditions, including a shoulder slope away from the traffic lanes. The result of these considerations is the recognition that at least some portion of the truck fleet would be expected to roll when traversing an edge drop of only 1 ft.

Because trucks have a relatively high cg compared to track width, they represent a more critical situation when vehicle roll is considered than do passenger automobiles. Although trucks are certainly fewer in number than automobiles, significant percentages of trucks are present on major highways. These major highways generally require maintenance and reconstruction more often.

A static stability factor, $T/2H$, is often chosen to show gross differences in the stability factors of the vehicle fleet. T is the track width and H is the cg height of a given vehicle. Figure 13 shows these values for a wide spectrum of vehicles, illustrating further that the truck end of the spectrum, with $T/2H$ values shown here as low as 0.3, is the most critical.

AUTOMOBILES

The phenomenon of rolling for an automobile is different. If an edge interaction similar to that shown in Figure 12 is considered for an automobile, it may seem obvious that a higher edge

TABLE 2 LOADING DATA AND RESULTING ROLLOVER THRESHOLDS FOR EXAMPLE TRACTOR-SEMI-TRAILERS AT FULL LOAD (*9*)

CASE	CONFIGURATION	WEIGHT (lbs.) GVW	PAYLOAD CG HEIGHT (in.)	ROLLOVER THRESHOLD (G's)
A (55' TYP.)	Full Gross, Medium-Density Freight (34 lb/ft^3)	80,000	83.5	.34
B (50 in.; 50 in.; 30% of Pyld. Wt; 70% of Pyld. Wt)	"Typical" LTL Freight Load	73,000	95.0	.28
C (100 in.)	Full Gross, Full Cube, Homogeneous Freight (18.7 lb/ft^3)	80,000	105.0	.24
D (88.6 in.)	Full Gross Gasoline Tanker	80,000	88.6	.32
E	Cryogenic Tanker (He_2 and H_2)	80,000	100.0	.26

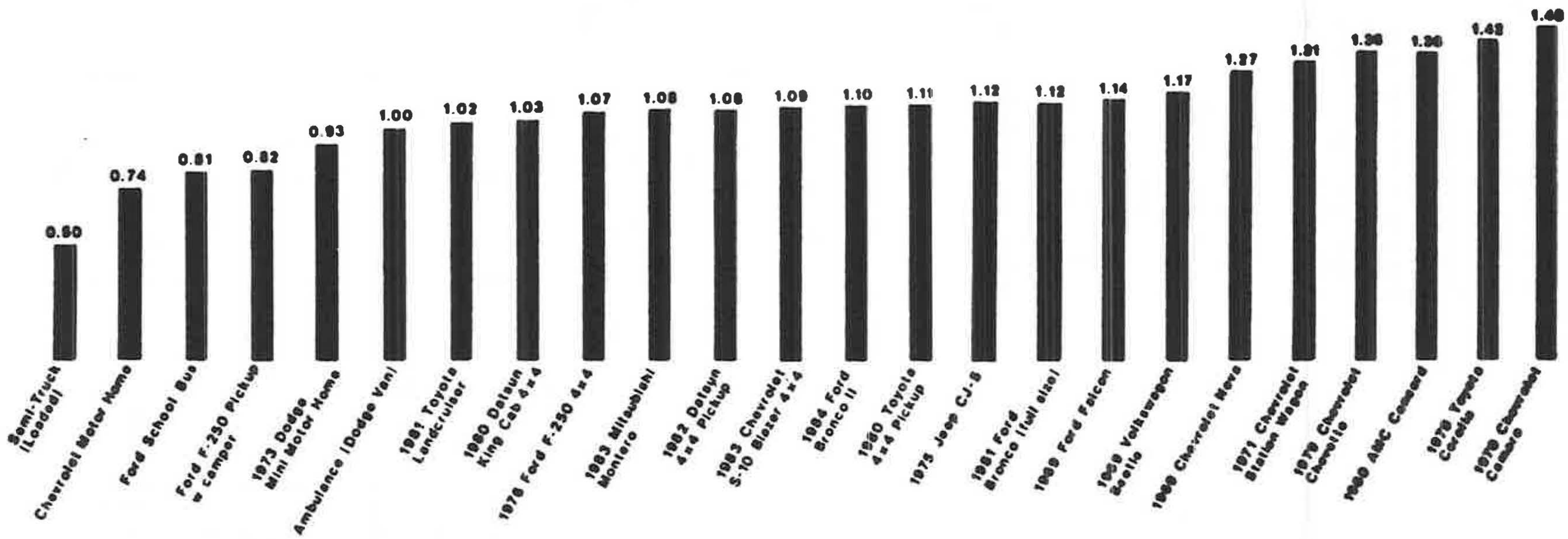


FIGURE 13 Static stability ratios for selected vehicles (*11*).

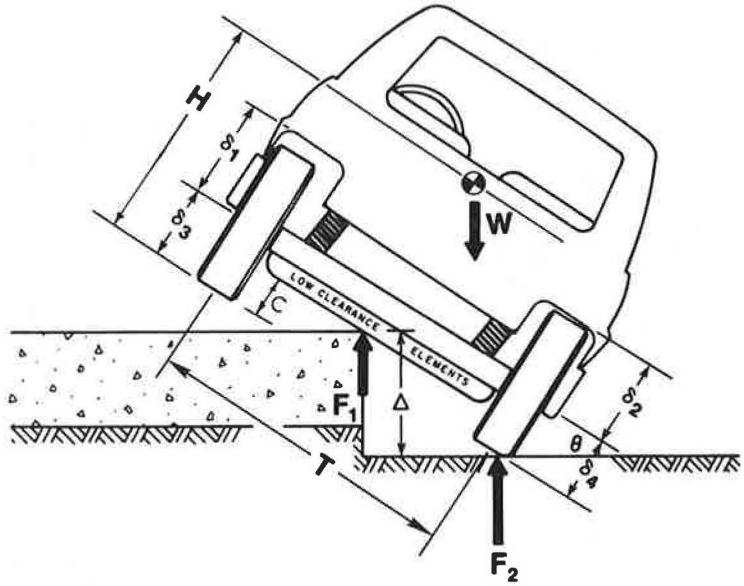


FIGURE 14 Automobile approaching critical roll condition.

is required to produce roll than the one estimated for a tractor-trailer. This may not be so obvious, however, if the interaction shown by Figure 14 is considered. Here the "low clearance elements" of the automobile are in contact with the edge of the pavement at a point about midway between the wheels.

If a $T/2H$ value of 1.2 is chosen as representative of a large part of the automobile fleet, and if the typical track width is 58 in. and a typical clearance is 5.6 in. (Figure 9), a typical cg height of

$$H = \frac{T}{2\,(1.2)} = \frac{58}{2.4} = 24 \text{ in.}$$

can be calculated. Further consideration of Figure 14 would allow the development of the following equation to predict when the line of action of W would become coincident with the line of action of $F2$, that is, when the moment preventing rollover becomes zero:

$$(H - C) \sin \theta = \left(\frac{T}{2} - c \tan \theta\right) \sin \theta$$

where

H = cg height;
T = track width;
C = ground clearance; and
θ = critical angle.

If the values suggested previously are used in this equation, θ is equal to 62 degrees. Now the force causing body roll is equal to W sin 62 degrees, or 0.88 W. This would be equivalent to a lateral acceleration of 0.88 g's. By using HVOSM, Sicking showed that a typical body roll value of a vehicle subjected to about 0.9 g's of lateral acceleration is about 10 degrees. Thus a critical angle would be about 62 degrees less 10 degrees, or 52 degrees. The following relationship can be derived by using geometric considerations:

$$\sin \theta = \frac{\Delta - (c/\cos \theta)}{(T/2) - c \tan \theta}$$

If the following values are substituted,

T = 58 in. C = 5.6 in. θ = 52 degrees

the value of Δ is found to be 26 in., roughly double the critical value of Δ for trucks.

This would be the maximum edge drop that the typical automobile could encounter without rolling, if the driver input of steering back to the left did not increase the roll moment, if a soft soil condition did not increase the effective edge height, and finally, if the right front lower corner or suspension elements did not dig into the lower surface, causing vehicle spin-out. The result of all these considerations and a selected number of HVOSM runs using a mini-compact vehicle leads to the conclusion that a small segment of the vehicle population, namely high-cg tractor-trailers, could experience rollover on edge drops as low as 1 ft, but that most vehicles would not be expected to roll until edge drops approached 2 ft, unless certain aggravating circumstances were present.

STUDY APPROACH

The authors have previously published guidelines for the maintenance of pavement edges (*2, 3*). Those guidelines dealt with a range of pavement edge heights up to 6 in. In the case of construction zones, however, the range of edge drops can be much larger. In a recent study of the use of barriers in construction zones, five sites were observed at which the drop was 10–20 ft and one at which the drop was 80 ft. In this work the small values are again considered, but the scope is increased to include much larger edge drops. There is another reason that recommendations for construction zones might be significantly different from recommendations for maintenance. In construction zones the time of exposure may be small, the existence of the edge or drop can be predicted, and appropriate warning devices can be placed. In contrast, on completed highways the knowledge of small edge drops must be based on surveillance, and the maintenance operations, when required, must be funded and scheduled. Finally, if surveillance does not detect the condition, the exposure of traffic to the situation may be long term, or even until an accident brings it to the attention of the highway agency.

The approach that was taken here (*11*) is that the degree of exposure to a certain condition is estimated, the result of that condition on vehicles that encounter it is predicted, the severity index and cost of specific types of accidents are estimated, and the costs of warning, delineation, edge treatment, and barriers determined. As a result of these estimates, predictions, and determinations, a benefit/cost ratio for various situations can be determined and used as a basis for treatment guidelines. These cost estimates are developed in detail elsewhere (*11*), including a matrix of predicted accident costs for a wide range of traffic and pavement edge conditions.

BENEFIT-COST FORMULATION

Accident Costs

The determination of accident costs requires the estimate of the number of accidents that are expected to occur and the severity of those accidents. The probabilities and severities used were developed by Ivey et al. (*11*). This work considered these five categories: (a) nibbling, (b) scrubbing, (c) scrubbing-drag, (d) drag-roll, and (e) rolling. Table 3, from the ABC-RS model by Sicking and Ross (*12*) was used to relate accident costs to accident severity index (SI).

By using the predictions of hazardous event probability and severity developed by Ivey et al. (*11*) and the ABC-RS accident costs, the accident costs due to edges and dropoffs in construction zones were predicted for the situations given in Table 4. A detailed presentation of accident costs was made for 524 combinations of these conditions. These results reveal that predicted accident costs cover an extremely wide range, varying from nothing for the 1 in. edge to over $100,000 per month per 1,000 ft of construction zone for 40-in. edge drops and high values of average daily traffic (ADT). Table 5 gives some of these values for the most critical situation investigated, the four-lane undivided highway.

TABLE 3 ACCIDENT COSTS FOR VARIOUS SEVERITY INDEX LEVELS

Severity Index	Accident Costs (1986 dollars)
0	2,120
1	4,290
2	6,450
3	8,620
4	18,230
5	49,450
6	103,020
7	238,500
8	463,340
9	604,820
10	723,970

Barrier Costs

After the costs of certain countermeasures are developed, they may be used to determine whether the countermeasures could be justified on a benefit-cost basis. One thing is apparent: a positive barrier, such as a precast concrete barrier (PCB), would not be economically justified to protect against edges unless ADT values are high (usually above 50,000) and edge drops are close by and deep.

TABLE 4 CONDITIONS USED TO PREDICT ACCIDENT COST

Highway Type	ADT	Edge Height (in.)	Lateral Position (ft)
Two-lane undivided	1,000 to 30,000	1 to 24	0 to 20
Four-lane, undivided	10,000 to 200,000	1 to 24	0 to 30
Six-lane, undivided	25,000 to 225,000	1 to 24	0 to 20

TABLE 5 REPRESENTATIVE ACCIDENT COSTS FOR 1,000 FT OF A SPECIFIED EDGE CONDITION IN A CONSTRUCTION ZONE

Lateral Clearance (ft)	Dropoff Height (in.)	Accident Cost[a] ($/month/1,000 ft)
ADT = 10,000		
0	5	30
5	5	28
20	5	13
0	24	639
5	24	587
20	24	263
ADT = 100,000		
0	5	442
5	5	402
20	5	182
0	24	9,302
5	24	8,539
20	24	3,833
ADT = 200,000		
0	5	1,493
5	5	1,370
20	5	615
0	24	31,498
5	24	28,851
20	24	12,949

[a]Dollars per month per 1,000 feet of edge condition.

By using the data from Table 4, Figures 15–17 were developed. These three figures show zones where a positive barrier is cost effective if the cost of the barrier is $2.00, $5.00, or $10.00/ft/month. Discussions with contractors, barrier suppliers, and highway engineers across the United States indicate a wide range in the cost of concrete barriers for construction zones. New barriers may cost from 25 to 30 dollars per foot, but this cost is not indicative of the cost in construction zones. If a highway department buys a portable barrier and uses it for several years, the ultimate cost per month of use may be only a fraction of the original cost. Further, if the concrete barrier is supplied by the contractor for use during construction and then ultimately installed as permanent barrier, the costs of temporary use are difficult to determine. It seems apparent, however, that they would again be only a fraction of the permanent barrier cost. In some states (Indiana, for example), concrete barriers are leased from a barrier precaster. The cost of these barriers is highly dependent on the distance from the storage yard to the job site but may approach as little as $2.00/ft/month on some jobs.

Figures 15–17, which were not used in the final section of the guidelines, are presented here as additional cost effectiveness tests that may be used in conditions where the guidelines show that positive barriers are optional. In terms of the edge height and the lateral distance from the nearest traffic lane to that edge, these curves define boundaries of cost effectiveness. All combinations of edge height and lateral distance that plot above a given curve would be expected to be cost effective; that is, the savings in accident costs would be less than or equal to the cost of providing a barrier. Obviously, the position of these curves is highly dependent on the cost of providing the barrier. For this reason, curves are provided at the barrier cost levels of $2.00, $5.00, and $10.00/ft/month.

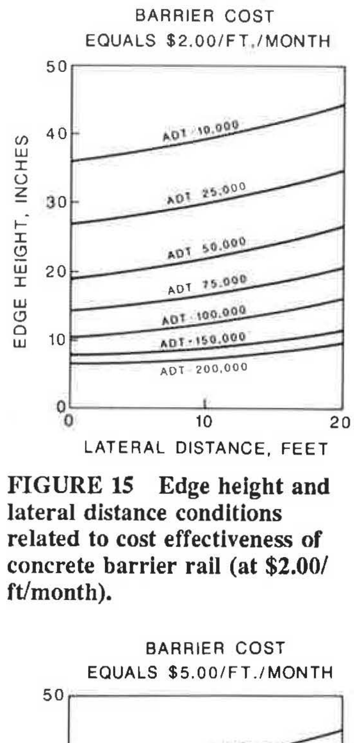


FIGURE 15 Edge height and lateral distance conditions related to cost effectiveness of concrete barrier rail (at $2.00/ft/month).

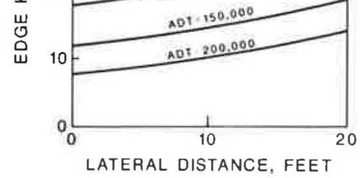


FIGURE 16 Edge height and lateral distance conditions related to cost effectiveness of concrete barrier rail (at $5.00/ft/month).

These curves are considered to be conservatively placed in that the accident costs of colliding with the barrier instead of interacting with the edge are not considered. Since the edge condition is usually only one of the factors considered when the decision to provide or not provide a barrier is reached, greater levels of sophistication in determining the cost-effective zones were not considered appropriate.

DEVELOPMENT OF GUIDELINES

In developing guidelines for the use of warning and protective devices in construction zones, the authors relied on the understanding of potential hazards of certain types of edges, as described in the first part of this paper, on the warning and protective devices considered practical and effective, and on the costs of positive barriers, such as portable concrete barriers. In the case of warning devices, every effort was made to be conservative (i.e., to provide, if anything, more than adequate guidance and warning). In the case of justifying positive barriers, simplifying and conservative assumptions were made in the guidelines suggesting use (i.e., barriers were recommended even in cases where cost effectiveness is marginal). It was also considered necessary to build flexibility into these guidelines so that the special cases could be treated in special ways.

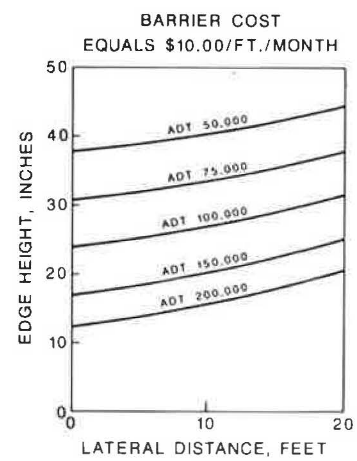


FIGURE 17 Edge height and lateral distance conditions related to cost effectiveness of concrete barrier rail (at $10.00/ft/month).

Because the authors have attempted to present their work succinctly, they have only been able to summarize the research that went into the guidelines. A more complete understanding of the factors that contributed to the final form of the guidelines may be gained by consulting previous work by the authors and their colleagues (*13–17*).

The resulting guidelines are given in the Appendix (after the Discussion and Authors' Closure). They are presented here for the consideration of states and other governmental agencies. These guidelines have been reviewed by the State Department of Highways and Public Transportation and the Federal Highway Administration, and numerous revisions were made before the guidelines were accepted. Many appropriate suggestions were made by state and federal reviewers, resulting in guidelines that are believed to be both practical and effective. The guidelines were provided to all Texas districts on November 30, 1987.

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DISCUSSION

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The need for tested and proven standards and guidelines for treating pavement edge dropoffs is critical. Edge dropoffs in construction work zones and on existing highways have become a recognized cause of accidents and have become an increasing cause of tort litigation in many states. Unfortunately, the treatments proposed in this paper and adopted by the Texas Department of Highways and Public Transportation are neither tested nor proven. The assumptions used with regard to the types of vehicles, vehicle response, and the expected actions of drivers who leave the road and encounter an edge dropoff raise substantial questions.

MOTORCYCLES

The paper does not discuss the effects of pavement edge dropoffs on the possibility of motorcycle instability when a rider encounters a vertical edge dropoff of less than 1.5 in. The paper states that vertical edge dropoffs between lanes of travel should not exceed 1.5 in. Author Ivey, in response to a question at the 1988 TRB meeting presentation of this paper, responded that edge heights of less than 1 in. could cause instability in a motorcycle. In fact, he said that any vertical edge can be a problem for a motorcycle. Clearly, the guidelines do not consider this hazard adequately and thus are flawed.

HAZARDS TO AUTOMOBILE OCCUPANTS

The authors assume only one hazard to occupants of an automobile that drops one or two wheels off the edge of a dropoff of 5–20 in. in depth and slides along the edge with its underside in contact with the pavement. This hazard is that the automobile may be rear-ended by another vehicle because of sudden slowing. The authors rule it "improbable" that either the drag on the automobile's underside or cornering forces on the tires could cause a rollover or other loss of control or spinout. This assumption is unsupported and could prove dangerous. Another real hazard to occupants of automobiles that should be considered is the presence of fixed objects along the pavement edge, such as bridge abutments or construction equipment and materials. Furthermore, construction workers can be struck by an out-of-control vehicle that slides along an edge dropoff or that may completely leave the pavement in an uncontrolled manner. Loss of vehicle control can be a complex event. No research, which might have included field testimony and observations, was presented to support assumptions made about the events being analyzed in this paper. The assumptions are far too simplistic and limiting to be accepted without more in-depth research.

TRUCKS

The parameters used to analyze the potential hazard to trucks from edge dropoffs are even more limited than those used for cars. The authors discount a number of events that can occur when a truck drops its wheels off the pavement. While the authors do discuss the high center of gravity characteristics of trucks, they discount the effects of shifting cargo. There is no mention of the worst case cargo—liquid. The surge of liquid loads, particularly partial loads, can overturn a truck on a pavement surface. When this same truck encounters a dropoff of 1, 2, or 3 ft, the hazard is magnified. Liquid loads, in addition to causing vehicle instability, can be composed of hazardous commodities that can injure large populations. (It should be noted that hazardous cargos are quite prevalent in many areas of Texas, where petrochemical plants are located.)

The authors also discount the sideward acceleration forces on the truck due to the driver's attempts to steer back onto the pavement. It is unrealistic to expect that drivers of cars or trucks will continue to steer their vehicles parallel to the pavement's edge after the wheels have dropped. Basic driver instinct is to return to the pavement. Any analysis that assumes otherwise makes an erroneous and dangerous assumption. The authors conclude that

> a small segment of the vehicle population, namely high-cg tractor-trailers, could experience rollover on edge drops as low as 1 ft, but that most vehicles [i.e., mini-compact automobiles] would not be expected to roll until edge drops approached 2 ft, unless certain aggravating circumstances were present.

There is no quantification of the "small segment," nor of the consequences to vehicle occupants. Such unsupported assumptions, which ignore "segments" of drivers, do not constitute an acceptable method of risk analysis in the vital area of public safety.

COST ANALYSIS

The authors discuss a benefit/cost approach to selecting treatments for edge dropoffs in construction zones. Unfortunately, the only basis for the costs used is a paper by one of the authors, and no mention is made of its availability. Thus an important aspect of the paper cannot be analyzed. As previously discussed, there is no discussion of accidents involving trucks carrying hazardous materials and the risk to people or public facilities (e.g., public water supplies). Any benefit/cost analysis is incomplete without an analysis of costs involved in these types of accidents and the potential reduction of cost and risk resulting from improved treatments.

DISCUSSION OF TREATMENT

In spite of the shortcomings of the analytical approach used by the authors, the guidelines are an improvement over previous guidelines recommended for maintenance of highways by Ivey and other researchers at TTI. Earlier TTI maintenance guidelines, published in 1982, recommended that a 6-in. dropoff have a slope of 45 degrees or 1 to 1. The new guidelines for construction provide that if an edge dropoff is more than 2 in. in depth (called "Edge Condition I"), a slope should be constructed outward from the pavement surface of compacted fill material at a 3-to-1 or flatter slope. If the sloped fill material is not added and the edge dropoff is within 30 ft of the travel lane's edge, then traffic control devices must be installed.

Under Edge Condition II, in which an edge is 5–24 in. or more in depth, the recommendation is to use a slope of 3 to 1 or to use signs, vertical panels, and barrels with steady burn lights if the drop is within 20 ft of the lane's edge. This provision allowing use of traffic control devices alone for a 24-in. edge dropoff is not adequate. Traffic control devices at the edge of a traveled lane, particularly where traffic is heavy, are often knocked off the road. As a result, the edge dropoff is exposed without warning. In addition, traffic control devices do not provide any shielding of the dropoff to contain or redirect an errant vehicle. Urban areas where high traffic volumes are common and trucks carry highly hazardous cargo are precisely where precautions must be exercised. Edge Condition II treatment is an apparent result of the flawed analytical approach used for this report. Such provisions encourage road designers and contractors to create edge dropoff hazards that may otherwise be prevented.

CONCLUSION

While the guidelines are an improvement over previous methods used by the Texas Department of Highways and Public Transportation for edge dropoffs up to 5 in., improvements are still needed in the adopted policy for edge dropoffs of 5–24 in. This paper, in supporting the guidelines, advocates an approach limited by unrealistic assumptions. These faulty assumptions could mislead engineers who follow the analytical example provided and thus create a more dangerous condition for the road user and construction worker than can be justified.

The lack of vehicular testing of the guidelines is a major shortcoming of the research. The end result of adopting these guidelines will surely be unnecessary accidents, injuries, and litigation.

AUTHORS' CLOSURE

The authors are pleased that Anderson has taken the time to discuss this paper. He is certainly sincerely concerned about safety but has apparently missed certain key statements and references that are important to developing a thorough understanding of these guidelines.

One of the basic problems that a reviewer has in understanding a paper that has been condensed to the degree necessitated by TRB publication and presentation requirements is the authors' reliance on extensive prior research. Unless the reviewer is already familiar with a dozen or more references or takes the trouble to read and understand them, he is operating from a very different perspective than that of the writers. It takes time and space for a writer to discuss these references. In this case that luxury is simply not available under TRB length requirements. The present TRB paper is a condensation of a report of over 100 pages that goes into much greater detail. In an effort to relieve Anderson's concerns, specific paragraphs will be discussed under the heading of his discussion.

Concerning the statement that "the treatments . . . are neither tested nor proven," the delineation and barrier devices have been in use for over 15 years and are qualified under state, AASHTO, and FHWA guidelines, policies, and standards.

Experience has proven that the recommended devices are quite effective in construction zone applications. No set of guidelines for a specific purpose is ever *proven* until it has been successfully *used* and *evaluated.* The State Department of Highways and Public Transportation (SDHPT) has taken the initiative to bring together *all* that is known about this particular problem as the basis of these guidelines. In so doing, we believe that SDHPT has acted in an extremely progressive and responsible manner.

MOTORCYCLES

Anderson is concerned about the fact that motorcycles may sometimes prove unstable if brought into contact with very small edges, the type of edge sometimes produced by a single lift of asphaltic concrete. Motorcycles are probably more difficult to control when in contact with any type of surface discontinuity, but to reach the conclusion that their omission in these guidelines is inappropriate is a mistake for several reasons. To this date there have been at least eight papers written on the pavement edge phenomenon. None have considered motorcycles. The reason for that is twofold. First, there is no precedent for use of a motorcycle as a "design vehicle" for highways. Consider the examples of median barriers, crash cushions, curbs, guardrails, breakaway structures, vertical curves, and horizontal curves. In fact, if the index of the 1984 *Policy on Geometric Design of Highways and Streets* is consulted, the word motorcycle will not be found (*1*). The same is true of the second edition of the *Transportation and Traffic Engineering Handbook* (2). Neither will certain types of large trucks be found in these documents. The highways are designed for automobiles and common types of trucks. It has never been considered practical to cover the entire spectrum of vehicles that may be found on a highway. The usual MUTCD signing of a construction zone should be enough to put the motorcycle rider on notice that he is moving into an area that may put unique requirements on him to drive with care. It was not considered appropriate to post warnings for extremely small sloped edges that have no significant influence on automobiles. The guidelines *do* suggest that the sharper edges (50–90 degrees, Edge Condition III) should be treated with warning signs (CW 21-13 or 14) and delineation (vertical panels) even when the edge height is less than 2 in. (see Figure A2, Edge Condition III of the guidelines).

HAZARDS TO CAR OCCUPANTS

These are guidelines for protection against the hazard caused by edges *only*. Note this sentence in the guidelines: "It does not consider the hazards of other conditions in the construction zones, such as heavy machines or the hazards to construction workers." Anderson is concerned about the guidelines' not doing something that it was never their purpose to accomplish. Use of the HVOSM model and careful assessment of the literature on vehicle stability strongly support the improbability of a vehicle roll as a result of underside drag.

TRUCKS

We are puzzled by Anderson's statement here. It is emphasized in the report that high-cg tractor semi-trailers are nearly twice as sensitive to rolling as are automobiles. The concerns he expresses are *all* dealt with in two sentences of the paper:

> This would be the critical edge drop to cause rolling for a small segment of the truck population. If the driver's steering back to the left increased the roll moment, or if a soft soil condition increased the effective edge height, or finally, if load shift produced significant lateral cg movement, trucks with lower cg heights might also roll. There are other compromising conditions, including a shoulder slope away from the traffic lanes.

and finally, on the subject of frequency:

> Although trucks are certainly fewer in number than automobiles, significant percentages of trucks are present on major highways. These major highways generally require maintenance and reconstruction more often.

It was recognized that no detailed investigations of truck instability problems had been made when this work was completed. In the time since presentation, however, such an investigation was made. The following quote is significant concerning the allegations of "simplistic assumptions" (*3*):

> Finally, it is concluded that the guidelines recommended for edge and shoulder maintenance in 1983 . . . and the recent guidelines for treatment of edges in construction zones . . . are as appropriate for TST's as they are for the vehicle which was then given primary consideration, the automobile.

COST ANALYSIS

The need to have this paper comply with TRB length guidelines prompted the removal of 37 pages of benefit-cost analysis. That analysis is in a report that is available from both SDHPT and TTI (*4*).

DISCUSSION OF TREATMENT

Perhaps is has been difficult for Anderson to recall the previous maintenance guidelines. A 6-in. dropoff was ***never*** recommended to have a slope of 45 degrees. The following was stated (*5*):

> If shape C (45-degree edge) can be constructed, either during original construction or as a maintenance activity, the need for edge maintenance could be significantly reduced. Shape C may also have a significant advantage in resisting pavement edge deterioration.

and (*5*)

> Pavement edge heights more than 5 in. in height can interfere with the underneath clearance and thus create safety problems for small automobiles.

Furthermore, Anderson has misinterpreted recommendations of the current guidelines. Edge Condition I is the result of the construction of an edge fill. The edge fill is not required; it is simply an option that will allow the use of minimum signing and delineation.

Here Anderson seems to be saying that traffic control devices are not adequate if they are not maintained. We would agree only with that part of his ideas, obvious as it may seem. It is true that some heavy trucks may roll when erroneously driven across such a edge. This has been discussed in detail in the section on trucks. It is not true that positive barriers are necessarily warranted in these conditions by high-cg trucks carrying hazardous materials. In fact, there are *no temporary construction barriers* commonly available that were designed for trucks. They are *all* designed for automobiles (*6, 7*). If by the term "flawed analytical approach," Anderson means that the benefit-cost analysis is imperfect, we certainly acknowledge that fact. As engineers, we have used the information available to arrive at a reasonable analysis. Where information is unavailable or is known to have certain limitations (is that what "flawed" means?), we have used well-considered engineering estimates. To do otherwise would have been to acknowledge that the job was impossible. It was not.

CONCLUSION

These methods are not "improvements" over previous methods used by SDHPT. These methods are for construction zones. The previous methods are for shoulder maintenance. The two have much time the same basis, however, and are in fact quite consistent.

It is *impossible* to do "vehicular testing of the guidelines." It is *possible* to test the *guidelines* but ultimately only through application.

The end result of adopting these guidelines "will surely be" a reduction of accidents and injuries. Considering that only a few highway agencies have adopted any type of comprehensive plan to deal with pavement edges, SDHPT must be considered a pioneer in this area.

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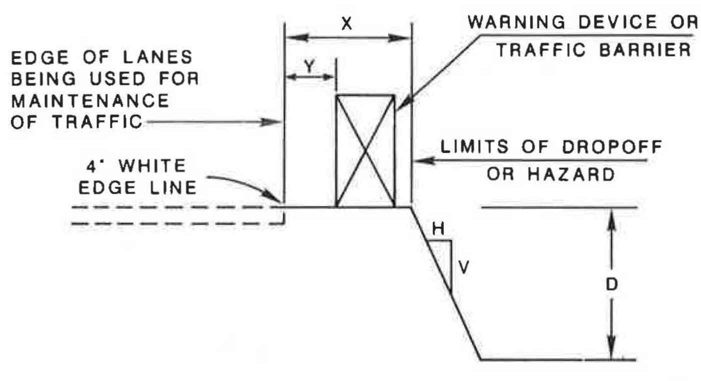
APPENDIX: GUIDELINES FOR WARNING AND PROTECTIVE DEVICES FOR PAVEMENT DROPOFFS

These guidelines are applicable to construction work where continuous pavement edges or dropoffs exist parallel and adjacent to a lane used for traffic.



NOTE: Minimum Lane Width = 10 ft.
Desirable Lane Width = 11–12 ft.

1. Distance "X" (lateral clearance) is to be the maximum practical under job conditions.
2. Distance "Y" is to be a minimum of 2 feet if feasible.
3. Warning devices must not encroach on lanes required for maintenance of traffic at any time.
4. When optional devices are specified, the contractor may select the type to be used. If distance "X" must be less than 3 feet use of positive barrier (e.g., concrete traffic barrier, metal beam guard fence, barrel mounted guard fence) may not be feasible. If in this case a positive barrier is needed according to Figure 4, considerations should be given to moving the lane of travel laterally to provide the needed space or to providing an edge slope such as Condition I.

FIGURE A1 Definition of terms.

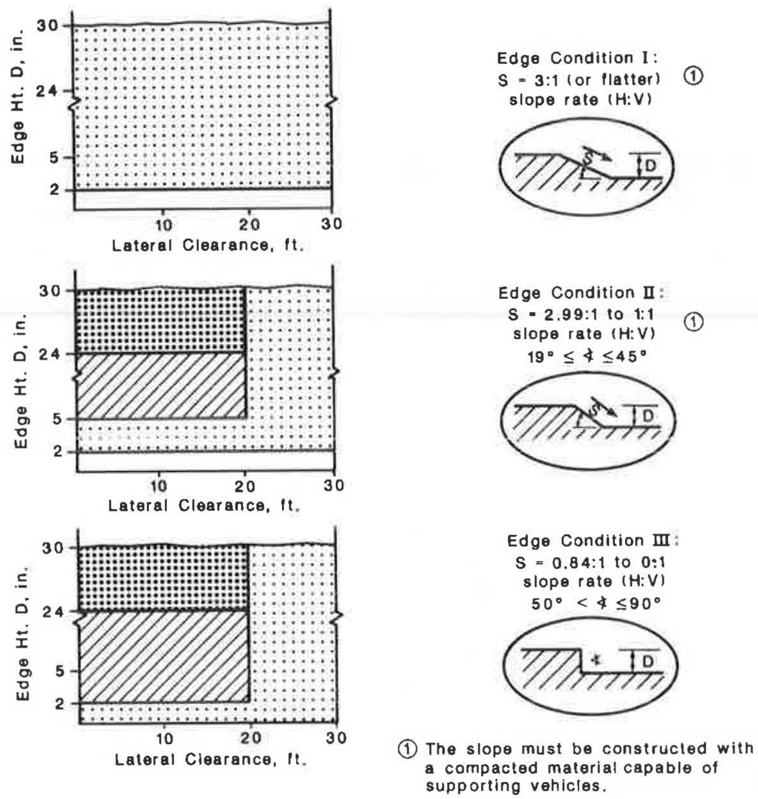


FIGURE A2 Definition of treatment zones for various edge conditions.

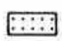
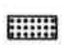
Zone	Usual Treatment
	CW 21-13 or CW 21-14 signs plus vertical panels (15).
	CW 21-13 or CW 21-14 signs plus drums with steady burn lights. Where restricted space precludes the use of drums, use vertical panels. An edge fill may be provided to change the edge slope to that of the preferable Edge Condition I (15, 16).
	Check indications (Figure 4) for positive barrier. Where positive barrier is not indicated, the treatment shown above for zone may be used after consideration of all other applicable factors.

FIGURE A3 Treatment selection guidelines (to be used with Figure A2) (*15, 16*).

The type of warning device and/or protective barrier selected depends upon several factors including traffic volume, lateral distance from the edge of travel lane to hazardous condition, depth of dropoff, duration of the hazardous condition, and shape of the edge or slope of the dropoff.

In urban areas where speeds of 30 mph or less can be predicted for traffic in a particular construction zone, these lower speeds may indicate less stringent requirements for signing, delineation, and the use of barriers. Still, less stringent requirements are not recommended for sharp 90-degree edges from 2 to 6 in. in height or for edges over 18 in. in height if located within a lateral offset distance of six feet or less from a traffic lane.

These guidelines are premised on a duration period of the edge condition of overnight or longer. Considerations of practicality will dictate against the use of positive barriers for very short periods of time. Figure A1 shows pertinent dimensions and terms, and Figure A2 gives a definition of the treatment zones for various edge conditions. Figure A3 gives the suggested treatments for each of these zones. Under certain circumstances the suggested treatment indicated by Figures A2 and A3 may not be practical and a unique treatment should be devised and its proper function substantiated. Figure A4 depicts traffic volume and dropoff offset conditions that justify positive barriers to shield hazardous edges.

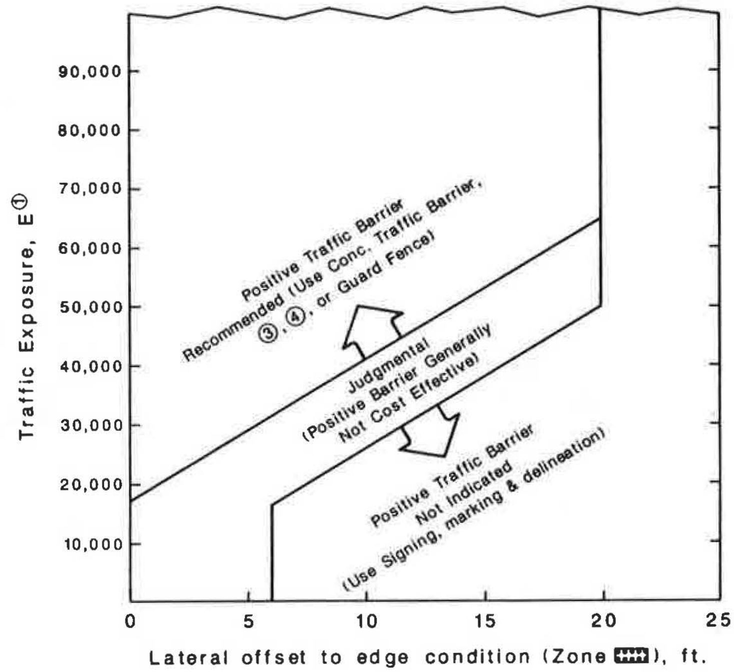
Several factors are important in applying Figure A2 and selecting an appropriate treatment:

Edge Condition I

Most vehicles are able to traverse an edge condition with a slope rate of 3:1 or flatter.

Edge Condition II

Most vehicles are able to traverse this edge as long as *D* does not exceed 5 in. Undercarriage drag on most automobiles will



Notes:

① E = ADT x T

Where ADT is that portion of the average daily traffic volume traveling within 20 feet (generally two adjacent lanes) of the edge dropoff condition; and, T is the duration time in years of the dropoff condition.

② Primarily applicable to high speed conditions only.

③ Barrel Mounted Guard Fence may be used in lieu of CTB where speeds of 45 mph or less and impacting angles of 15 degrees or less are anticipated.

④ An approved end treatment should be provided for any positive barrier end located within a lateral offset of 20′ from the edge of the travel lane.

FIGURE A4 Conditions indicating use of positive barrier.

occur as *D* exceeds 6 in. As *D* exceeds 24 in., the possibility of rollover will be greater for most vehicles.

Edge Condition III

Edges where *D* is greater than 2 in. can present a problem to drivers if not properly treated. In the zone where *D* is 2 to 24 in., different types of vehicles have safety-related problems at different edge heights. Automobiles have more difficulty in the 2- to 5-in. zone. Trucks, particularly those with high loads, have more difficulty in the 5- to 24-in. zone. As *D* exceeds 24 in., the possibilities of rollover will be greater for most vehicles.

Limitations of Figure A4

This figure is an effort to provide a practical approach to the use of positive barriers for the protection of vehicle passengers from the hazards of pavement dropoffs. It does not consider the hazards of other conditions in the construction zones, such as heavy machines or the hazards to construction workers. These other factors may make the choice of a positive barrier appropriate even when the edge condition would not justify the barrier.

Unusual Conditions

Under certain circumstances a higher type treatment is appropriate for the pertinent conditions. For example, a dropoff located along the outside of a sharp horizontal curve is more vulnerable, and a treatment exceeding that indicated for usual conditions may be appropriate. Although most construction zones may be signed for a slower speed, a higher type treatment may be appropriate if the posted speed through the construction zone exceeds 50 miles per hour.

Edges Across Travel Lane

An Edge Condition II or III that traffic is expected to cross during construction should not have a height (value of *D*) greater than 1.5 in. Any height greater than that but not to exceed 3 in. should be treated using an ACP wedge to produce Edge Condition I where the slope is 3:1 or flatter. This

6B-28.4 Uneven Lanes Sign (CW21-14) (15)

The UNEVEN LANES sign is intended to be used during resurfacing operations which create a difference in elevation between adjacent lanes greater than one (1) inch. The image may be mirrored to indicate the proper elevations of the lane.

CW21-9
30" x 30"

CW21-13
36" x 36"
24" x 18"

6B-28.3 Shoulder Drop-Off Sign (CW21-13) (15)

The SHOULDER DROP-OFF sign is intended for use when a shoulder drop-off exceeds three (3) inches in height and is not protected by a positive protective barrier. The image may be mirrored to show a drop-off on the left.

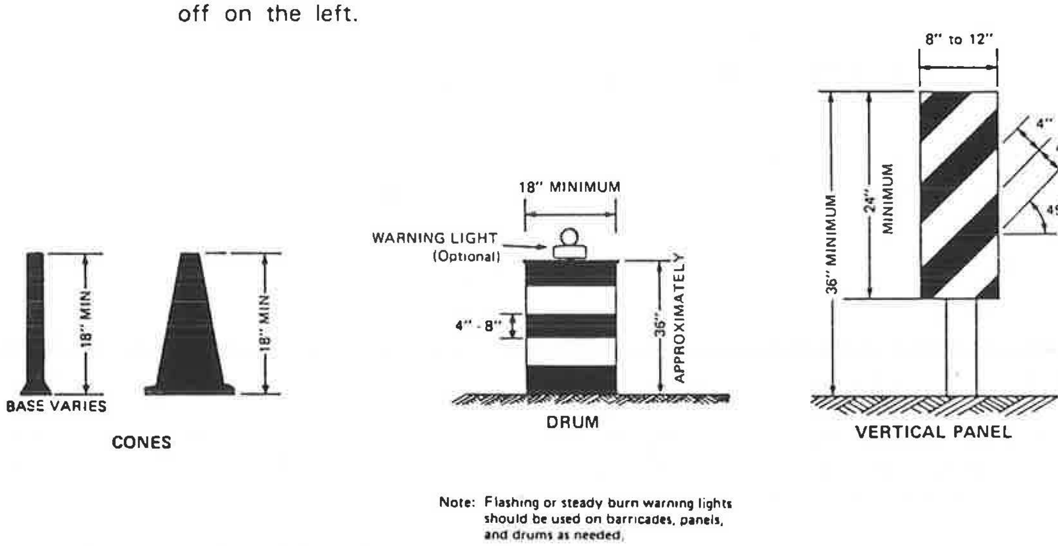


FIGURE A5 Definition of warning devices (*15, 16*).

treatment should be maintained as long as traffic is traversing the edge.

Each dropoff situation should be individually analyzed, taking into account cross sectional features, traffic volume, posted speed, and the practicality of treatment options. Figures A2, A3, and A4 are not a rigid standard or policy; rather, they are a guide that is based on certain, but not all, factors that should be considered.

Publication of this paper sponsored by Committee on Traffic Safety in Maintenance and Construction Operations.

Field Evaluation of Highway Safety Hardware Maintenance Guidelines

BENJAMIN H. COTTRELL, JR.

The objective of this study was to use field tests to evaluate a procedure developed for the Federal Highway Administration for determining the frequencies at which highway safety hardware needs to be inspected and repaired. The selection of the frequencies that were determined was based on the accident history of the safety hardware and the level of service to be provided, which has its basis in the probability of completing the inspection and repair before a subsequent accident. It is concluded that the procedure is a useful method for determining highway safety hardware maintenance guidelines. Some problems are noted, and suggestions are made to resolve them.

In Virginia during 1984 there were 3,511 fixed-object accidents (1,726 on Interstate roads and 1,785 on primary roads) in which vehicles struck highway safety hardware, such as guardrails, sign and signal supports, and impact attenuators (*1*). These figures represent 22.5 percent and 7.4 percent, respectively, of all accidents that occurred on these types of roads. On Interstate roads, 26 (1.5 percent) of the fixed-object accidents involving highway safety hardware resulted in fatalities, 754 (43.7 percent) in injuries, and 946 (54.5 percent) in property damage. On the primary roads, 32 (1.8 percent) of the fixed-object accidents involving highway safety hardware resulted in fatalities, 802 (44.9 percent) in injuries, and 951 (53.3 percent) in property damage.

If highway safety hardware items are struck and damaged by vehicles, they can no longer fully perform their intended function, which is to protect motorists from identified hazards. Therefore an adequate level of maintenance is required to preserve the functional integrity of the safety hardware (*2*). This can be achieved by inspecting and repairing the hardware at intervals that are frequent enough to maximize its safety benefits, subject to the available resources.

The sequence of events in the damage and repair of safety hardware is shown in Figure 1. It is desirable for the restoration time (t_r) to be less than the time between accidents (t_a) for maximum safety.

A METHOD FOR DETERMINING INSPECTION AND REPAIR FREQUENCIES

The Federal Highway Administration (FHWA) has developed a method for determining the frequencies at which safety hardware should be inspected and repaired (*2*). The frequencies for the inspection and repair of hardware items are determined on the basis of the accident history of the items and the level of service to be provided, which is defined as the desired probability of completing the inspection and repair before a subsequent accident. This definition of level of service is fairly new and consequently has limited acceptance to date. The Poisson frequency distribution is used to determine inspection and repair intervals statistically.

Examples of the method may be made by using Table 1. If the average annual accident frequency is 2.0 and the probability of no accidents before completing a repair equals 0.95, then the repair must be completed in 9.4 days. For a lower confidence level of 0.90, the period for completion is 19.2 days.

The method is flexible in that it can be applied at different organizational levels for different types of hardware and for different classes of roads. Its versatility has been demonstrated by its usage for planning and managing the inspection and repair of safety hardware and other types of equipment, for preparing budgets, and for allocating funds. This method has much potential, but it had not been field tested.

OBJECTIVE

The objective of this research was to use fields tests to evaluate the method developed for the FHWA. The method was tested on five sites at which one or more of the following types of safety hardware had been installed: roadway barriers, bridge rails, impact attenuators, breakaway sign supports, and breakaway luminaire supports.

IDENTIFICATION OF HIGH-HAZARD SITES

Site Selection Criteria and Approach

The identification and selection of sites took into consideration the following factors: the highest accident frequencies involving safety hardware, a broad range of average daily traffic (ADT) volumes with a minimum of 15,000 vehicles or more, no planned construction or maintenance activities that would affect the site during the monitoring period, and the willingness of maintenance personnel to participate.

Description of Field Sites

A description of the five sites is provided in Table 2. This description includes location, length, ADT, mean number of accidents involving highway safety hardware per year for 1981–1983, roadway description, and an inventory of highway safety hardware.

Virginia Department of Highways and Transportation, Transportation Research Council, Box 3817 University Station, Charlottesville, Va. 22903.

Legend

E_i = event i
E_0 = safety hardware installation
E_1 = accident involving safety hardware
E_2 = detection of damaged safety hardware
E_3 = repair work is begun
E_4 = repair work is completed
E_5 = subsequent accident involving safety hardware

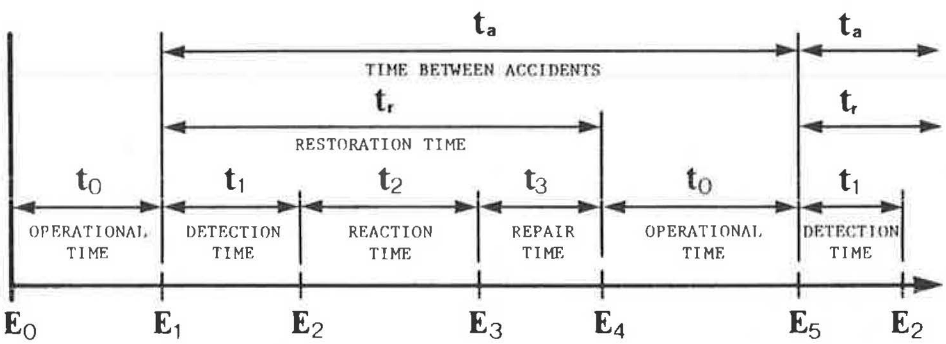


FIGURE 1 Sequence of events in damage and repair of safety hardware [source: *A Method for Determining Frequencies for Inspection and Repair of Highway Safety Hardware* (*2*)].

DATA COLLECTION PROCEDURE

The objective of the field test was to collect data on highway safety hardware inspection and repair activities at the five sites for 1 year so that the highway safety hardware maintenance guidelines could be evaluated. A monthly inspection and repair report and a damage and repair report were completed by the maintenance foreman responsible for inspection and repair at each site. The following information was collected on the forms:

- The frequency of inspection and repair activities;
- The number of times that the highway safety hardware was damaged by vehicle impact;
- The maintenance crew time in person-hours to maintain the safety hardware;
- The cost of materials and parts used to maintain the highway safety hardware;
- How the maintenance supervisor found out about the damage to highway safety hardware, the cause of the damage, and knowledge of previous damage;
- When the damage was scheduled for repair and when the repair work began and was completed.

ANALYSIS

The analysis of the data is divided into the following sections: inspection and repair activities, inspection schedule adherence, damage reporting, damage and repair report summary, and second accidents. The highway safety hardware inspection and repair activities at the field sites are discussed below for each site.

Inspection

A summary of the inspection and repair reporting activities is presented in Table 3. The two study sites on I-395 and the Route 50 site were divided by highway safety hardware and traffic signs because these roads are maintained by different area headquarters. The reporting of damaged highway safety hardware on Interstate 395 and Route 50 depends very heavily on the police because the inspector only reported severely damaged guardrails. On the basis of a two-sided *t*-test for significant difference between the actual and scheduled inspection intervals, there was no significant difference between the two intervals with a level of confidence of 0.05 (*3*).

Repair Activities

Traffic signs and impact attenuators (except on Route 50) are repaired immediately by departmental forces, but guardrail damages are repaired on contract. Ground-mounted traffic signs are repaired during inspection, and overhead signs are repaired by the district traffic staff.

Guardrail repair contracts are negotiated for each district. The basic contract provisions are as follows (*4*):

> This work shall consist of replacing and installing guardrail and median barrier in reasonably close conformity with the existing lines and grades or as directed by the engineer. Minimum repair call will be 200 linear feet per city or county and repair operations shall begin within five (5) working days after notice is received. The contractor shall advise the engineer at least 24 hours prior to commencement of work. The contractor shall not begin work at any location until the location and extent of work has been verified and approved by the engineer or his representative.

If the department is not able to perform emergency guardrail repairs, such as on Route 150 and Interstate 64, the following provision is added (*4*):

> The contractor will be expected to make an emergency response within twenty four (24) hours for locations where emergency repairs of guardrail end sections and exposed guardrail sections are necessary.

TABLE 1 MAXIMUM INSPECTION OR RESTORATION TIME IN DAYS AS A FUNCTION OF AVERAGE ANNUAL ACCIDENTS AND POISSON PROBABILITIES

$\bar{A}$ Average Annual Accidents	P(0) = PROBABILITY OF NO ACCIDENTS								$\bar{A}$ Average Annual Accidents
	0.800	0.850	0.900	0.925	0.950	0.975	0.990	0.995	
0.2	407.2	296.6	192.3	142.3	93.6	46.2	18.3	9.1	0.2
0.4	203.6	148.3	96.1	71.1	46.8	23.1	9.2	4.6	0.4
0.6	135.7	98.9	64.1	47.4	31.2	15.4	6.1	3.0	0.6
0.8	101.8	74.1	48.1	35.6	23.4	11.6	4.6	2.3	0.8
1.0	81.4	59.3	38.5	28.5	18.7	9.2	3.7	1.8	1.0
1.2	67.9	49.4	32.0	23.7	15.6	7.7	3.1	1.5	1.2
1.4	58.2	42.4	27.5	20.3	13.4	6.6	2.6	1.3	1.4
1.6	50.9	37.1	24.0	17.8	11.7	5.8	2.3	1.1	1.6
1.8	45.2	33.0	21.4	15.8	10.4	5.1	2.0	1.0	1.8
2.0	**40.7**	**29.7**	**19.2**	**14.2**	**9.4**	4.6	1.8	0.9	2.0
2.2	37.0	27.0	17.5	12.9	8.5	4.2	1.7	0.8	2.2
2.4	33.9	24.7	16.0	11.9	7.8	3.9	1.5	0.8	2.4
2.6	31.3	22.8	14.8	10.9	7.2	3.6	1.4	0.7	2.6
2.8	29.1	21.2	13.7	10.2	6.7	3.3	1.3	0.7	2.8
3.0	27.1	19.8	12.8	9.5	6.2	3.1	1.2	0.6	3.0
3.2	25.5	18.5	12.0	8.9	5.9	2.9	1.1	0.6	3.2
3.4	24.0	17.4	11.3	8.4	5.5	2.7	1.1	0.5	3.4
3.6	22.6	16.5	10.7	7.9	5.2	2.6	1.0	0.5	3.6
3.8	21.4	15.6	10.1	7.5	4.9	2.4	1.0	0.5	3.8
4.0	20.4	14.8	9.6	7.1	4.7	2.3	0.9	0.5	4.0
4.2	19.4	14.1	9.2	6.8	4.5	2.2	0.9	0.4	4.2
4.4	18.5	13.5	8.7	6.5	4.3	2.1	0.8	0.4	4.4
4.6	17.7	12.9	8.4	6.2	4.1	2.0	0.8	0.4	4.6
4.8	17.0	12.4	8.0	5.9	3.9	1.9	0.8	0.4	4.8
5.0	16.3	11.9	7.7	5.7	3.7	1.8	0.7	0.4	5.0
5.2	15.7	11.4	7.4	5.5	3.6	1.8	0.7	0.4	5.2
5.4	15.1	11.0	7.1	5.3	3.5	1.7	0.7	0.3	5.4
5.6	14.5	10.6	6.9	5.1	3.3	1.7	0.7	0.3	5.6
5.8	14.0	10.2	6.6	4.9	3.2	1.6	0.6	0.3	5.8
6.0	13.6	9.9	6.4	4.7	3.1	1.5	0.6	0.3	6.0
6.2	13.1	9.6	6.2	4.6	3.0	1.5	0.6	0.3	6.2
6.4	12.7	9.3	6.0	4.4	2.9	1.4	0.6	0.3	6.4
6.6	12.3	9.0	5.8	4.3	2.8	1.4	0.6	0.3	6.6
6.8	12.0	8.7	5.7	4.2	2.8	1.4	0.5	0.3	6.8
7.0	11.6	8.5	5.5	4.1	2.7	1.3	0.5	0.3	7.0
7.2	11.3	8.2	5.3	4.0	2.6	1.3	0.5	0.3	7.2
7.4	11.0	8.0	5.2	3.8	2.5	1.2	0.5	0.2	7.4
7.6	10.7	7.8	5.1	3.7	2.5	1.2	0.5	0.2	7.6
7.8	10.4	7.6	4.9	3.6	2.4	1.2	0.5	0.2	7.8
8.0	10.2	7.4	4.8	3.6	2.3	1.2	0.5	0.2	8.0
8.2	9.9	7.2	4.7	3.5	2.3	1.1	0.4	0.2	8.2
8.4	9.7	7.1	4.6	3.4	2.2	1.1	0.4	0.2	8.4
8.6	9.5	6.9	4.5	3.3	2.2	1.1	0.4	0.2	8.6
8.8	9.3	6.7	4.4	3.2	2.1	1.1	0.4	0.2	8.8
9.0	9.0	6.6	4.3	3.2	2.1	1.0	0.4	0.2	9.0
9.2	8.9	6.4	4.2	3.1	2.0	1.0	0.4	0.2	9.2
9.4	8.7	6.3	4.1	3.0	2.0	1.0	0.4	0.2	9.4
9.6	8.5	6.2	4.0	3.0	2.0	1.0	0.4	0.2	9.6
9.8	8.3	6.1	3.9	2.9	1.9	0.9	0.4	0.2	9.8
10.0	8.1	5.9	3.8	2.8	1.9	0.9	0.4	0.2	10.0

$$t = -\frac{365 \ln P(0)}{\bar{A}}$$

TABLE 2 DESCRIPTION OF THE FIELD SITES

Site No.	Location	Length (mi)	1984 ADT	Highway Safety Hardware Accidents, 1981–1983 (mean no./yr)	Roadway Description	Guardrail (linear ft)	Bridge Rail (linear ft)	Concrete Barrier (linear ft)	No. of Impact Attenuators	No. of Ground-Mounted Signs Exposed to Traffic
1	I-395, Part 1: from I-95 to Arlington Co. line (Fairfax Co. and Alexandria)	5.30	121,020	62.3	6 lanes with 2 reversible HOV lanes in median	58,365	1,441		4	13
2	I-395, Part 2: Arlington Co.	4.38	135,105	52.3	6 lanes with 2 reversible HOV lanes in median	19,130	4,995	16,900	9	5
3	I-64 from Route 258 (Mercury Blvd.) to Route 167 (La Salle Ave.) Hampton	2.00	61,135	19.0	4 lanes divided by grass	17,420	400	3,690	2	16
4	Route 50, Arlington County	5.20	46,765	12.0	6 lanes divided by guardrail barrier, with a short 4-lane undivided section	7,320	713	4,013	1	101
5	Route 150 from Route 360 to Route 1	5.45	28,880	9.0	4 lanes divided	37,940	5,600	0	0	32

NOTE: The typical lane width is 12 ft for Route 50; lane width varies from 11 to 12 ft. With the exception of Route 50, all sites have paved shoulders. On the two sections of I-395, luminaire posts are located behind guardrail at a spacing of 160–200 ft. Highway safety hardware on ramps to and from the test sections were not inventoried.

TABLE 3 INSPECTION AND REPAIR ACTIVITIES

Site No.	Site Description	Inspection Intervals (days)	Repairer	Repair Frequency
1	I-395, hardware, Part 1	Impact attenuator: 15	Department	Immediately
		Guardrail: special[a]	Contract	Scheduled
	I-395, signs, Part 1	3	Department	Immediately
2	I-395, hardware, Part 2	Impact attenuator: 15	Department	Immediately
		Guardrail: special[a]	Contract	Scheduled
	I-395, signs, Part 2	3	Department	Immediately
3	I-64	5	Hardware: contract	Scheduled
			Signs: department	Immediately
4	Route 50, hardware	Special[a]	Contract	Guardrail: scheduled
				Impact attenuators: immediately
	Route 50, signs	3	Department	Immediately
5	Route 150	4	Hardware: contract	Scheduled
			Signs: department	Immediately

[a]Damage reporting is provided primarily by police, who make their reports in three ways: (a) dispatcher to dispatcher for emergencies (impact attenuator damage and severe guardrail damage), (b) road hazard report (sent immediately), and (c) accident report. A maintenance foreman notes badly damaged hardware during inspection drives.

TABLE 4 RANGE OF INSPECTION AND RESTORATION INTERVALS

Expected Number of Days Between Successive Hits							
Group No.	Hits per Year	Selected Probability Levels 0.7	0.8	0.9	0.95	0.975	0.99
1	14.0	9.3	5.8	2.7	1.3	0.7	0.3
2	4.4	29.6	18.5	8.7	4.3	2.1	0.8
3	3.0	43.4	27.1	12.8	6.2	3.1	1.2
4	2.0	65.1	40.7	19.2	9.4	4.6	1.8
5	1.0	130.2	81.4	38.5	18.7	9.2	3.7
6	5.0	26.0	16.3	7.7	3.7	1.8	0.7
Number of Second Hits							
Interstate							
Subgroup 6	1	1	1	0	0	0	0
Subgroup 3	1	1	1	0	0	0	0
Subgroup 4	3	2	2	2	2	1	1
Subgroup 5	0	0	0	0	0	0	0
Total	5	4	4	2	2	1	1
Primary							
Subgroup 2	6	3	2	1	0	0	0
Subgroup 3	1	0	0	0	0	0	0
Subgroup 4	4	0	0	0	0	0	0
Subgroup 5	0	0	0	0	0	0	0
Total	11	3	2	1	0	0	0

The minimum repair call of 200 linear feet is included to ensure that at least a full day's work on guardrail repair is requested. The objective is to maximize the productivity of the guardrail repair crew while minimizing the travel required between locations for 1 day.

FOLLOWING THE METHOD

Five steps are suggested for applying the method:

Step 1: Obtain the frequency data on traffic accidents involving highway safety hardware. The 1-year monitoring of inspection and repair activities provided these data in lieu of department traffic accident records or special studies. In fact, the monitoring may be considered a special study. The monitoring identifies reported and unreported accidents involving highway safety hardware. The basic locational unit is typically 0.1 mi.

Step 2: Rank accident locations in decreasing order of average annual safety hardware accidents.

Step 3: Sort the locations by road class and identify accident groups (by similar accident frequencies).

Steps 2 and 3 were performed together by using Lotus 1-2-3 microcomputer software functions. The locations were sorted by Interstate and primary route sections. Six groups were formed for the 49 interstate locations and for the 30 primary-route locations.

Step 4: Identify the ranges of inspection and restoration intervals for each group. The ranges of inspection and repair intervals are presented in Table 4. The procedure to develop the ranges is based on the equation for t in Table 1. The average and maximum numbers of hits of the group are displayed, as well as the average number of hits by road class. The second part of Table 4 shows the impact of the level of service on the

TABLE 5 EXISTING LEVEL OF SERVICE AND DESIRED LEVEL OF SERVICE

	Hits per Year	Existing		Desired	
		Restoration Interval (days)	Level of Service	Restoration Interval (days)[a]	Level of Service
I-395, hardware, Part 1	5	7 + 26 = 33	.64	16.3	.80
I-395, hardware, Part 2	3	7 + 33 = 40	.72	27.1	.80
I-395, signs	6	3	.95	3	.95
I-64	4	5 + 121 = 126	.25	20.4	.80
Route 50, signs	18	3	.86	3	.86
Route 50, hardware	2	3 + 33 = 36	.82	36	.82
Route 150	5	4 + 42 = 46	.53	27.1	.80

[a]From Table 1.

number of hits. A level of service of 0.975 is required to minimize the number of second accidents for Interstate subgroup 4. The one accident remaining is the result of two accidents having been reported on the same day.

Step 5: Select a level of service. Because the selection of a level of service requires a policy decision, the policy was based on existing practice and contract provisions. The inspection interval required was equal to the existing average inspection interval but was not greater than 7 days. The restoration period specified in the contract for guardrail maintenance was 5 working days; this was expanded to 7 calendar days. The long reaction times are the primary factor in the level of service, and they are contingent on the requirement that there be 200 linear feet of guardrail in need of repair before the repair crews are committed to the repair work. This requirement makes the restoration period unpredictable and widely variable from county to county. Moreover, in at least one county the contractor does not have the equipment and human resources to perform the work within contract provisions. The existing levels of service calculated for the field sites and the restoration levels required to achieve a minimum level of service of 0.8 are presented in Table 5. The minimum level of service was based on the assumption that it is a practical lower limit of level of confidence in statistics.

Four of the levels of service are below 0.8. To reduce the existing restoration intervals so that the intervals required for a 0.8 level of service are obtained, substantial time reductions are needed. Obviously, changes in the contract's provisions and their enforcement would be essential to reach the minimum desired level of service, along with a reduction in inspection intervals.

PROBLEMS WITH THE METHOD

Overestimate of Second Accidents

The number of second accidents expected was significantly greater than the actual number of second accidents. According to the maintenance supervisors at the study sections, second accidents seldom occur. It is quite common, however, for accidents to occur about 50 to 100 ft from the damaged safety hardware. This problem may be resolved by applying an adjustment factor to reduce the estimate of second accidents or by basing the expected number of second accidents on the actual experience of second accidents. The value of using an adjustment factor is questionable because it lacks a theoretical basis. This problem is eliminated if the overestimate is perceived as a margin of safety.

The number of second accidents expected on the basis of the procedure is approximately equal to the annual number of accidents. This explains why the procedure predicted the actual number of second accidents poorly. It is very important in the procedure to state that the worst conditions are addressed, so that the procedure will not be expected to predict actual second accidents.

Definition of a Location

The number of accidents at a location would be significantly reduced by using 0.01 mi (52.8 ft) as the basic unit of measurement, as is done in Virginia, rather than the recommended 0.1 mi (528 ft). This change would also allow better identification of the accidents that occur near the damaged safety hardware. The next step in defining the location more specifically is to consider the direction of travel of the vehicle and the side of the road on which the damaged safety hardware is located. These changes substantially reduced the number of hits per year for each site. Consequently, when the current inspection repair activities are applied to the revised number of accidents, the level of service substantially increases. The existing level of service in Table 5 is revised in Table 6 for a 0.01-mi basic unit, direction, and side of road. The level of service increases to greater than 0.7 for all sections, compared to three sections with levels of service below 0.7 for the 0.1-mi basic unit. Consequently, the method of defining the location significantly affects the results of the procedure. The more well defined the location, the more accurately the potential for a second accident is estimated. The need for improving the accuracy in identifying accident locations by the police who complete the accident reports has been recognized.

Immediate Versus Scheduled Repairs

In practice, the damage to the highway safety hardware is assessed and is either considered for immediate repair if there is a definite hazard or scheduled for later repair if the damage is minor or less of a hazard and the guardrail is functional. The procedure does not take this classification into consideration. Moreover, severely damaged highway safety hardware is sometimes reported immediately by police. Consequently, the safety hardware may be repaired before the next inspection. These activities reduce the potential for the occurrence of a second accident. It would be helpful if this issue were taken

TABLE 6 COMPARISON OF LEVEL OF SERVICE BY LOCATION UNIT

	Location Unit = 0.1 mi			Location Unit = 0.01 mi, by Direction and Side of Road	
	Hits per Year	Restoration Interval	Level of Service	Hits per Year	Level of Service
I-395, hardware, Part 1	5	7 + 26 = 33	.64	3	.76
I-395, hardware, Part 2	3	7 + 33= 40	.72	3	.72
I-395, signs	6	3	.95	2	.98
I-64	4	4 + 121 = 126	.25	1	.71
Route 50, signs	18	3	.86	8	.94
Route 50, hardware	2	3 + 33 = 36	.82	1	.91
Route 150	5	4 + 42 = 46	.53	1	.88

into consideration in the procedure. An immediate repair may assume a level of service of 0.995.

Need for Traffic Safety Evaluation

It would be helpful if the procedure emphasized the need for traffic safety evaluations at locations with high accident frequencies. Safety improvements may be substantially effective in reducing first accidents as well as second accidents. Although safety improvements are not in the scope of the study, the procedure is remiss in not mentioning the need.

CONCLUSION

The method described in *A Method for Determining Frequencies to Inspect and Repair Highway Safety Hardware* (2) appears to have a high potential for improving highway safety hardware maintenance practices. On the basis of the findings of this field evaluation, the method has been determined to be useful for highway safety hardware maintenance guidelines.

Most maintenance guidelines are determined subjectively. This method provides statistically based quantitative guidelines that allow incremental maintenance needs (inspection and restoration intervals) and benefits (reduced number of second accidents) to be realized. Moreover, because the method determines inspection and repair intervals for the worst conditions, a substantial margin of safety is built in.

ACKNOWLEDGMENTS

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