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Traffic and Grade Crossing Control Devices

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Foreword

The papers in this Record address the problems and policy issues facing the practicing traffic engineer on a day-to-day basis. From determining the appropriate traffic control device at intersections, school crossings, or railroad-highway grade crossings; selecting the design of traffic control devices that will accommodate the needs of the aging driving population; choosing the optimal timing sequence for actuated signals; or obtaining motorist compliance with the speed limits that are in place, the papers in this Record will provide information and guidance for the reader.

The first two papers are related by their interest in motorist compliance with speed limits and the effects of operating speeds on accidents. Upchurch examines Arizona's experience with the 65-mph speed limit since the speed limit was raised in 1987. He reports on driver behavior as exhibited by the operating speeds of the vehicles and presents a before-and-after analysis of accident data in terms of accident frequency and also accident rate. Pigman et al. evaluate the effectiveness of unmanned radar installations in reducing the number of vehicles traveling at excessive speeds. Their results indicated both a significant reduction in the speeds of vehicles that are equipped with radar detectors and a reduction in truck-related and speed-related accidents.

The next two papers address the issue of selecting the proper traffic control for existing conditions. Celniker proposes a new policy for determining the need for all-way stop sign control based on accidents, unusual conditions, traffic volumes, and pedestrian volumes. A before-and-after analysis of accident data and field performance was conducted as a test of the policy, and positive results were reported. Bonneson and Blaschke review the current criteria used to determine the need for traffic control at school crossings and propose a simpler procedure that also addresses the issue of interruption of pedestrian flows and the need for minimum pedestrian volumes.

Bullen uses detector type and placement and the settings for minimum green and vehicle extension in the EVIPAS simulation and optimization model to analyze actuated traffic signals in terms of vehicle delay. He determined that the most critical variable affecting vehicle delay at high-volume, traffic-actuated signals is the vehicle extension settings, particularly for passage-type detectors.

The next two papers identify and evaluate active warning devices for railroad-highway grade crossings. Heathington et al. compare the four-quadrant gate system with the standard two-quadrant system in terms of various measures of effectiveness. They found that the four-quadrant system had no negative effect on a crossing's level of service while providing a positive effect on driver behavior and compliance. Fambro et al. evaluated two other active traffic control devices—four-quadrant flashing light signals with overhead strobes and standard highway traffic signals. On the basis of field evaluation, they found driver response to the highway traffic signal at railroad-highway grade crossings to be excellent, outperforming the standard flashing beacon on several safety and driver behavior measures of effectiveness.

From laboratory studies, Staplin et al. compared the differences in visual performance for older drivers relative to their much younger counterparts and the resultant recognition of delineation and sign word-messages. The results from their studies provide interesting implications for the design of traffic control devices to accommodate the older-driver population.

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Contents

Arizona's Experience with the 65-mph Speed Limit

JONATHAN UPCHURCH

Arizona's experience with the 65-mph speed limit is presented in terms of driver behavior and accident experience. The speed limit on Arizona's rural interstate was raised to 65 mph on April 15, 1987. Driver behavior is presented in terms of the speeds at which motorists actually drive on the rural interstate. Before and after data are presented from the last quarter of 1983 through the first quarter of 1988. Vehicle speeds increased by only about 3 mph or less during the four quarters following the speed limit increase. A 5-year history of interstate accident data—from 1983 through spring 1988—is presented that provides a before-and-after comparison. Information on total accidents, fatal accidents, and injuries is presented. Accident rate information is presented to account for the effect of increasing vehicle-miles of travel. Accident data on the urban interstate are presented for comparison purposes.

Speed limits on rural highways has been a topic of intense interest to both the general public and the traffic engineering and enforcement communities during the past 15 years. The national maximum speed limit of 55 mph was enacted in 1974 and remains in effect on most of the nation's rural highway mileage. In April 1987, the United States Congress passed legislation allowing individual states to increase the speed limit on the rural interstate system to 65 mph. To date, about 40 states have chosen to increase the speed limit on the rural interstate.

Increasing the speed limit to 65 mph has led to an intensified debate about the impact of the higher speed limit on safety. Proponents and opponents have engaged in spirited discussion. Quantitative data have been assembled and presented to show changes in the number of accidents but, thus far, the information has been based on relatively short periods. Information on changes in driver behavior (actual speeds driven) has received little attention.

A statistically sound evaluation and appraisal of the accident impacts of increasing the speed limit will require nationwide data from both the states in which the speed limit has been raised and the states in which it has not been raised. It will also require at least 12 months of "after" data from each of the states in which the speed limit has been raised. Because some states raised their speed limits as late as the fall of 1987, the type of rigorous evaluation described above is unlikely to have been completed before the end of 1988.

This paper presents the experience of a single state—Arizona—with the 65 mph speed limit. This single state experience is not intended to be representative of experience in other states. Information is presented on both driver behavior (a before-and-after comparison of the speeds that motorists are actually driving) and accident experience (a before-and-after comparison). The "before" period in Arizona ends on April 15, 1987, the date that the speed limit was raised. The after period begins on April 16, 1987.

Throughout this paper the term "rural interstate" is used to denote those portions of the Arizona interstate system that now have a 65-mph speed limit. "Urban interstate" is used to denote those portions that still have a 55-mph speed limit.

This paper is intended to simply present the facts on changes in driver behavior and actual numbers of accidents. It is not intended to interpret, demonstrate, or imply any cause and effect relationship between changes in the speed limit and accident experience.

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DRIVER BEHAVIOR

The Arizona Department of Transportation has about 76 speed-monitoring compliance locations on its highway system. Thirty-five are located on the rural interstate, 12 are located on the urban interstate, and 29 are located on the rural primary system. Although federal law no longer requires speed monitoring data to be collected on the 65 mph interstate, Arizona has continued to do so.

Fourteen calendar quarters of before speed data and four quarters of after speed data were analyzed, and the results are presented in the following paragraphs. This study used raw speed data collected at speed monitoring sites—data that have not been adjusted in the ways that are used for the 55-mph compliance purposes. For example, the speeds have not been adjusted for speedometer error. In addition, the speeds reported here are only for the rural interstate. Speeds on the urban interstate and on rural primary highways are lower. As a result, the speeds reported herein for the rural interstate are different from, and higher than, those reported by Arizona for speed limit compliance purposes. Overall, Arizona motorists are complying with the 55-mph speed limit.

Figure 1 presents data on the 50th percentile speed and the 85th percentile speed for a composite of 26 locations on the rural interstate. In Arizona speed is not measured at every speed monitoring station in every calendar quarter. Thus, Figure 1 presents data from sites that varied, to some extent, from quarter to quarter. The percentile speed in a given quarter was computed as the weighted average of the percentile speed at each of the locations (weighted in proportion to the traffic volume at each location). The 50th percentile speed stayed almost constant, at about 59 to 60 mph—from 1984 through 1986. The 85th percentile speed also stayed fairly constant—at about 65 mph. An observable, though small, increase in speeds occurred in the after period. Fiftieth percentile speeds

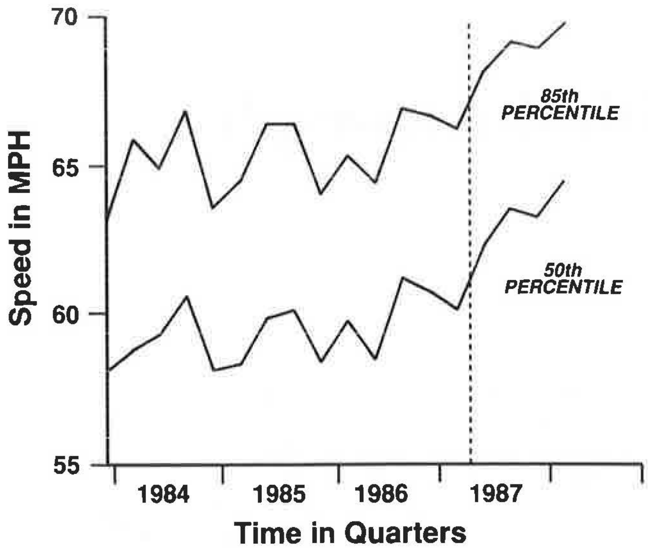

FIGURE 1 Percentile speeds on rural interstate (26 locations).

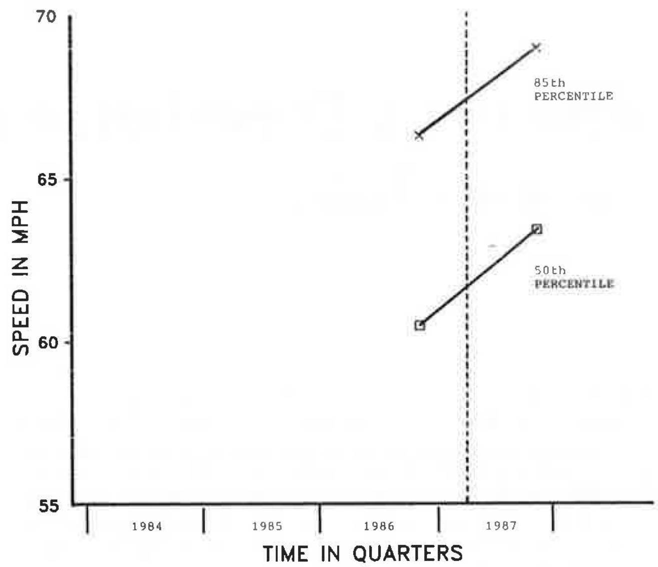

FIGURE 2 Percentile speeds on rural interstate (9 locations).

increased to about 62 to 64 mph and 85th percentile speeds increased to about 68 to 69 mph.

Figure 2 presents 50th and 85th percentile speed data for nine rural interstate locations where data were collected at all nine sites in the fall of 1986 and again in the fall of 1987. Fiftieth percentile speeds increased from 60.5 to 63.4 mph and 85th percentile speeds increased from 66.3 to 69.0 mph. Both cases represent an increase of less than 3.0 mph.

Figure 3 presents, for the composite of 26 interstate locations, the percent of vehicles in the traffic stream that were exceeding 55, 60, and 65 mph. Once again, there appears to be no trend in speeds during the before period. The percent of vehicles exceeding 55 mph increased from about 80 percent in the before period to about 88 to 91 percent in the after period. The percent of vehicles exceeding 60 mph increased from about 50 percent in the before period to about 70 to 76 percent in the after period. The percent of vehicles exceeding 65 mph increased from about 20 percent in the before period to about 37 to 47 percent in the after period.

Figure 4 presents the same type of information for the nine rural locations. The percentages are summarized below:

Percent of Vehicles Exceeding	*Fall 1986*	*Fall 1987*
55 mph	82	89
60 mph	52	71
65 mph	19	38

The data indicate that there is slightly more dispersion of vehicle speeds. In the fall of 1986, 63 percent of the vehicles were traveling between 55 and 65 mph. In the fall of 1987, 51 percent of the vehicles were traveling between 55 and 65 mph.

Driver behavior on the Arizona urban interstate was also evaluated by using data from the 12-speed monitoring compliance locations on the urban interstate. This evaluation was done to determine if there was any change in driver behavior in urban areas after the change in the rural interstate speed limit. Since the objective was to measure driver behavior during free-flow, unconstrained conditions, speed data for those hours in which high traffic volumes caused speeds to be reduced were not included in the evaluation. The evaluation showed that urban interstate speeds remained the same or exhibited a slight decrease after the rural interstate speed limit was increased.

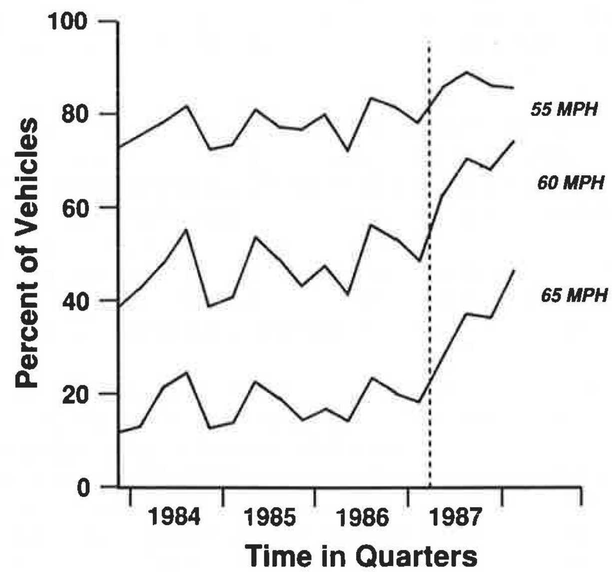

FIGURE 3 Percent of vehicles exceeding given speed (26 locations).

ACCIDENT EXPERIENCE

Data on numbers of accidents in 1983 through April 15, 1988, are presented in this section. To supplement these data, Table 1 presents information on vehicle-miles of travel on the urban and rural interstate in the same years. The data on numbers of accidents and vehicle-miles of travel are combined to present information on accident rates.

In this section of this paper, the "1-year-after period" refers to the 12 months from April 16, 1987, to April 15, 1988.

Historical accident data, beginning in 1983 and extending through April 15, 1988, are presented in Table 2.

For comparison purposes, Figure 5 presents a 5-year record of accidents on the urban interstate. As shown, there was only a very slight growth in the total number of accidents from 1984 through April 15, 1988. During this period vehicle-miles of travel on the urban interstate increased from 1.360 billion in 1983 to 1.907 billion in the 1-year-after period. As shown in Figure 6, there was a downward trend in the accident rate from 1984 through April 1988.

Figure 7 shows that accidents on the rural interstate stayed fairly constant from 1984 through 1986. An observable increase occurred for the 1-year-after period; vehicle-miles of travel on the rural interstate increased from 3.745 billion in 1983 to 4.966 billion in the 1-year-after period.

When accident rates are plotted (Figure 8), the observable increase in accidents in the 1-year-after period is not so apparent. Although the accident rate in the 1-year-after period is higher than that in 1986, it is virtually the same as the 1983–1985 average. Figure 9 presents a bar chart for fatal accidents on the rural interstate. The figure shows an increase in the

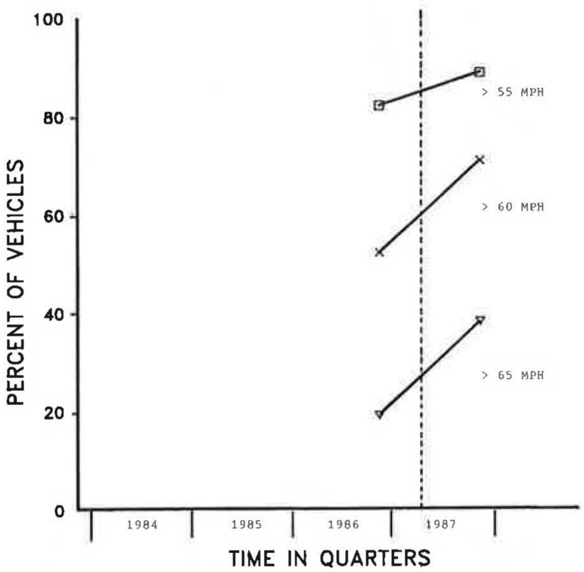

FIGURE 4 Percent of vehicles exceeding given speed (9 locations).

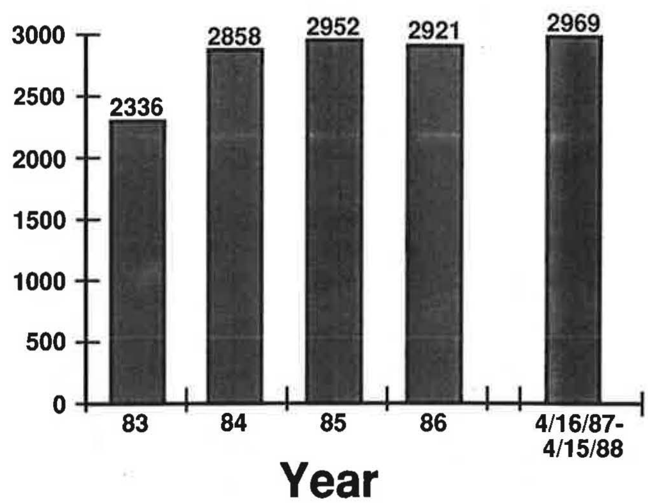

FIGURE 5 Total accidents on urban interstate.

TABLE 1 VEHICLE-MILES OF TRAVEL ON INTERSTATE SYSTEM

Type of Highway	Annual Vehicle-Miles of Travel ($\times$ 10^6) 1983	1984	1985	1986	1987	4/16/87 through 4/15/88[a]
Urban interstate	1,360.0	1,469.7	1,577.0	1,791.4	1,862.1	1,906.6
Rural interstate	3,745.0	3,991.7	4,128.7	4,619.9	4,869.5	4,966.1

[a]Estimate based on 4-year growth trend in vehicle-miles of travel.

TABLE 2 NUMBER OF ACCIDENTS ON INTERSTATE SYSTEM

Type of Damage	No. of Accidents 1983	1984	1985	1986	1/1/87 through 4/15/87	4/16/87 through 4/15/88
Urban interstate						
Property damage only	1,717	2,092	2,124	2,105	681	2,217
Injury	609	750	815	803	215	737
Fatal	10	16	13	13	7	15
Total	2,336	2,858	2,952	2,921	903	2,969
Rural interstate						
Property damage only	1,428	1,654	1,757	1,669	718	1,969
Injury	978	1,052	1,015	1,047	326	1,322
Fatal	71	82	92	97	20	117
Total	2,477	2,788	2,864	2,813	1,064	3,408

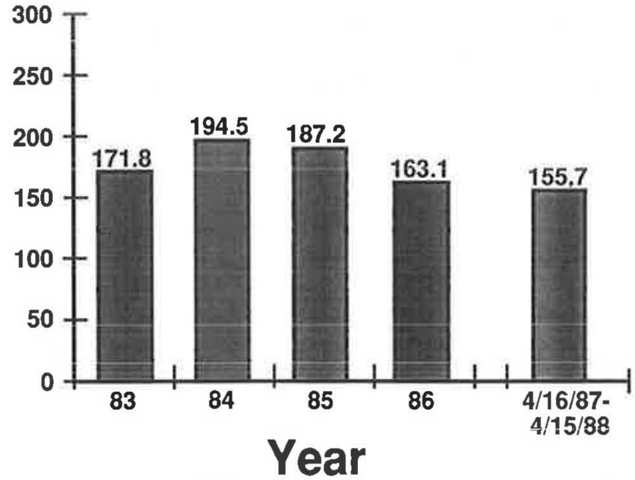

FIGURE 6 Accident rate for total accidents on urban interstate.

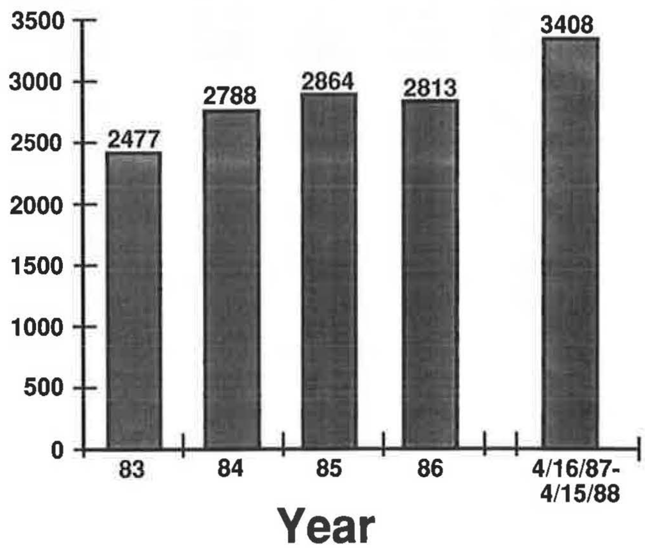

FIGURE 7 Total accidents on rural interstate.

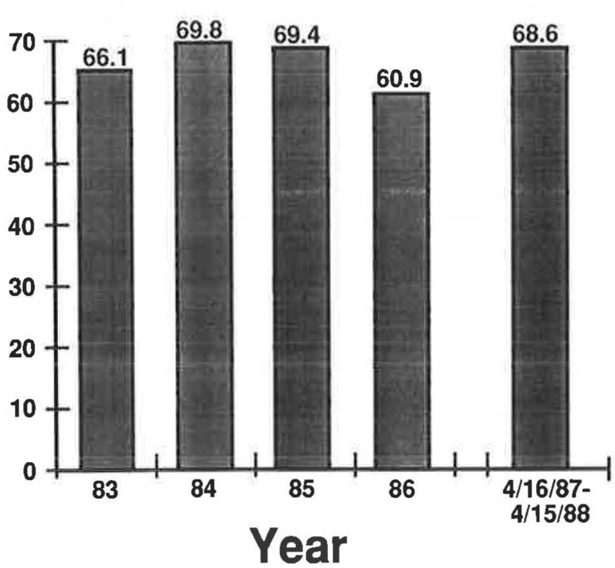

FIGURE 8 Accident rate for total accidents on rural interstate.

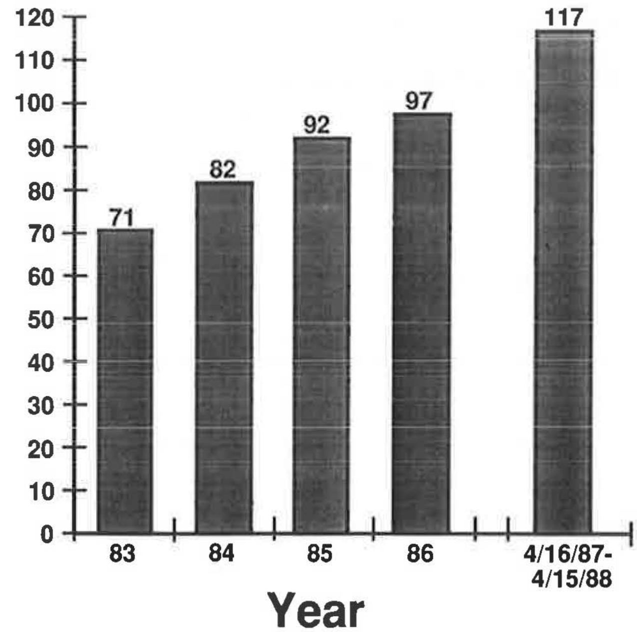

FIGURE 9 Fatal accidents on rural interstate.

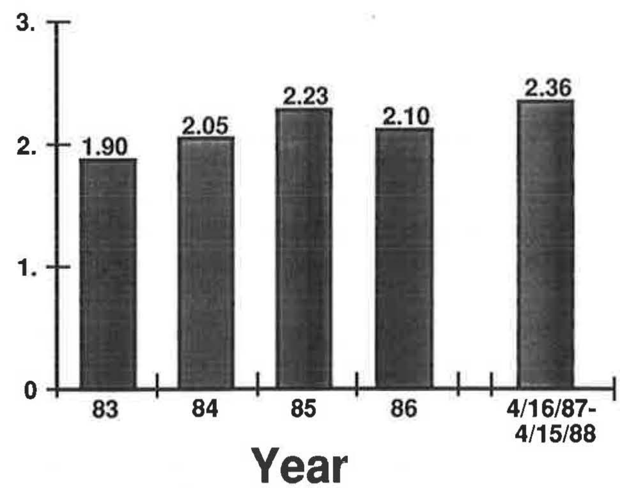

FIGURE 10 Accident rate for fatal accidents on rural interstate.

number of fatal accidents from 1983 to 1986. An additional increase is found in the number of fatal accidents for the 1-year-after period. When adjusted for vehicle-miles of travel there is still an upward trend. Figure 10 shows that the fatal accident rate generally increased from 1983 through April 1988.

Injury accidents (Figure 11) show little change from year to year from 1984 through 1986. An increase is found for the 1-year-after period. Figure 12 presents the injury accident rate. When presented in this form, the increase in accidents in the 1-year-after period is not so apparent. The accident rate in the 1-year-after period is more than that in 1986 and 1985 but it is about the same as that in 1984 and 1983.

Figures 13 and 14 present comparisons of changes in the number of accidents on the rural interstate versus that on the urban interstate. As shown in Figure 13, fatal accidents

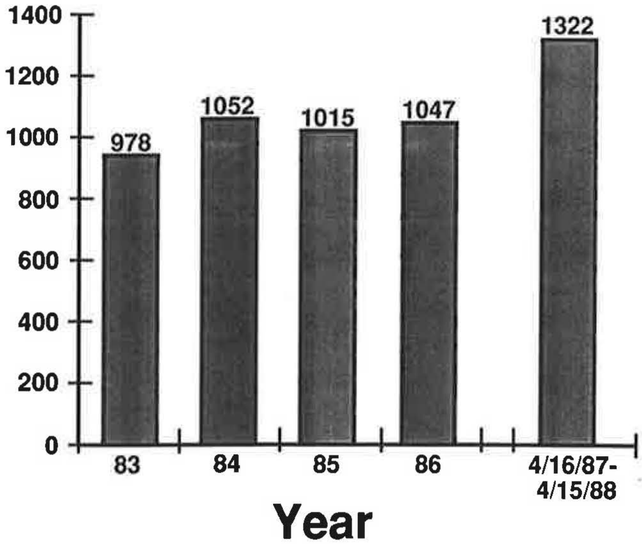

FIGURE 11 Injury accidents on rural interstate.

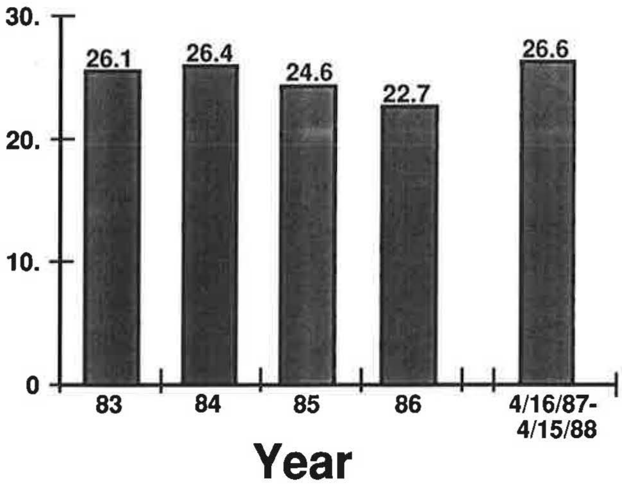

FIGURE 12 Accident rate for injury accidents on rural interstate.

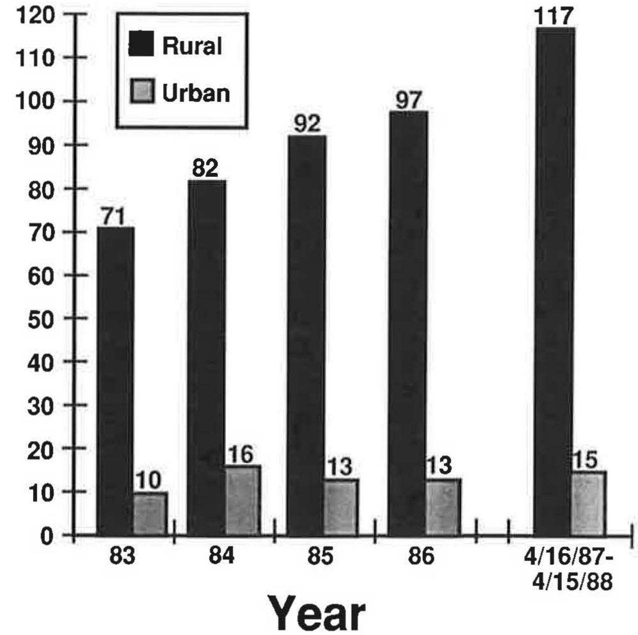

FIGURE 13 Interstate fatal accidents, urban versus rural.

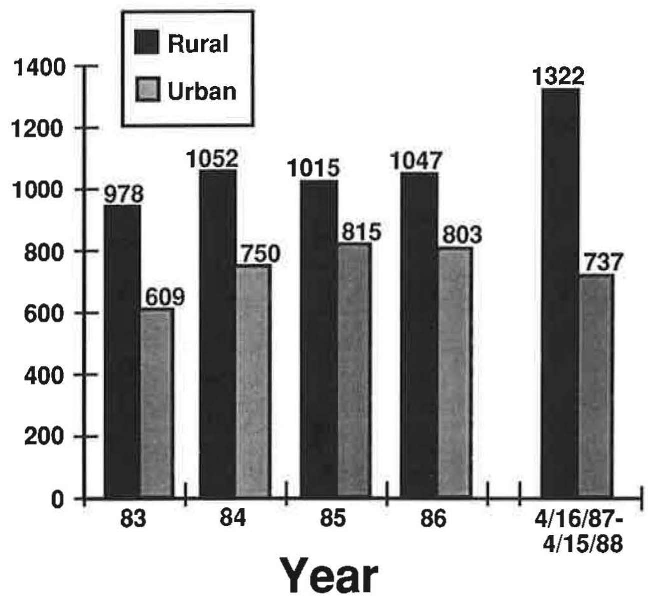

FIGURE 14 Interstate injury accidents, urban versus rural.

remained fairly constant on the urban interstate. During the same time periods, rural fatal accidents increased during the before years and increased after the speed limit was raised. Although urban injury accidents exhibited a slight decline after the increase in the speed limit, rural injury accidents increased.

CONCLUSIONS

1. Actual speeds driven by motorists on Arizona's rural interstate stayed almost constant during the 3 years before the speed limit was increased.
2. Actual speeds driven increased by only about 3 mph or less during the four quarters after the increase in the rural interstate speed limit.
3. There is slightly more dispersion in vehicle speeds now than there was before the speed limit was increased.
4. The number of accidents on the urban interstate changed very little during the 3 years before and the 1 year after the speed limit was increased on the rural interstate.
5. The accident rate on the urban interstate was on the decline beginning in 1984 and continuing through the 1-year-after period.
6. The number of accidents on the rural interstate increased after the speed limit was increased.
7. The accident rate on the rural interstate increased for total accidents and for injury accidents when the 1-year-after

period was compared with that for 1986. However, the accident rate was approximately the same as that for 1984.

8. The fatal accident rate on the rural interstate was higher in the 1-year-after period than in any of the years between 1983 and 1986.

9. The information presented in this paper does not prove or disprove a cause and effect relationship between actual speeds driven and accident experience. Many other factors—including factors not addressed in this paper, such as seat belt use, alcohol involvement, and weather conditions—have an influence on accident experience.

ACKNOWLEDGMENT

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The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Arizona Department of Transportation.

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Evaluation of Unmanned Radar Installations

JERRY G. PIGMAN, KENNETH R. AGENT, JOHN A. DEACON, AND RICHARD J. KRYSCIO

Several unmanned radar devices were installed on Interstate 75 in northern Kentucky in an attempt to reduce speeds. It was assumed that drivers use radar detectors to exceed the speed limit thus causing a variance between their speeds and those of others in the traffic stream. Since historical data indicated an unusually high accident rate for the study area, a reduction in overall speeds and variance was expected to reduce the probability of accidents. The high accident rate also resulted in a plan to reduce trucks on I-75 in the study area by diverting them onto a bypass route (I-275). Emphasis was placed on collection and analysis of speed-related data. In addition, a survey of radar detector use was made and accident patterns were documented. Speed measures analyzed included mean speed, standard deviation in speed, numbers of vehicles exceeding specified speed levels, and 85th percentile speed. Results indicate that unmanned radar was an effective means of reducing the number of vehicles traveling at excessive speeds. The differences in mean speeds were small and the impact of unmanned radar was less obvious than it was for the percentage of vehicles exceeding speed levels of 65, 70, 75, and 80 mph. The speeds of vehicles with radar detectors decreased significantly as a result of unmanned radar, whereas the speeds of vehicles without detectors were not affected. Radar detector use was found to be 42 percent in trucks and 11 percent in cars. When comparing accident data 3 years before and 1 year after truck diversion and unmanned radar installations, there was a reduction in truck-related and speed-related accidents.

In an attempt to improve safety by reducing speeds on Interstate 75 (I-75) in northern Kentucky, five unmanned radar units were installed in the summer of 1986. These units remained on for approximately 3 months and were then turned off after the Federal Communications Commission ruled that unmanned radar transmitters were in violation of their regulations. In the fall of 1986, legislation was passed by Congress that exempted a short section of I-75 in northern Kentucky from Federal Communications Commission requirements (*1*). This legislation mandated that a demonstration project be conducted to assess the benefits of continuous use of unmanned radar equipment. After the legislation was signed by the President on October 27, 1986, plans were made for conducting the demonstration project. As a result of a meeting in Frankfort, Kentucky, on December 21, 1986, between representatives of the Kentucky Transportation Cabinet, FHWA, and the Federal Communications Commission, the units were turned on again.

Preliminary plans were made for an evaluation study to be performed by the University of Kentucky's Transportation Research Program, in cooperation with the Kentucky Department of Highways and FHWA. Additional radar units were installed in the spring of 1987, with all except one unit operational by June 11, 1987. The last unit to be installed began operating in early August 1987. The study area was divided into two sections of radar signal coverage as shown in Figure 1. The full coverage area included nine radar units and extended from Milepoint (MP) 187.2 at 0.5 mi south of the Ft. Mitchell interchange to Milepoint 191.2 at the Ohio River. The partial coverage area included six units and extended from Milepoint 178.2 (about 1 mi south of Florence) to Milepoint 187.2. In the partial coverage area, the radar units were spaced intermittently; however, there were approximately equal distances (4.5 mi) from which the radar signal could and could not be received with a radar detector. The radar units were installed for northbound traffic; however, the signal also could be received by southbound traffic. Throughout the period from the first unmanned radar installations until the study was completed, significant media attention was given to the project. This attention created some problems with respect to vandalism; however, the overall data collection effort was not adversely affected.

STUDY AREA CHARACTERISTICS

The section of I-75 in northern Kentucky covering a length of approximately 4 mi from Ft. Mitchell to the Ohio River has been noted for its exception to the general interstate guidelines for grade and curvature. Most of I-75 in the study area was constructed in the early 1960s, and the problems associated with excessive grade and curvature in an urban area have been documented since. Parts of the study area have grades of 5 percent (downgrade for northbound traffic) and curves of 6 degrees. In 1971, a congressional subcommittee held a public hearing in Covington, Ky., to discuss the hazardous nature of that section of I-75. Soon afterwards, the Department of Highways' Division of Research conducted an evaluation of various safety features that had been installed on that section of I-75, and the results indicated a reduction in accidents (*2*). Other improvements have been made over the years, but the positive impact of improved safety generally has been offset by an increased volume of traffic and resulting congestion. Another recent change in an attempt to improve safety was the diversion of through trucks onto the I-275 circle route around Cincinnati, Ohio (started on July 8, 1986).

The section between Ft. Mitchell and the Ohio River has six lanes of through traffic and carries the highest volumes of any roadway in Kentucky. Average daily volumes for this

Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, Ky. 40506.

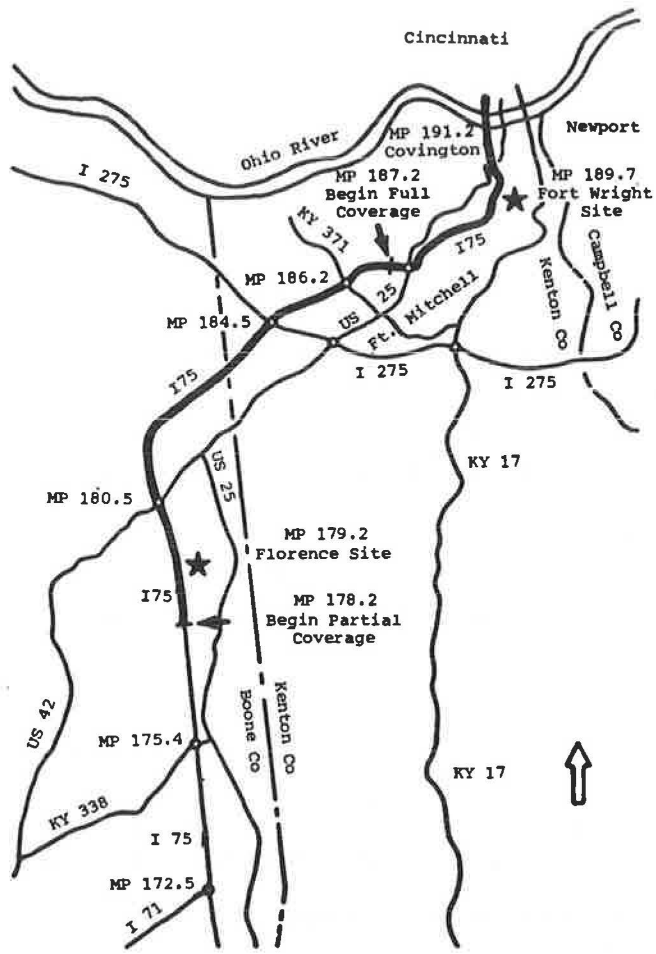

FIGURE 1 Map showing significant points in study area.

section are in the range of 120,000 vehicles. This compares to an annual average daily traffic (AADT) volume of about 60,000 at Florence, which is approximately 10 mi south. For northbound traffic, the percentage of trucks ranged from approximately 26 percent just south of the I-275 interchange to 9 percent in Covington.

The speed limit on I-75 is 55 mph in the southern part of the study area and changes to 50 mph for cars at Milepoint 188.0, 0.3 mi north of the Ft. Mitchell (U.S. Route 25) interchange (Figure 1). In the area of a 50-mph speed limit for cars, the limit for trucks is 45 mph. The breakpoint for change from the 65-mph speed limit (effective June 8, 1987, for rural interstates in Kentucky) to 55 mph is at the KY 338 interchange (MP 175.4), just south of the study area.

RELATIONSHIP BETWEEN SPEED AND SAFETY

Speed has been determined to be one of the most common contributing factors in vehicular accidents. In Kentucky, speed is listed as a contributing factor in 8.9 percent of all accidents and 36.7 percent (the most frequently cited factor) of fatal accidents (*3*). Consideration of speed presents a dilemma in highway transportation because it affects both safety and efficiency. The basic relationship between speed and stopping distance indicates that stopping distance increases in relation to the square of the speed, and the result can be a higher accident potential. Conversely, increased speed can reduce travel costs and increase the operating efficiency of a highway.

The relationship between speed variance and safety has been investigated, and it has been shown that the greater the variation in speeds, the higher the probability of an accident, assuming equal exposure (*4,5*). Another study examined speed variance, and it was found that both slow drivers and fast drivers had accident rates that were approximately six times that of drivers operating close to the mean traffic speed (*6*).

It also has been documented that the greater the absolute speed, the greater the likelihood of increased accident severity (*7*). The energy dissipated during a collision is directly proportional to the vehicle's weight and to the square of its speed. Therefore, increased speed results in more energy dissipation, which translates into greater damage to the vehicle and more injuries to the occupants.

The question of whether the use of radar detectors results in increased accidents remains unanswered. Insufficient research has been conducted to address the issues that are necessary for proper evaluation. Those issues include (a) socioeconomic characteristics of drivers using radar detectors as compared with those of the normal driving population, (b) accident rates based on exposure by type of highway, and (c) overall safety and handling characteristics of vehicles in which radar detectors are used.

EFFECT OF ENFORCEMENT ON SPEED

The presence of police enforcement has been shown to have the effect of decreasing speeds (*8,9*). The use of speed enforcement, a speed-check zone, or a parked patrol vehicle produced significant reductions in speeds in the vicinity of the enforcement unit in another study (*10*). Increased police enforcement in work zones has produced positive effects in terms of speed reduction (*11*). Active police enforcement in conjunction with the use of radar units has been used in many situations to reduce speed.

DATA COLLECTION

Several types of data were collected in an attempt to evaluate the impact of unmanned radar installations on speed. In addition to speed-related data, a survey of radar detector use was made and historical accident patterns were documented.

Automatic Speed Data

Speed data were collected by using automated equipment connected to loops embedded in the roadway at two locations. The speed monitoring station at Ft. Wright (MP 189.7), installed specifically to collect data for this study, became operational on July 6, 1987. Data were collected for approximately 70 days, with some gaps, through November 1, 1987. During the period of data collection, each of the three northbound lanes of I-75 were monitored separately and data for a sample of 2,180,512 vehicles were collected with radar on and for 1,576,615 vehicles with radar off.

The second speed monitoring station was located at Florence (MP 179.2), approximately 10.5 mi south of the Ft. Wright location. This site is among those included in the 55 mph Compliance Speed Monitoring Program of the Kentucky Department of Highways. Data collection was limited to an 18-day period in October and the sample size was 236,471 vehicles with radar on and 266,267 vehicles with radar off.

Manual Speed Data

Manual speed data were collected to supplement the automatic data so that speed data could be collected at additional points in the study area. Data were collected by using time-distance methods (stopwatch measurements over a pre-selected distance) rather than by radar to ensure that radar signals would not be present in the radar-off condition. Data were collected by three observers at four locations in the study area between June 11, and August 27, 1987. A sample of 150 vehicles was collected for each of the three lanes on each of 15 days. Some efforts were made to monitor citizens band radio transmissions to determine the extent of changes in driving behavior by making observations during the manual data collection effort.

The sample size of 150 vehicles in each of the three lanes of travel was sufficient to ensure, at the 95 percent confidence level, that estimates for the mean speed were statistically reliable within ± 1.0 mph. The procedures for determining sample size were obtained from the publication titled *Manual of Traffic Engineering Studies*, published by the Institute of Transportation Engineers (*12*).

Speed Data With and Without Radar Detectors

A determination was made that, in addition to automatic and manual speed data, it would be desirable to determine the speeds of individual vehicles and also be able to note the presence of radar detectors in those vehicles. This type of data was collected at the Ft. Wright speed monitoring location with the speed-classifier unit used to determine speed and the presence of radar detectors determined by visual inspection. Inherent problems with visual inspections were recognized; however, it was felt that a high percentage of detectors on the dash or windshield of a vehicle were noted. Vehicles with detectors mounted in the grille area obviously could not be seen. An observer was stationed on the side of the road at the speed-classifier unit so that speeds of vehicles could be noted the same time that detectors were observed. Data were collected on 14 days between September 1 and November 19, 1987. Total samples were 1,223 with radar off and 2,074 with radar on.

Speed Data With and Without Police Enforcement

In an attempt to assess the impact of police enforcement on speeds in the study area, additional data were collected with radar on and radar off in the vicinity of the Ft. Wright speed monitoring station. The Kentucky State Police cooperated in this effort, and data were collected on October 21 with radar on and October 28 with radar off. There were three hours of active enforcement on each day.

Radar Detector Data

Samples of data were collected throughout the study period to determine the percentages of vehicles in the I-75 corridor with visible radar detectors. The samples of cars were collected manually by observers as they were traveling on I-75 from Lexington to northern Kentucky. Visual observations were made as they passed or were passed by other vehicles. It also was recognized that some vehicles have built-in detectors that are not visible to observers positioned in another vehicle. Approximately half of the data for cars were collected without distinguishing whether they had in-state or out-of-state licenses. In the second part of the data collection, a distinction was made.

Additional radar detector data were collected by the Kentucky Transportation Cabinet's Division of Motor Vehicle Enforcement. These data were collected as part of vehicle-driver safety inspections (at the truck weight station on I-75 in Scott County), during which truck cab interiors were checked and the presence of radar detectors was noted.

Accident Data

Accident data were obtained from the Department of Highways' Division of Traffic and analyzed for the period between July 1, 1983 and June 30, 1987. This period included 3 years before the initial radar installations in the summer of 1986 and 1 year during which radar was on part of the time and trucks were being rerouted. The accident data were collected for two sections of I-75 as shown in Figure 1—one section representing the area between MP 175.4 (the KY 338 interchange) and MP 187.7 (the Ft. Wright interchange) and the other for the section between MP 187.7 and MP 191.7 (the Ohio River bridge). These sections represent contrasting conditions in terms of geometrics and volume levels. The section between MP 175.4 and MP 187.7 is relatively straight and level with AADTs in the range of 50,000 to 60,000. By contrast, the section starting at MP 187.7 and continuing to the Ohio River at MP 191.7 is the area of sharp curvature and steep grades with AADTs in excess of 100,000.

ANALYSIS OF DATA

Automatic Speed Data

Highway safety researchers generally agree that the safest traffic conditions include those in which vehicles travel at uniform speeds and those in which excessive speeding is minimized. Since any likely impact of radar on safety stems from its effect on speed, measures of primary interest to this study included those that measure both lack of uniformity—i.e., speed variability—and those that measure excessive speeding—i.e., the fractions of vehicles in the traffic stream exceeding stipulated speeds. Speed levels chosen for analysis herein included several at the high end of the speed spectrum, namely, 65, 70, 75, and 80 mph. Other speed measures chosen for analysis included the mean speed and the 85th percentile speed, two measures often examined by traffic engineers in speed studies. The statistical procedure used to analyze these data depended on the speed measure of interest as well as how other factors affecting these speed measures were treated.

The major hypothesis under examination herein is that radar signals can beneficially affect these speed measures, reducing both variability and level of speeds. To test this hypothesis, speed measurements were taken on I-75 during both radar-on and radar-off conditions. Unfortunately, simple differences between these two conditions may be quite misleading: many factors affect speeds and it is imperative to ensure that the analysis is conducted to isolate the effects of radar from those of such other factors.

Factors potentially affecting speed that were controlled in the collection of the automatic data included radar (on or off), day of week (weekday or weekend), light condition (daylight or darkness), and lane of travel (median, center, or shoulder). Unfortunately, other variables possibly affecting speed, such as amount of truck traffic and amount of precipitation, could be neither measured nor controlled. Since data were collected over a sufficiently long interval, the potential confounding effects of these other variables was considered to be small enough to be treated as part of measurement error. An effect not thought to be minimal, however, is that due to volume. That speeds are reduced by the congestion of increased volume levels is an established fact. Since it cannot be controlled in the sense that the above factors can be controlled, volume is treated as a covariate in the analysis of mean speeds and variability of speeds described below.

For the mean speed, the analysis considers the experiment to be a 2^3 factorial (factors: radar, day, and light) with repeated measures (the three lanes of traffic) each with a separate covariate (volume of vehicles in a given lane). The unit of analysis was the mean speed for 1 hour of observation. Evaluation of such an experiment requires an analysis of covariance procedure for a split plot experiment with a covariate for each unit in the split plot (lanes). Because of the size of the data base and the number of factors and their levels, separate analyses were performed for each lane of travel.

Variance of vehicle speeds, a second speed measure computed for each hour of observation, is not normally amenable for investigation by using analysis of covariance techniques because variances are distributed as chi-squared variates and not normal variates. However, for large sample sizes, the chi-squared distribution is well approximated by the normal distribution. Because speeds were measured for a large number of vehicles during each hour of data collection, it was assumed that variance could be treated as a normal variate and that standard analysis of covariance routines could be used for analyzing variance of speed as well as for its mean.

Excessive speeding was measured by the proportions or numbers of vehicles exceeding certain high speed levels. At very high levels, use of the standard analysis of covariance technique becomes suspect because of the small numbers of vehicles involved. An alternate statistical procedure, attributed to Campbell (*13*), is available, however, and is not constrained by the small numbers or proportions of affected vehicles. This procedure, adopted for the analysis herein, treats traffic volume not as a covariate but as a factor similar to day of the week and lane of travel. Although effects of radar can be accurately assessed, the Campbell procedure does not allow analysis of the statistical significance of interactions among the experimental factors.

Manual Speed Data

Data collected with radar on and radar off were separated, and all data for each condition were combined. Using the combined data, the average speed and standard deviation were calculated as well as the percentage of vehicles exceeding 55, 60, 65, and 70 mph. The *t*-test was used to test the statistical significance of the differences in the mean speeds and the *F*-test was used to test differences in standard deviations (*14*).

Speed Data With and Without Radar Detectors

Speeds of vehicles with and without radar detectors were summarized as a function of whether the radar was on or off. For each set of data, the average speed and standard deviation were calculated as well as the percentages of vehicles exceeding 60, 65, 70, and 75 mph. An analysis of variance procedure, with appropriate contrasts, was used to compare mean speeds between the four conditions formed by the combinations of the factors of radar on and off and cars with and without detectors. Bartlett's procedure was used to compare the variability of speeds between these four conditions, and a contingency table analysis was used to compare the proportion of vehicles exceeding 60, 65, 70, and 75 mph between these four conditions.

Speed Data With and Without Police Enforcement

The data used for evaluating the impact of police enforcement on speeds with radar on and radar off consisted of 3 hours of data during each of the conditions. Time periods for data collection were limited because of the availability of enforcement personnel; however, the total sample of vehicles included in each 3-hour period was approximately 8,000. These data were combined into four sets representing (a) active enforcement—radar off, (b) no enforcement—radar off, (c) active enforcement—radar on, and (d) no enforcement—radar on. The combined sets of data were compared statistically by calculating the mean speed, standard deviation, and percentages of vehicles exceeding 65, 70, 75, and 80 mph. The *t*-test was used to test for statistical differences in mean speeds and the chi-squared test was used to determine if differences in the number of vehicles exceeding the speed levels of 65, 70, 75, and 80 mph were different (*13*).

Accident Data

The data were summarized into two location categories and two time categories. The location categories were (a) from the KY 338 interchange to the Ft. Mitchell (U.S. Route 25) interchange and (b) from the Ft. Mitchell interchange to the Ohio River. The time periods were the 3-year period from July 1, 1983 to June 30, 1986, before the start of the unmanned radar and the truck diversion and the 1-year period of July

1, 1986, through June 30, 1987. For each category, the total number of accidents per year and the accident rate were calculated along with the percentages of accidents involving trucks, injuries or fatalities, speed as a contributing factor, darkness, and a wet or snowy pavement.

RESULTS

Automatic Speed Data

A comparison of the mean speeds at the Ft. Wright and Florence speed monitoring stations is presented in Table 1. Specifically, Table 1 gives the mean speeds at each station with radar on and with radar off for each lane of traffic under all other conditions by type of day (weekday and weekend) and by type of light (daylight and darkness). Mean speeds were computed by first regressing average speed on traffic volume for each hour of study via an analysis of covariance and then computing the predicted mean speed at the average level of traffic volume in the resulting regression equation. These adjusted mean speeds were next compared by using the analysis of covariance, and the corresponding *P* values.

At the Ft. Wright station, the adjusted mean speeds for both the median and center lanes with radar on were lower than the corresponding adjusted mean speeds with radar off for each type of condition listed above. None of these differences was determined to be statistically significant. Although the adjusted mean speeds were not consistently lower in the shoulder lane when radar was on, there was no statistically significant difference between adjusted mean speeds when radar-off and radar-on speeds were compared for this lane.

At the Florence station the use of the unmanned radar installation produced significantly lower mean speeds with radar on when compared with radar-off speeds for all three lanes of traffic. According to Table 1, the effect of radar varied by the day of week, with radar producing a larger reduction in speeds on weekends for all three lanes. The effect of radar also varied with the type of light, with radar producing a larger reduction in speeds at night for both center and shoulder lanes.

Adjusted mean speeds at the Florence station were higher than at the Ft. Wright station, which was expected because of the lower speed limit, higher traffic volumes, and restricted roadway geometrics at the Ft. Wright station. The speed limit at Florence was 55 mph compared with 50 mph for cars and 45 mph for trucks at Ft. Wright. AADTs at Florence were in the range of 50,000 to 60,000 compared with 100,000 to 120,000 at Ft. Wright. In addition, roadway geometrics at Florence were generally straight and level compared with relatively sharp curves and steep grades at Ft. Wright.

A comparison of the actual and expected number of vehicles traveling above various speeds is shown in Table 2. The actual number of vehicles was the number of vehicles traveling above the given speed with radar on. This number was compared with an expected number of vehicles traveling above a given speed, which was calculated by using the data obtained with radar off. Essentially, data collected under radar-off conditions were adjusted so that the proportion of total observations occurring within each elemental analysis unit was identical to that occurring under radar-on conditions. Each speed measure, so adjusted, is considered to be the expected value in the absence of radar: it is compared with the actual value measured with radar on to identify the most likely effects of the radar.

The data in Table 2 show what was found to be a statistically significant decrease in vehicles traveling above the high speeds of 65 to 80 mph at both locations. This decrease was greater at Florence than at Ft. Wright, which is logical since the speeds at the Florence station were higher. The traffic volume at the Florence station was about one-half that at Ft. Wright. The high traffic volume combined with the restrictive roadway geometrics at Ft. Wright could result in a greater safety benefit from the reduction in excessive speeding than at Florence, even though fewer vehicles were affected. Daily reductions in the number of vehicles exceeding the various speeds are listed. The reductions per day vary from 2,199 exceeding 65 mph at the Florence station to 6 exceeding 80 mph at Ft. Wright.

A comparison of the actual and expected number of vehicles traveling above various speeds was made as a function of lane. The reductions in speed were generally highest in the median lane at Florence, whereas the reductions were generally highest for the shoulder lane at Ft. Wright. There were reductions in each lane at both locations, with all the differences determined to be statistically significant.

The differences in actual and expected number of vehicles traveling above various speeds, as a function of the day of

TABLE 1 ADJUSTED MEAN SPEEDS FROM ANALYSIS OF COVARIANCE

Variable	Category	Median Lane Radar On	Median Lane Radar Off	Center Lane Radar On	Center Lane Radar Off	Shoulder Lane Radar On	Shoulder Lane Radar Off
Florence							
All	All	64.50	66.36	62.06	63.72	57.15	58.61
Day of week	Weekday	65.07	66.45	62.52	63.79	57.41	58.58
	Weekend	63.93	66.28	61.60	63.65	56.90	58.64
Light	Daylight	65.42	67.27	63.11	64.45	57.75	58.88
	Darkness	63.58	65.46	61.01	62.99	56.56	58.34
Ft. Wright							
All	All	62.82	62.98	57.85	57.88	54.57	54.46
Day of week	Weekday	62.74	62.91	57.71	57.77	53.58	53.52
	Weekend	62.89	63.05	57.99	58.00	55.56	55.40
Light	Daylight	64.26	64.40	59.01	59.11	55.65	55.48
	Darkness	61.38	61.56	56.69	56.66	53.48	53.44

NOTE: Mean speeds are adjusted to the average level of traffic volume in the lane.

the week, were analyzed. There was a larger reduction in excessive speeds on the weekend at Florence than on weekdays; no such difference was detected at Ft. Wright. All reductions were statistically significant.

The differences in actual and expected number of vehicles traveling above various speeds, as a function of light condition, were also analyzed. At Florence, the reductions during darkness were slightly higher than those during daylight. There were no substantial differences between daylight and darkness at Ft. Wright. All of the differences were statistically significant.

Comparisons were made of actual and expected numbers of vehicles above various speeds as a function of traffic volume. There were reductions in every category, and almost all were statistically significant; however, no trend was detected in which the reductions could be related to traffic volume.

A comparison of the variation of speeds at the two stations is presented in Table 3. This table includes the adjusted standard deviations of speeds at each station with radar on and with radar off for each lane of traffic and for various combinations of radar with the type of day and type of light. At the Ft. Wright station the adjusted standard deviation of speeds with radar on (4.97) in the median lane is significantly lower than the corresponding standard deviation with radar off (5.08); the standard deviation with radar on (4.66) in the center lane is significantly lower than the corresponding standard deviation with radar off (4.79). For the shoulder lane the adjusted standard deviation with radar on is significantly lower than the standard deviation with radar off for weekdays but not weekends or for daylight but not darkness. For both the center and shoulder lanes the adjusted standard deviation of speeds was significantly higher on weekdays as opposed to weekends and during daylight as opposed to darkness.

At the Florence station, similar results were obtained for the effect of radar in that the adjusted standard deviation of speeds was significantly lower when radar was on compared with when radar was off for both the center and shoulder lanes. For the median lane there was a significant radar-by-light interaction. The effect of light is different at the Florence station, with darkness producing more variable speeds for the median lane, fewer variable speeds for the shoulder lane, and no significant effect for the center lane.

The 85th-percentile speed is a measure commonly used to describe variation in traffic speeds. A summary of the actual and expected 85th percentile speeds at the Ft. Wright and Florence stations for the various categories is presented in Table 4. The actual speeds with radar on were lower than the expected speeds, using the radar-off data, for every category. The differences, although small, were larger than those found

TABLE 2 RADAR EFFECTS ON NUMBER OF VEHICLES ABOVE VARIOUS SPEEDS

		No. Over Speed		Percent Over Speed		Percent Reduction Because of Radar	No. Over Speed per Hour[c]		Reduction per Day
Location	Speed	Radar On (Actual)[a]	Radar Off (Expected)[b]	Radar On (Actual)	Radar Off (Expected)		Radar On (Actual)	Radar Off (Expected)	
Florence	80	751	1,265	0.32	0.53	40.6	3.5	5.0	36
	75	2,336	4,396	0.99	1.86	46.9	11.0	20.8	234
	70	11,954	19,828	5.06	8.38	39.7	56.5	93.7	894
	65	55,631	75,023	23.53	31.73	25.8	262.8	354.5	2,199
Ft. Wright	80	983	1,240	0.05	0.06	20.6	1.0	1.3	6
	75	5,018	6,228	0.23	0.31	25.8	5.2	6.5	31
	70	44,940	50,668	2.07	2.53	18.2	46.8	52.8	144
	65	258,991	273,301	11.90	13.42	11.3	269.7	284.6	358

NOTE: All differences were significant at the 0.05 level.
[a]Actual number of vehicles recorded above given speed with radar on.
[b]Expected number of vehicles above given speed using data obtained with radar off.
[c]Based on number of hours of data obtained with radar on (635 lane-hours at Florence and 2,881 lane-hours at Ft. Wright).

TABLE 3 STANDARD DEVIATION OF SPEED FROM ANALYSIS OF COVARIANCE

		Median Lane		Center Lane		Shoulder Lane	
Variable	Category	Radar On	Radar Off	Radar On	Radar Off	Radar On	Radar Off
Florence							
All	All	5.52	5.82	5.38	5.51	5.41	5.58
Day of week	Weekday	5.57	5.60	5.35	5.47	5.31	5.48
	Weekend	5.48	6.02	5.42	5.55	5.51	5.68
Light	Daylight	5.38	5.36	5.41	5.44	5.55	5.65
	Darkness	5.67	6.24	5.36	5.57	5.28	5.51
Ft. Wright							
All	All	4.97	5.08	4.66	4.79	6.02	6.08
Day of week	Weekday	4.95	5.08	4.71	4.83	6.27	6.39
	Weekend	4.99	5.08	4.61	4.74	5.76	5.76
Light	Daylight	4.82	4.91	4.71	4.80	5.93	6.05
	Darkness	5.12	5.24	4.62	4.77	6.11	6.12

NOTE: Mean variances of speed are adjusted to the average level of traffic volume in the lane. Standard deviations reported above are square roots of the adjusted mean variances.

for the mean speeds at the Ft. Wright station. The differences were larger at Florence than at Ft. Wright and were similar to those found for the mean speeds. No statistical analyses were performed to compare the 85th percentile speeds.

Manual Speed Data

The manual data collected at the four locations were summarized and included average speed, standard deviation, and the percentage of vehicles exceeding various speeds. Statistical tests indicated that none of the differences in average speed was significant. There was no general trend in the speeds with radar on or radar off at two locations. Speeds at one location were lower with radar on. The results show that the sample of speed data collected manually was apparently insufficient to include all the conditions that would identify differences expected by time of day, day of week, light conditions, and traffic volumes.

Speed Data With and Without Radar Detectors

The summary of speed data for vehicles with and without a radar detector is presented in Table 5. The data also are summarized with radar on and radar off. All data were collected in the median lane at the Ft. Wright speed monitoring station. The analysis showed that, when the radar was off, the percentage of vehicles with a speed over specified high speeds was higher for vehicles with radar detectors. Conversely, when the radar was on, the percentage of vehicles with speeds over these high speeds was higher for vehicles without a radar detector. It is also interesting to note the reduction in the percentage of vehicles with detectors traveling above these speeds when the radar was on. For example, the percentage of vehicles exceeding 65 mph was about 36 percent for vehicles with radar detectors during radar-off conditions, and this percentage decreased to about 20 percent during radar on conditions. Conversely, this percentage did not change for vehicles with no radar detector, with 28 percent during radar-off and 27 percent during radar-on conditions.

TABLE 4 RADAR EFFECTS ON 85TH PERCENTILE SPEED

	Ft. Wright		Florence	
Variable	Radar On (Actual)	Radar Off (Expected)	Radar On (Actual)	Radar Off (Expected)
All	65.41	65.55	67.31	68.58
Day of week				
Weekday	64.14	64.28	67.47	68.62
Weekend	64.79	64.93	66.73	68.47
Lane				
Median	67.68	67.88	69.44	71.27
Center	62.21	62.39	67.77	68.91
Shoulder	59.60	59.63	63.01	64.04
Light conditions				
Daylight	64.46	64.61	67.74	68.88
Dark	63.69	63.85	65.81	67.61
Traffic volume (vehicles per hour)				
<300	64.22	64.45	67.82	69.14
300–599	64.44	64.61	66.46	67.93
600–899	64.40	64.50	67.76	68.90
900–1,200	65.39	65.68	68.15	68.91
Over 1,200	63.36	63.48	—[a]	—[a]

[a]There were no data in this traffic volume category.

TABLE 5 RADAR EFFECTS ON SPEEDS OF VEHICLES WITH AND WITHOUT DETECTORS

	Radar Off		Radar On	
	With Detector	No Detector	With Detector	No Detector
Sample size	132	1,091	121	1,953
Average speed (mph)	64.64	63.57	62.60	63.49
Standard deviation	4.64	4.21	3.74	4.02
Percent speeds over 60 mph	81.8	79.9	71.9	80.4
Percent speeds over 65 mph	36.4	27.7	19.8	26.7
Percent speeds over 70 mph	10.6	5.0	4.1	4.1
Percent speeds over 75 mph	2.3	1.0	0.0	0.9

NOTE: All data were taken in the median lane at Ft. Wright speed monitoring station.

A comparison of mean speeds between the four conditions given in Table 5 using a one-way analysis of variance F-test, indicated statistically significant differences in the means. These data show that, although mean speeds decreased significantly for cars with detectors when comparing radar-off and radar-on conditions (64.64 mph compared with 62.60 mph), mean speeds did not change significantly for cars without detectors (63.57 mph compared with 63.49 mph). With radar off, the average speeds of vehicles with detectors were higher than vehicles without detectors (64.64 mph compared with 63.57 mph); conversely, with radar on, the average speeds of vehicles without detectors were higher than vehicles with detectors (63.49 mph compared with 62.60 mph).

The change in the variability of speeds can be shown in the standard deviations. A comparison between the standard deviation of speeds under the four conditions given in Table 5 was made using Bartlett's statistic ($P < 0.05$). The data show that the variability of speeds was decreased significantly under the radar-on condition for vehicles with radar detectors as well as for those without detectors. For vehicles with radar detectors, the standard deviation decreased substantially (4.64 compared with 3.74) as a result of radar. When the radar was off the standard deviation of speeds of vehicles with detectors was higher than those without detectors (4.64 compared with 4.21); when the radar was on, the standard deviation of speeds of vehicles without detectors was higher than those with detectors (4.02 compared with 3.74). These data show that the variability of speeds was decreased under the radar-on condition, especially for vehicles with radar detectors.

Speed Data With and Without Police Enforcement

The effect of active enforcement show that both the mean speeds and the percentages of vehicles exceeding various speeds were reduced as a result of active police enforcement. These reductions occurred both with radar on and radar off. The reductions in mean speed and the percentage exceeding 65 mph and 70 mph were determined to be statistically significant. There were greater reductions in mean speeds and percentage of vehicles exceeding 65 and 70 mph for radar-on conditions compared with radar-off conditions. For example, the reduction in percentage of vehicles exceeding 65 mph was 48 percent with and without active enforcement for radar off compared with 65 percent with and without active enforcement for radar on.

Radar Detector Data

A sample of 318 trucks was inspected by the Division of Motor Vehicle Enforcement during its regular inspection activities at the Scott County weigh station on I-75 between May 15 and June 1, 1987. A visual inspection of the truck cab interiors revealed that 135, or 42.4 percent, of the trucks had radar detectors.

Observations of the number of vehicles with visible detectors were conducted on 14 days between June 2 and August 22, 1987, on I-75 during trips between Lexington and northern Kentucky. A sample of 768 cars between June 2 and July 30 showed that 66, or 8.6 percent, had radar detectors. Another sample between August 4 and August 22 classified the cars as in-state and out-of-state vehicles. There was very little difference between in-state and out-of-state cars, with 13.5 percent (55 of 406) in-state cars and 12.9 percent (55 of 426) out-of-state cars having radar detectors. Combining all the data yielded 11.0 percent of cars with detectors.

Accident Analyses

A summary of the analysis of accident records is presented in Table 6. The summary for the 12.3-mi section between the KY 338 interchange and the Ft. Mitchell (U.S. Route 25) interchange was tabulated separately from the 4.1-mi section between the Ft. Mitchell interchange and the Ohio River. The section between KY 338 and Ft. Mitchell had an ADT of about 82,000 over the 4-year study period compared with about 102,000 for the section between Ft. Mitchell and the Ohio River. During the time covered by the radar experiment, there was basically full radar coverage of the section between Ft. Mitchell and the Ohio River and partial coverage for the other section.

The number of accidents and the accident rate were much higher for the section between Ft. Mitchell and the Ohio River. The accident rate for this section during the 3 years before truck diversion and initial radar installations was 245 accidents per 100 million vehicle miles (MVM). This figure was higher than the statewide average of 156 accidents per 100 MVM and a 3-year critical rate of 171 accidents per 100 MVM for urban interstates. Critical rates for various types of highways in Kentucky were determined as part of other research (*3*). The accident rate for the section between the KY 338 and Ft. Mitchell interchanges was much lower (a rate of 42 accidents per 100 MVM during the 3 years before truck diversion and radar installations). Although this section of I-75 is classified as an urban interstate, some parts are more representative of a rural interstate. The average rate for rural interstates is 69 accidents per 100 MVM, and for similar urban interstates the rate is 156 accidents per 100 MVM.

The data were summarized for a 3-year period before July 1986 and a 1-year period after that date. That date coincided with a diversion of northbound trucks from I-75 onto I-275 and also represents the approximate date when the unmanned radar was started. Both of these factors could have the potential for affecting accidents within the northbound lanes in the July 1986 through June 1987 time period. Also, the impact should be most obvious on the section between Ft. Mitchell and the Ohio River since both factors would apply to the total length of this section. However, only a portion of the section between the KY 338 and Ft. Mitchell interchanges would be affected.

A comparison between the two roadway sections and two time periods showed that the major change was on the section between Ft. Mitchell and the Ohio River. Specifically, the accident rate was reduced during the July 1986 to June 1987 time period. This decrease in the number of accidents, primarily in the northbound direction, was shown to be related to a reduction in the number of truck accidents, which was also related to the truck diversion. There was also a reduction in the percentage of speed-related accidents for northbound traffic in this section, which could be related to the unmanned radar.

TABLE 6 ACCIDENT ANALYSIS

	KY 338-FT. MITCHELL		FT. MITCHELL-OHIO RIVER	
	7/1/83 - 6/30/86	7/1/86 - 6/30/87	7/1/83- 6/30/86	7/1/86 - 6/30/87
Total Accidents	441	147	1,122	310
Accident/Year				
Total	147	147	374	310
Northbound	82	77	170	121
Southbound	65	70	204	189
Accidents/Mile/Year	120	120	91.2	75.6
Accident Rate(ACC/100 MVM)	42	40	245	204
Percent Truck Accidents				
Total	26.8	23.8	28.9	20.0
Northbound	26.1	23.4	27.6	16.5
Southbound	27.6	24.3	30.3	22.2
Percent Injury or Fatal Accidents				
Total	23.8	25.9	30.7	35.5
Northbound	22.4	23.4	31.2	32.2
Southbound	25.5	28.6	30.5	37.6
Percent Speed Related Accidents				
Total	10.9	6.8	8.0	7.4
Northbound	9.4	9.1	8.0	6.6
Southbound	12.8	4.3	8.1	7.9
Percent During Darkness				
Total	30.6	28.6	33.6	32.3
Northbound	29.0	31.2	26.0	31.4
Southbound	32.7	25.7	40.7	32.8
Percent on Wet or Snowy Pavement				
Total	33.6	22.4	30.6	18.7
Northbound	29.0	23.4	35.2	22.3
Southbound	39.3	21.4	28.5	16.4

SUMMARY AND CONCLUSIONS

The following is a summary of the major findings and conclusions from the analyses performed during this study.

1. At the Ft. Wright speed monitoring station, there was no statistical difference in mean speeds with radar on and radar off.
2. At the Florence speed monitoring station, data indicated that the mean speeds showed a statistically significant decrease with radar on.
3. At both speed monitoring stations, there were statistically significant reductions in the numbers of vehicles exceeding speed levels of 65 to 80 mph when radar-on (actual) and radar-off (expected) speeds were compared.
4. Unmanned radar was demonstrated to be an effective means of reducing the number of high-speed drivers. The reduction per day in numbers of vehicles exceeding the speed limit (55 mph) by 15 mph was determined to be approximately 900 at Florence compared with approximately 350 vehicles per day exceeding the speed limit (50 mph) by 15 mph at Ft. Wright.
5. The variability of speeds at the speed monitoring stations (as measured by the standard deviation) decreased with radar on compared with radar off.
6. The 85th percentile speeds were lower with radar on at the speed monitoring stations. The differences were small at the Ft. Wright station.
7. The manual data collection did not reveal any statistically significant differences when comparing mean speeds with radar-on and radar-off speeds. Results indicated that the sampling periods were apparently insufficient to include all conditions that might identify differences that were shown at locations where automatic equipment was used to collect continuous data.
8. About 42 percent of trucks and 11 percent of cars were observed to have radar detectors. There was no substantial difference in the percentage of in-state and out-of-state cars with radar detectors.
9. Speeds of vehicles with and without detectors for radar-on and radar-off conditions indicated that the use of radar detectors had a significant effect on vehicle speeds. With radar-on conditions, the speeds of vehicles with radar detectors decreased significantly compared with those with radar-off

conditions, whereas the speeds of vehicles without detectors were not affected by the radar. These data also indicated that the variability of speeds was decreased under the radar-on condition, especially for vehicles with radar detectors.

10. Active police enforcement was found to produce a statistically significant reduction in mean speeds and the percentage of vehicles exceeding various speeds for both radar-on and radar-off conditions. However, the effect was more pronounced with radar on.

11. Accidents in the northbound direction on I-75 between Ft. Mitchell and the Ohio River were found to have decreased in the 1-year period after July 1986 compared with the 3-year period before. This reduction was apparently related to the truck diversion and, possibly, the unmanned radar. There was a reduction in the percentage of truck-related and speed-related accidents for northbound traffic in this section.

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All-Way Stops: A New Policy

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This project was undertaken to develop a new all-way stop policy that would, with success and credibility, select intersections best suited to all-way stop controls. A variety of categories is considered by the new policy: accidents, unusual conditions, traffic volumes, and pedestrian volumes. Each category contributes points to a total that may, in sum, justify all-way stops for the intersection. Conversely, the circumstances within one category may be sufficiently extreme as to justify all-way stops based on that category alone. Existing all-way stop policies were determined to not be sufficiently flexible. The new policy combines the best features from national policies and the old City of San Diego policy. Also, the provisions within the new policy are derived from research and experience with all-way stops, not simply modifications of traffic signal warrants. The policy was tested by comparing accidents and field performance in a before-and-after study of existing all-way stop intersections. Some of these intersections met the all-way stop criteria in the new policy, whereas others did not. The study showed convincingly that the intersections that met the new policy's criteria had fewer accidents and stop sign violations than the intersections that did not.

San Diego, like many cities, has struggled with the issue of all-way stops for many years. The city receives many requests for all-way stops, which can be an emotional issue for some citizens. To many elected officials, a group of citizens requesting an all-way stop may themselves provide sufficient warrant to install an all-way stop, regardless of whether traffic engineering warrants have been met. Traffic engineers, however, want to be able to differentiate good all-way stop candidate intersections from bad ones through analysis of operational and safety factors. Part of the problem is that many engineers, in San Diego and elsewhere, are not comfortable with the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*) warrants.

A better all-way stop policy that is accepted and respected by both professionals and nonprofessionals will make it more likely for a confident engineering staff to successfully limit all-way stop installations to only those locations where the safety and operation of the intersection will improve with all-way stops.

Traffic Engineering Principles

The function of all-way stops is to control the right-of-way assignment at intersections. With all-way stops, vehicles on the intersecting streets alternate having the right-of-way. Therefore, all-way stops function best when the traffic volume at the intersection is high enough that vehicle conflicts are common and when the traffic volume is evenly split between the intersecting streets. All-way stops may also be effective at locations where there have been numerous correctable right-angle type accidents or where numerous unusual conditions exist.

It is neither wise nor practical to install all-way stops indiscriminately. On streets with frequent stops, motorists tend to drive at higher speeds to make up for the "lost time." Some motorists may even be tempted to disregard stop signs when there is no apparent "need" to stop because of cross traffic, pedestrians, or limited visibility. When motorists fail to obey stop signs, they are jeopardizing safety for themselves, other drivers, and pedestrians. Furthermore, the installation of unwarranted stop signs on major streets can create excessive queuing, delay, exhaust emission, fuel use, and noise.

PROBLEMS WITH EXISTING ALL-WAY STOP POLICIES

The MUTCD policy has three warrants. For an all-way stop to be justified, only one warrant must be met, but the warrant must be met in its entirety.

The MUTCD warrants the following (*1*):

1. Where traffic signals are warranted and urgently needed, the multiway stop as an interim measure that can be installed quickly to control traffic while arrangements are being made for the signal installation.
2. An accident problem, as indicated by five or more reported accidents in a 12-month period of a type susceptible to correction by a multiway stop installation. Such accidents include right- and left-turn collisions, as well as right-angle collisions.
3. Minimum traffic volumes:
   - The total vehicular volume entering the intersection from all approaches must average at least 500 vehicles per hour for any 8 hours of an average day; and
   - The combined vehicular and pedestrian volume from the minor street or highway must average at least 200 units per hour for the same 8 hours, with an average delay to minor street vehicular traffic of at least 30 seconds per vehicle during the maximum hour; but
   - When the 85th percentile approach speed of the major street traffic exceeds 40 mph, the minimum vehicular volume warrant is 70 percent of the above requirements.

There are numerous reasons to question the MUTCD policy. First, the MUTCD all-way stop policy is dependent on signals. Warrant 1 states that all-way stops may be used as interim measures before signal installation. Warrant 2 is a variation of Signal Warrant 6, and Warrant 3 is nearly identical to Signal Warrant 1.

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It is questionable to rely on an all-way stop policy derived from signals, not stop signs. The policy does not consider accidents or volumes when the numbers are below the specified thresholds. The MUTCD policy does not consider other factors that should be examined in an all-way stop evaluation, such as visibility, schools, or pedestrians. Furthermore, the "mixed" situation (moderate volumes, a few accidents, some pedestrians) is not addressed.

CITY OF SAN DIEGO'S EXPERIENCE

As an alternative to the national policy, the City of San Diego developed an all-way stop policy based on a point system in 1962. The system was based on several warrants, each worth a few points. All-way stops were justified at candidate intersections that were assigned a majority of the total available points. This policy was an improvement over the MUTCD policy because it was not dependent on signals, and it addressed the areas that the MUTCD policy overlooked. Another strength was the introduction of the Traffic Volume Difference Warrant, which awarded points to intersections based on the closeness of the traffic volumes on the intersecting streets.

The policy also had several weaknesses. For instance, no single warrant could in itself justify all-way stops. Each warrant simply contributed points to a total. In some circumstances, a candidate intersection may have received maximum points from one or more warrants but still did not qualify for all-way stops because a majority of the total points had not been accumulated. Another weakness was that the policy did not contain the MUTCD provision for using all-way stops as interim measures before installing traffic signals.

City staff encountered situations in which engineering judgment indicated that all-way stops would be appropriate at a particular location, yet neither the MUTCD warrants nor the city's own policy could justify the installation. Consequently, the City began in 1986 to research all-way stops and develop a revised all-way stop policy. The goals of the new policy were as follows:

1. Consistency. The policy should be in conformance with traffic engineering principles of safety and operation for all-way stop intersections.
2. Accountability. The policy should be based on all-way stops, not signals.
3. Flexibility. The policy should equally consider intersections that have extreme circumstances in one category that may justify all-way stops, as well as intersections that have a combination of factors, none of which individually would justify all-way stops.
4. Selectivity. The policy should be effective at distinguishing the candidate intersection that will benefit from the installation of all-way stops.

THE NEW POLICY

The new policy consists of five warrants and a total of 50 points. All-way stops may be justified at intersections that are assigned 25 or more points. The 25-point requirement may be waived, and all-way stops justified, under any one of the following special provisions:

1. Five or more accidents susceptible to correction by all-way stops have occurred in a 12-month period.
2. Traffic signals are warranted and not yet installed.
3. The intersection has an extreme combination of unusual conditions, and engineering judgment determines that the location would be best served by all-way stops. Examples of unusual conditions are a school, fire station, playground, bus route, steep hill, and visibility limitation. A school in itself is not considered to be sufficient justification for all-way stops.

Provisions 1 and 2 are adopted from the MUTCD warrants. Provision 3 should be used sparingly, usually after less severe controls have been attempted.

The following includes an explanation of each warrant:

1. Accident experience—maximum 15 points. Three points are assigned for each correctable accident that occurred in the preceding 12-month period.
2. Unusual conditions—maximum 5 points. Points are assigned for unusual conditions based on engineering judgment. The point value assigned to each condition should be correlated to the improvement to the situation that all-way stops would provide. When awarding points in this warrant, it is important to consider only the actual benefits that all-way stops provide, not the perceived benefits attributed to all-way stops by many nonprofessionals. Speed control should never be a basis for awarding points.
3. Traffic volumes—maximum 15 points. Two tables, one for the minor street and one for the major street, are used to assign points based on volume. The major street is defined as the traffic approaches that are not controlled by stop or yield signs at the time of the evaluation. The minor street is defined as the approaches that are controlled. For the minor street, the number of points awarded increases as the volume increases up to a maximum of ten points. For the major street, the maximum of five points is assigned to a range of volumes at which all-way stops function best. Above or below this optimum volume range, fewer points are awarded. To determine the optimum range for all-way stop volumes in the new policy, the 1985 *Highway Capacity Manual* (*2*) was consulted. The following is the method used for deriving "ideal" volume:

The 1985 *Highway Capacity Manual* was consulted for determining the point assignment tables for traffic volume. The level-of-service (LOS) C service volumes for four all-way stop intersections are as follows:

Demand Split	*LOS C Service Volume (vph) by Lane Configuration* 2 by 2	2 by 4	4 by 4
50/50	**1,200**	1,800	2,200
55/45	1,140	1,720	2,070
65/40	1,080	1,660	1,970
65/35	1,010	1,630	1,880
70/30	960	1,610	1,820

The tabulation is sorted into demand splits ranging from 50/50 to 70/30 and lane configurations (2 by 2, 2 by 4, and 4 by 4). It was determined that the traffic volume point assignment table should be derived from the case of a 50/50 demand split at a two-lane by two-lane intersection. The LOS C service volume for this situation is 1,200 vehicles per hour (vph) entering the intersection.

Since the City of San Diego uses 4-hour counts for traffic studies, the 1,200 vph translated into 4,800 vehicles in 4 hours. Therefore, with an ideal 50/50 split, each street should have a 4-hour approach volume of 2,400 vehicles. Consequently, the figure of 2,400 vehicles is within the maximum point range for both the major street and the minor street point assignment tables. For the major street, the optimum range is between 2,201 and 2,600 vehicles in 4 hours. For the minor street, all volumes above 2,201 are considered optimum and are assigned maximum points. The point assignment tables are shown in Table 1.

4. Traffic volume difference—maximum 10 points. This warrant differs from the "traffic volumes" warrant in that it considers only the difference between the 4-hour volumes of the two streets. All-way stops function best when the difference between the volumes is small. Accordingly, a small traffic volume difference is assigned maximum points. The point assignment table for this warrant is shown in Table 2.

5. Pedestrian volumes—maximum 5 points. The volume of pedestrians crossing the major street is of concern when evaluating for all-way stops. One point is assigned for each set of 50 pedestrians in 4 hours, as shown in Table 3.

An evaluation sheet is shown in Figure 1.

TABLE 1 POINT ASSIGNMENT FOR TRAFFIC VOLUME

Major Street		Minor Street	
4-hour Volume	Points	4-hour Volume	Points
0–1,000	0	0–400	0
1,001–1,300	1	401–600	1
1,301–1,600	2	601–800	2
1,601–1,900	3	801–1,000	3
1,901–2,200	4	1,001–1,200	4
2,201–2,600	5	1,201–1,400	5
2,601–2,900	4	1,401–1,600	6
2,901–3,200	3	1,601–1,800	7
3,201–3,500	2	1,801–2,000	8
3,501–3,800	1	2,001–2,200	9
3,801–over	0	2,201–over	10

TABLE 2 POINT ASSIGNMENT FOR TRAFFIC VOLUME DIFFERENCE

Volume Difference (4-hour count)	Points
0–150	10
151–300	9
301–450	8
451–600	7
601–750	6
751–900	5
901–1,050	4
1,051–1,200	3
1,201–1,350	2
1,351–1,500	1
1,501–over	0

TABLE 3 POINT ASSIGNMENT FOR PEDESTRIAN VOLUME

No. of Pedestrians Crossing Major Street in 4 hours	Points
0	0
1–50	1
51–100	2
101–150	3
151–200	4
201–over	5

TESTING THE NEW POLICY

Once it had been developed, there was interest in how the new policy compared to the city's previous policy. A total of 23 intersections in the City of San Diego were used to test the ability of the new policy to select intersections that benefit from and function well with all-way stops. The intersections chosen for the study all had all-way stops that had been installed (either by engineering judgment or City Council directive) despite having failed to meet the city's previous policy. The intersections were then reevaluated, by using the new policy with data from the original evaluation.

Fourteen of the intersections met the criteria of the new policy. That is, if the new policy had been in effect at the time that the intersections were originally evaluated for all-way stops, then 14 of the 23 would have qualified. The 14 were placed in Group A for comparison purposes. The remaining nine intersections, those that failed to meet all-way stop warrants under either the old or new policy, were placed in Group B.

The study consisted of analyses of accidents and field performance. The accident analysis involved 19 intersections, 12 from Group A and 7 from Group B. The field analysis used 15 intersections, 8 from Group A and 7 from Group B. All 23 of the intersections were included in at least one of the analyses.

The first analysis, a comparison of the number of accidents 12 months before and after the all-way stops were installed, showed that the intersections in Group A experienced a significant reduction. In contrast, the intersections in Group B did not experience a significant change in accidents; in fact, the number of accidents rose slightly. Figures 2 and 3 show the results of the before-and-after accident comparison. For Group A, the reduction in accidents that occurred at the intersections was found to be statistically significant at the 99 percent confidence level. For all accidents at or near the intersections (midblock accidents are assigned to the nearest intersection), the decrease was also significant at the 99 percent confidence level.

The field analysis also gave interesting results. Group A had an average volume ratio of major street to minor street of 1.8, whereas the ratio for Group B was 4.0, as shown in Figure 4. These data support the idea that all-way stops function best when the cross-street volumes are nearly equal. A key finding was that Group B had a higher frequency of major street motorists failing to stop, as shown in Figure 5. In Group A, 6.8 percent of the motorists on the major street failed to stop, whereas in Group B, 13.0 percent failed to stop. The difference between the two groups was found to be statistically

Intersection ____________ (MAJOR) ____________ (MINOR)

File ____________
Date ____________
Investigator ____________

Qualifies for All-Way Stop based on 25 or more points:
Yes ___ No ___ Points ___

Qualifies for All-Way Stop based on other criteria: Yes ___ No ___
If yes, explain:

____

Sketch of intersection with visibility data
On back ___ Attached ___

	Points	Possible
1. Accident Experience		
From __/__/__ to __/__/__		
Accidents/year correctable by Stops x 3 points/accident	______	15
2. Unusual Conditions		
______________________________	______	5
______________________________		
______________________________		
3. Traffic Volumes (Peak 4 Hours)		
Major approaches ______	______	5
Minor approaches ______	______	10
4. Traffic Volume Difference ______	______	10
5. Pedestrian Volume		
Pedestrians ______ crossing the major street during 4 hour count	______	5
TOTAL	______	50
Points Required	25	

FIGURE 1 All-way stop evaluation worksheet.

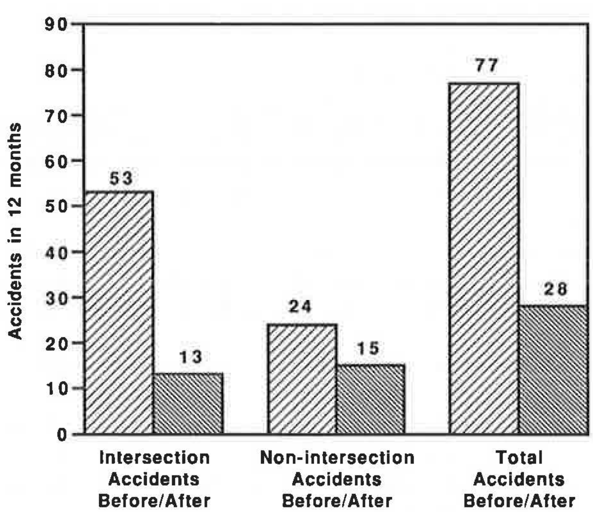

FIGURE 2 Before-and-after accident comparison (Group A).

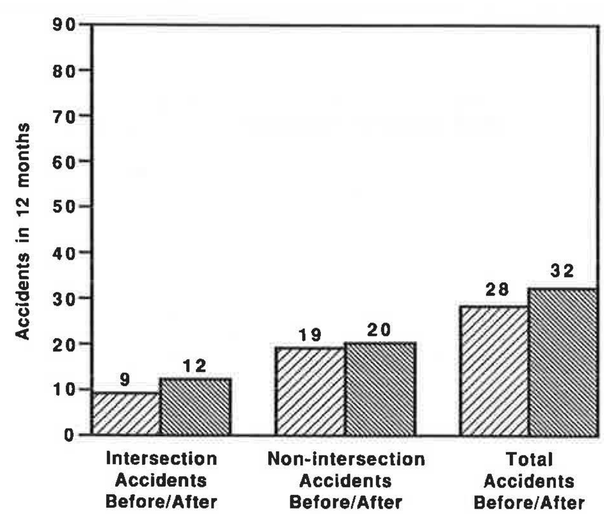

FIGURE 3 Before-and-after accident comparison (Group B).

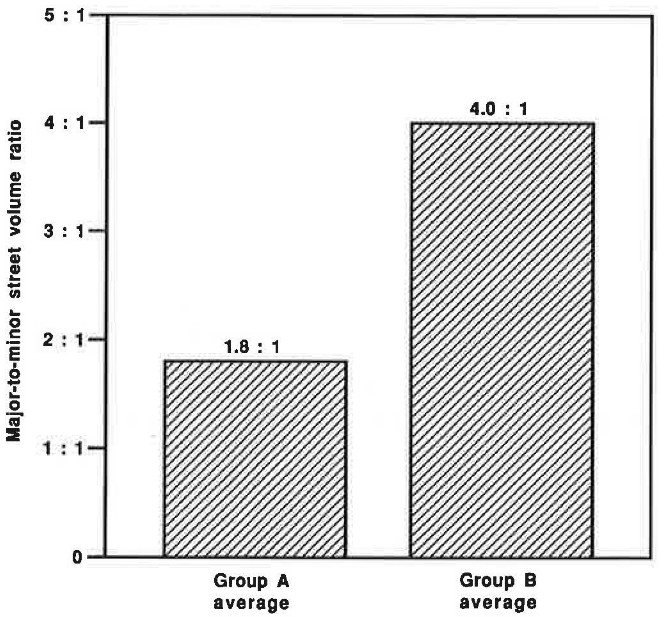

FIGURE 4 Volume ratio comparison (Group A versus Group B).

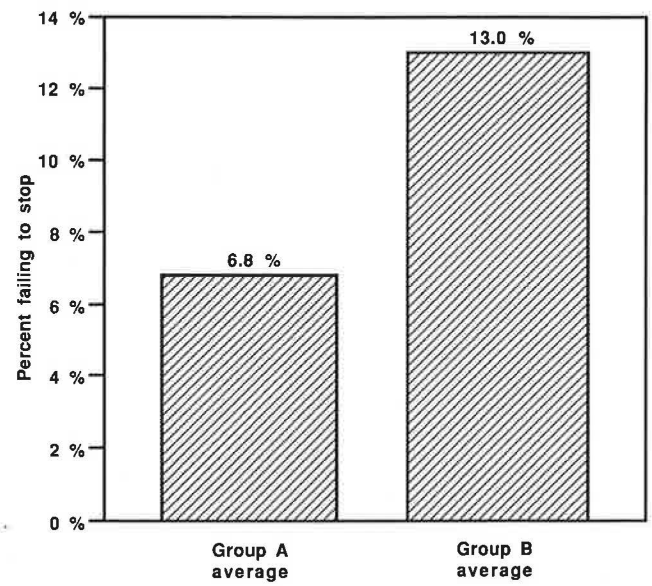

FIGURE 5 Failure-to-stop comparison (Group A versus Group B).

significant at the 95 percent confidence level. These figures indicate that the new policy is successful at selecting intersections where all-way stop controls will earn motorists' respect and have a better rate of stop sign compliance.

The results of the statistical analyses are shown below:

Note: Only those tests that showed statistical significance are shown.

1. Accidents at intersection: 53 in 12 months before all-way stop was installed; 13 in 12 months after.
   - Calculated t = 3.028, d.f. = 22;
   - Tabulated t (at 99 percent confidence) = 2.819;
   - Therefore the difference is significant at the 99 percent confidence level.

2. Total of accidents at and near intersection: 77 in 12 months before all-way stop was installed, 28 in 12 months after.
   - Calculated t = 2.865, d.f. = 22;
   - Tabulated t (at 99 percent confidence) = 2.819;
   - Therefore the difference is significant at the 99 percent level.

3. Percent of vehicles on major street failing to stop: Group A—6.8 percent, Group B—13.0 percent.
   - Calculated z = 2.334, d.f. = 13;
   - Tabulated z (at 99 percent confidence) = 3.012;
   - Tabulated z (at 95 percent confidence) = 2.160;
   - Therefore the difference is significant at the 95 percent confidence level.

CONCLUSION

The new policy meets all of the goals for a model all-way stop policy. The policy is consistent with traffic engineering principles, is not dependent on traffic signal warrants, is flexible for use in differing conditions, and is successful at selecting intersections that benefit from the installation of all-way stops. It will give traffic engineers confidence in the all-way stop warrants when discussing the issue with citizens' groups and elected officials. The policy will assist traffic engineers in their mission of educating the public about traffic safety and providing the public with safe streets and efficient traffic flow.

ACKNOWLEDGMENTS

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DISCUSSION

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The paper by Celniker is a welcome departure from the MUTCD multiway stop warrants, which have been criticized elsewhere (*1,2*) for their lack of scientific validity.

This comment deals with Celniker's statement that safety is jeopardized when drivers disobey a stop sign. When a sign imposes a needless stop, it fails to meet two basic requirements for a traffic control device to be effective: it does not fulfill a need and it does not command respect. In Dyar's study, (*3*) 88 percent of all motorists disregarded stop signs in light traffic and treated them as yield signs when there was no one to stop for—clear evidence of an overly restrictive control (*4*).

The compulsory stop regardless of traffic conditions should not only be justified by evidence showing that the failure to stop per se (rather than the failure to yield) contributes to collisions, but also that the cost of these collisions outweighs the cost of the additional delay, fuel consumption, and air pollution. Without such proof, the unconditional stop is not warranted (*5*).

To first maintain that needless stops should be avoided in the interest of safety, efficiency, and respect for traffic controls, and then claim that the failure to come to a peremptory but needless stop jeopardizes safety, is a contradiction the traffic engineering profession has yet to explain. The logical way out of this contradiction is the all-way yield, a technique capable of competing with traffic signal control in terms of costs to the road user and highway agency (*6*).

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AUTHOR'S CLOSURE

The purpose of the new all-way stop policy is to balance the public's request for all-way stops with traffic engineering principles of safety and operations. The new policy is positive for the following reasons:

- The all-way stop is an existing, familiar traffic control device.
- The new policy is flexible to a variety of factors, yet it allows only all-way stops to be installed at intersections where they will function well.
- The concept of avoiding unnecessary stop signs is consistent with, not contradictory to, the statement that a failure to stop jeopardizes safety. The policy's goal is to install all-way stops only where they will have a high rate of compliance.

The "all-way yield" proposal is a deeply flawed alternative. Traditionally, a yield sign says to motorists "yield the right-of-way to cross traffic by either stopping or slowing down; then, when there are no vehicle conflicts go ahead." This message is very useful and successful in cases of low-volume intersections or channelized right-turn lanes. The yield signs face only the direction of traffic that yields.

The proposed "all-way yield" changes the message of the yield sign to "slow down, a complete stop is not necessary; yield the right-of-way to cross traffic as you would at an all-way stop or an uncontrolled intersection, then go ahead."

The all-way yield is a basic contradiction in terms, potentially dangerous, and unnecessary. Motorists will be confused about the new use of a familiar sign, and such confusion may lead to accidents. Also, the successful, traditional use of the yield sign will be lost if yield signs take on a new meaning.

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Proposed Procedure for Selecting Traffic Signal Control at School Crossings

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This paper describes the current criteria used to determine the need for traffic control at school crossings. It discusses the background and assumptions used in establishing these current criteria and points out a few of its deficiencies. Based on this assessment, a methodology is proposed that is intended to improve on the current criteria. This methodology has the advantages of enabling more equitable treatment of vehicular and pedestrian traffic as well as allowing for easier application. The current procedure for determining the need for traffic signal control is based on a field study technique that yields the length of time needed by pedestrians to cross the street and the frequency with which gaps of this duration occur in the vehicular traffic stream. The proposed procedure is similar to the current procedure; however, by assuming that the distributions of pedestrian and vehicular arrivals are random, time-dependent processes, the Poisson distribution can be used to estimate the gap duration and its frequency of occurrence. The intent of this approach is to eliminate the need to conduct a field study at locations where the assumption of random arrivals is valid. As a result of this investigation, a procedure is proposed that incorporates the intent of the current procedure but is much simpler to apply. Additional criteria have been added that address the issue of interruption of pedestrian flows and the need for a minimum pedestrian volume. The goal of this procedure is to provide a means of ensuring the uniform application of traffic control devices and to avoid the unnecessary installation of traffic signals.

The safety of children traveling between home and school is a highly sensitive subject. There are often conflicting opinions among citizens and public officials about what must be done to ensure the safety of the children. Citizens typically want additional police officers, crossing guards, and traffic controls on any street they feel is "potentially dangerous." Public officials representing the school and the city must respond to the citizen's concerns by determining a proper course of action. Their determination is founded on the desire to maximize both motorist and pedestrian safety, but it must be tempered by the effective use of limited funding resources and the application of reasonable and uniform traffic control policies. (The term "pedestrian" is used throughout to refer to school children walking along, adjacent to, or across the roadway.)

Engineering studies have shown that the uniform application of reasonable traffic control procedures will improve public understanding and acceptance of any control measure. In contrast, use of non-uniform procedures or controls creates confusion and lessens the respect given to them by pedestrians and motorists alike. Hence, establishing and adhering to a set of reasonable and uniform criteria for applying traffic control is a desired goal because it ensures equal treatment at similar locations and moderates the influences of emotion and bias.

This paper describes the current criteria used to determine the need for traffic control at school crossings. It discusses the background and assumptions used in establishing the current criteria and points out a few of its drawbacks. Based on this assessment, a new methodology is proposed that is intended to improve on the current criteria. This methodology has the advantages of enabling more equitable treatment of vehicular and pedestrian traffic as well as allowing for more convenient application.

STATEMENT OF PROBLEM

According to the Uniform Vehicle Code, motorists must yield the right-of-way to pedestrians within crosswalks or within unmarked crosswalks at intersections (*1*). From this it might be concluded that signal control is never needed because motorists should stop whenever a pedestrian crosses within a crosswalk. However, the reality of the situation is that motorists do not always yield to pedestrians for a variety of reasons. Hence, there is a need for signal warrants at pedestrian crossings, particularly when the pedestrians are school-age children.

A procedure for establishing school crossings is currently addressed in the *Manual on Uniform Traffic Control Devices* (MUTCD), Traffic Signal Warrant 4—School Crossing (*2*). In particular, the manual suggests that a signal may be warranted when "the number of adequate gaps in the traffic stream, during the period when the children are using the crossing, is less than the number of minutes in the same period (*2*,p.4C-5)." The intent of this warrant is to limit the time a child must wait to cross to less than 60 sec. This maximum has been established because studies have shown that pedestrians will become impatient after waiting 30 sec to cross and will edge out into the roadway after 40 sec (*3–6*).

The application of this warrant requires the conduct of a traffic engineering study to determine the duration of an "adequate" gap and the availability of these gaps in the vehicular traffic stream. Although this type of study is simple to conduct, it has the drawback of consuming both time and resources. Another drawback of this warrant is that it considers only the delay to pedestrians, which is not necessarily the safest or best operation of the crossing. For example, at those crossings where pedestrian flows are characterized by high volume and random arrivals, it is entirely likely that motorists (stopped

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for pedestrians) will experience near 100 percent stopping and high delays and will often be seen exhibiting the same impatient attitudes that are displayed by pedestrians waiting for gaps in vehicular traffic. This situation often precipitates unsafe actions, such as motorists "weaving" through the pedestrian streams, diverting onto nearby streets, and disregarding the basic rules of vehicular and pedestrian right-of-way at intersections.

Other drawbacks of the MUTCD school crossing warrant and its application include the following:

- It does not have a minimum pedestrian flow rate below which signal control should not be considered. As a result, it could take only one school-age pedestrian wanting to cross a major arterial to "warrant" a signalized crossing.
- The procedure for determining adequate gap size does not reflect the distribution of the arriving pedestrian flow but, rather, the size of the pedestrian group accepting a gap provided by the vehicular stream. Hence, the gap size study is affected by the distribution of vehicular gaps and, as a result, is biased toward higher group sizes.
- The procedure does not identify a minimum period or duration during which the pedestrian flow rate would equal or exceed the minimum rate.

Each of these drawbacks has been considered for this paper and will be discussed in the following sections.

Determining the Need for School Crossing Control

The procedure for determining the duration of an "adequate" gap and the frequency of its occurrence was originally described in an Institute of Transportation Engineers (ITE) publication (*3*). The procedure has since been reproduced in the *Transportation and Traffic Engineering Handbook* (*7*). This procedure is based on field survey techniques in which the size of the pedestrian groups crossing the roadway and the duration of the gaps between vehicles traveling on the roadway are recorded for a common interval of time. Obviously, this interval is associated with the periods before and after school when children are crossing the roadway.

The adequate gap time is calculated by using the following formula:

$$G = R + (W/S) + K * (N - 1) \qquad (1)$$

where

G = adequate gap time (sec),
R = pedestrian perception and reaction time (sec) (assumed to be 3.0 sec),
W = curb-to-curb width of the street crossed (ft)
S = walking speed of child (ft/sec) (assumed to be 3.5 ft/sec),
K = time between successive rows (sec/row) (assumed to be 2.0 sec/row),
N = number of rows in the 85th percentile pedestrian group size rounded up to the next integer value $\{= \text{integer}[(Q85/n) + 1]\}$,
$Q85$ = 85th percentile pedestrian group size observed during the field survey, (pedestrians/group), and
n = number of pedestrians in each "row" crossing the street (assumed to be five pedestrians per row).

This study would be conducted during a normal school day during the heaviest period of pedestrian crossing activity.

Once the duration of the adequate time gap is known, the number of gaps of equal or greater size in the vehicular traffic stream is determined and compared with the number of minutes that transpired during the period of pedestrian activity. If the number of adequate gaps is fewer than the number of minutes (implying an average wait of more than 60 sec), then a signal may be installed to artificially create the necessary gaps.

Basis for Proposed Procedure

Recognizing the limitations of the current procedure, a proposed procedure was formulated that could be used to determine the pedestrian and vehicular volume conditions that would require some type of control to improve safety and minimize delay. The proposed procedure for determining the need for school crossing control is based on and intended to supplement the existing warrant criteria. It is offered as a tool to simplify the evaluation of MUTCD's signal warrant criteria for school crossings; it also addresses some of the drawbacks of the ITE field study procedure. The proposed procedure is based on the following assumptions:

- Both pedestrian and vehicular flows are random processes.
- The assumed values used in Equation 1 are reasonable.
- The average wait of 60 sec describes the threshold of pedestrian patience.

If these assumptions do not hold or if the situation requires special consideration for other reasons, then the ITE field study procedure should be used. The development of the proposed new procedure is described in the next section.

DEVELOPMENT OF THE PROPOSED PROCEDURE

The proposed procedure is directed toward the development of a series of graphs that relate traffic volume, pedestrian volume, and the type of control needed. The graphs are similar to those used for MUTCD Signal Warrant 11—Peak Hour Volume—in that the volume levels are assigned to the horizontal and vertical axes and the recommended control is identified by a region within the graph. The intent of this procedure is to avoid the need for a field study to determine the adequate gap size or the frequency of its occurrence. The steps in the procedure's development are described in the following sections.

Determination of Adequate Gap

The procedure for determining the adequate gap (Equation 1) was used without modification, recognizing that some of the assumed values may not be universally accepted. In particular, it has been suggested by some that the reaction time be less than 3.0 sec (*6,8*). In addition, it has been suggested that the width of the street be reduced by the width of the far-side curb parking lane (*9*). It has also been implied that all pedestrians will cross as a single row (i.e., N = 1) regardless of the pedestrian volume (*6,8–10*). These deviations are recognized here only for completeness; the values recommended elsewhere (*7*) are used for this analysis.

By using the assumed values stated in Equation 1, the size of adequate gap was calculated by using the following equation:

$$G = 3.0 + (W/3.5) + 2.0 * (N - 1) \quad (2)$$

where

G = adequate gap time (sec)
N = integer[$(Q85'/5) + 1$],
$Q85'$ = estimated 85th percentile pedestrian group size, $(q*t) + k * (q*t)^{0.5}$
q = pedestrian flow rate averaged over 15 or more minutes (pedestrians/second)
t = time interval between acceptable gaps (sec) (equal to 60 sec or 1 gap/min),
k = number of standard deviations the 85th percentile volume is away from the mean volume (assumed to be 1.0), and
$(q*t)^{0.5}$ = standard deviation of the Poisson distribution.

Using the above assumed values, the number of rows of pedestrians (N) that wish to cross each minute can be calculated as

$$N = \text{integer}\left[\frac{(q*60) + (q*60)^{0.5}}{5} + 1\right] \quad (3)$$

By using Equations 2 and 3, the size of the 85th percentile pedestrian group size can be estimated given only the pedestrian flow rate and the width of the street to be crossed. Values of the adequate gap size (G) have been calculated for various roadway widths and are shown in Figure 1.

This approach attempts to model group size as a function of arriving demand and not as it would be observed in a field study in which the distribution of gaps in the vehicular stream would artificially "bunch" the pedestrian groups. What is typically measured by a study of pedestrian flow at the crosswalk is not the true demand versus the time profile but the maximum flow rate through a bottleneck. Based on this observation, it is suggested that the traditional study of demand at the crosswalk will likely yield an inflated estimate of the group sizes of arriving pedestrians. The consequences of overestimating pedestrian group size would be the exclusion of traffic gaps that are, in fact, sufficient for pedestrian use.

Frequency of Adequate Gaps

The calculation of the frequency of adequate gaps is based on the assumption of random arrivals. This assumption implies that the probability that any one gap is equal to or greater than the adequate gap (G) is

$$P(g > G) = \exp(-v * G) \quad (4)$$

where

G = adequate gap time from Equation 1 or 2 (sec),
v = vehicular flow rate averaged over 15 or more minutes (vehicles/second), and
$\exp(x)$ = the base of the natural log e (2.718...) raised to the power x.

From this relation, the number of adequate gaps can be calculated by using

$$PG = [v*t * \exp(-v * G)] / [1 - \exp(-v * G)] \quad (5)$$

where

PG = number of adequate pedestrian gaps [gaps/interval (t)] (defined as one gap/interval),
v = vehicular flow rate averaged over 15 or more minutes (vehicles/second),
t = time interval between acceptable gaps (sec) (equal to 60 sec), and
G = adequate gap time from Equation 1 or 2 (sec).

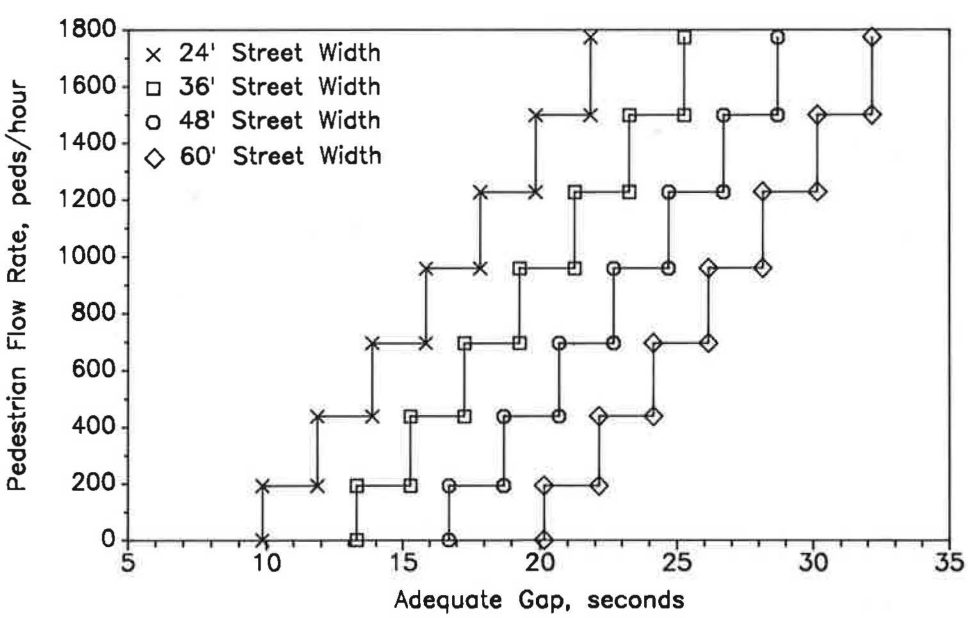

FIGURE 1 Adequate gap versus pedestrian flow rate.

From Equation 5, the maximum vehicular flow rate can be calculated that will yield one gap of adequate size at an average of every 60 seconds. The relation used in Equation 5 is an extension of other gap relations that have been used by other authors but does not share their limitations. These other relations include

$$PG = v * t * \exp(-v * G) \quad (6)$$

$$PG = (60 / G) * \exp(-v * G) \quad (7)$$

Equations 5, 6, and 7 are compared in Figure 2. As shown in this figure, Equation 6 is in close agreement with Equation 5 under high-volume conditions, but it also implies that there is a maximum size to the gaps in the traffic stream. The problem with Equation 6 is its failure to recognize the potential for more than one pedestrian group to cross during the longer traffic gaps. Additional insight into this problem is described elsewhere (*10*).

Equation 7 also appears to be in close agreement with Equation 5 under low-volume conditions. The discrepancy between these equations appears in the higher volume range. Equation 7 yields much too conservative an estimate of the maximum vehicular flow rate for small gap sizes. The problem with Equation 7 is that it estimates the number of gaps that can occur in one minute (i.e., $60/G$) rather that the actual number of gaps in traffic per minute (i.e., $v*t$). As a result, it underestimates the number of gaps available for pedestrian use, particularly for higher volume conditions.

Based on the preceding discussion, Equation 5 appears to agree with Equations 6 and 7 under specific volume conditions. The advantage of Equation 5 is that it yields reasonable estimates over the entire range of volume conditions. In recognition of these benefits, Equation 5 is used in the development of the proposed procedure.

By applying Figures 1 and 2, it is possible to determine the size of the adequate gap required by pedestrians and whether or not gaps of this size are available in the existing vehicular stream. If the combination of existing vehicular traffic and pedestrian gap length is so great as to fall above the solid line in Figure 2, then signal control is warranted.

Interruption of Pedestrian Flow

Occasionally the volume of pedestrians is so high that, once given the right-of-way, they do not relinquish it for a considerable time. This can result in lengthy delays for motorists and is the reverse of the problem just described (i.e., pedestrians delayed by motorists). Motorists can, and should be expected to, tolerate longer delays than pedestrians because of the added protection of motorists from inclement weather and because of the added danger to pedestrians standing along the roadside. However, a threshold level of pedestrian volume should be determined wherein some artificial means of interruption (such as a traffic signal or crossing guard) is necessary to allow a minimum number of vehicles to pass. The following discussion describes the calculation of such a threshold.

Based on the assumptions of random arrivals of pedestrians and that motorists will continue to yield as long as pedestrians are in the crosswalk, Equation 5 is again used to calculate the number of available gaps. However, this time the available gaps are in the pedestrian flow and they are entered by a standing queue of vehicles. The equation used is defined as follows:

$$VG = NL * [q * T * \exp(-q * a)]/[1 - \exp(-q * b)] \quad (8)$$

where

VG = maximum number of adequate vehicular gaps (gaps/hour),
q = pedestrian flow rate averaged over 15 or more minutes (pedestrians/second),
T = duration of pedestrian crossing activity (sec) (assumed to be 3600 seconds),

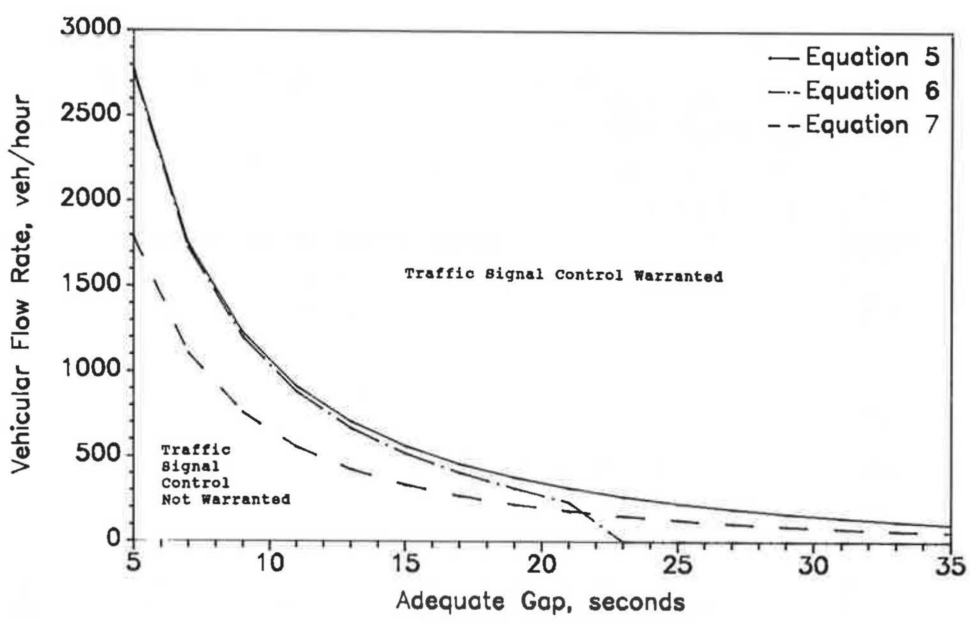

FIGURE 2 Adequate gap versus vehicular flow rate.

a = minimum gap for one vehicle to cross (sec) $r + (W/S)$,
r = motorist perception and reaction time (sec) (assumed to be 2.0 seconds),
W = curb-to-curb width of the street crossed (ft),
S = walking speed of child (ft/sec) (assumed to be 3.5 feet/second),
b = minimum headway between vehicles on a two-way, two-lane street (assumed to be 2.57 sec), and
NL = number of traffic lanes within street (assumed to be 2 for $W < 40$ ft and 4 for $40 < W < 65$ ft).

Recognizing that this represents the maximum number of vehicles that can cross through a pedestrian flow (i.e., vehicular capacity), a lower value was selected that reflected a more stable, nominal delay condition. Consulting the *Highway Capacity Manual*'s level-of-service descriptions for unsignalized intersections, it was found that a reserve capacity of 100 to 199 vehicles per hour per lane (vphpl) described a condition of "long traffic delays" (*11*). This was denoted as level-of-service "D." Based on this description, it was determined that the vehicular capacity calculated from Equation 8 should be reduced by 100 vphpl. Thus, the threshold values of pedestrian volume that would still yield a level-of-service "D" condition for traffic were calculated as

$$NG = VG - NL * 100 \qquad (9)$$

The number of gaps available (NG) was further adjusted to account for an assumed 55/45 percent directional split in vehicular demand.

Minimum Pedestrian Flow Rate

The reason for establishing minimum pedestrian and vehicular flow rates is to avoid the indiscriminate installation of traffic signals. Installing and maintaining unwarranted signals will lead to intentional violation, increased hazard, and unnecessary delay. Furthermore, the cost of installing and maintaining unwarranted signals does not constitute an efficient use of limited safety funds.

Recognizing that minimum volume warrants are more often based on rational judgment and experience rather than on theoretical analysis, a review of the literature was conducted to determine if any formally adopted pedestrian volume warrants existed. One agency that has established such warrants is the California Department of Transportation (CDOT) (*12*). These warrants are reasonable and consistent with the intent of this paper. More importantly, they imply that there are minimum pedestrian and vehicular volumes below which signal control is not necessary. In recognition of this general agreement, the minimum pedestrian flow rates developed for the proposed procedure are loosely based on CDOT warrants for school crossing traffic signals.

In general, it is recommended that the minimum pedestrian flow rate be established at 100 per hour. This minimum may be reduced to 50 pedestrians per hour if

1. A crossing is being considered at a location where the nearest existing traffic signal, controlled crossing, or pedestrian overpass is over 300 ft away, or
2. The crossing is in an area where adequate and safe sidewalks are not available to and from the location with the existing signal, crossing, or overpass.

In addition, it is suggested that a minimum of 500 pedestrians use the crossing during an average day. If the crossing is in a rural area or when the 85th percentile speed exceeds 40 mph, then 70 percent of the above minimums should be used.

The period of analysis should correspond to the period of peak pedestrian demand. However, the minimum period of analysis is a 15-minute interval (even if the duration of pedestrian demand is shorter). The vehicular and pedestrian flow conditions must occur during the same peak period and must be representative of the average day.

The intent of these minimums is to eliminate the unnecessary installation of signalized pedestrian crossings. This is not to suggest that pedestrian flow rates less than these minimums do not need traffic signal control but only that other forms of control or transport (of the students across the street) should be considered.

Combination of Pedestrian and Vehicular Demand

As discussed at the beginning of this section, the goal was to develop a graph with the pedestrian and vehicular flow rates on horizontal and vertical axes, respectively. The approach just described accomplishes this goal. Furthermore, this approach incorporates the determination of adequate gap size for pedestrian flow rates and, thereby, eliminates the need to conduct the ITE field study procedure. This procedure also considers the possible need for interruption of pedestrian flows to maintain reasonable vehicular operation.

All variables were eliminated in the derivation of this procedure except the width of the cross street (W). As a result, a series of graphs were constructed for selected street widths. These graphs are shown in Figures 3, 4, 5, and 6 for street widths of 24, 36, 48, and 60 ft, respectively. There are essentially three determinations (identified by regions) that can be made from these graphs based on the known vehicular and pedestrian flow rates:

1. Pedestrian and vehicular demands are sufficiently light that no control is necessary.
2. Pedestrian demand is heavy and vehicular demand is light (or vice versa) and therefore some type of control is necessary to interrupt pedestrians to maintain a minimal level of vehicular service. The control considered may include a crossing guard or a traffic signal.
3. Both pedestrian and vehicular demands are sufficiently heavy that a traffic signal may be needed to separate the two conflicting flows.

Each of these regions is identified on the graphs where appropriate. The left-most vertical boundary represents a minimum flow rate of 100 pedestrians per hour. This boundary should be shifted left if a lower minimum flow rate is selected.

Validation of Proposed Procedure

Because of a limited amount of field data, pedestrian crossing activity was computer simulated for the purpose of validating

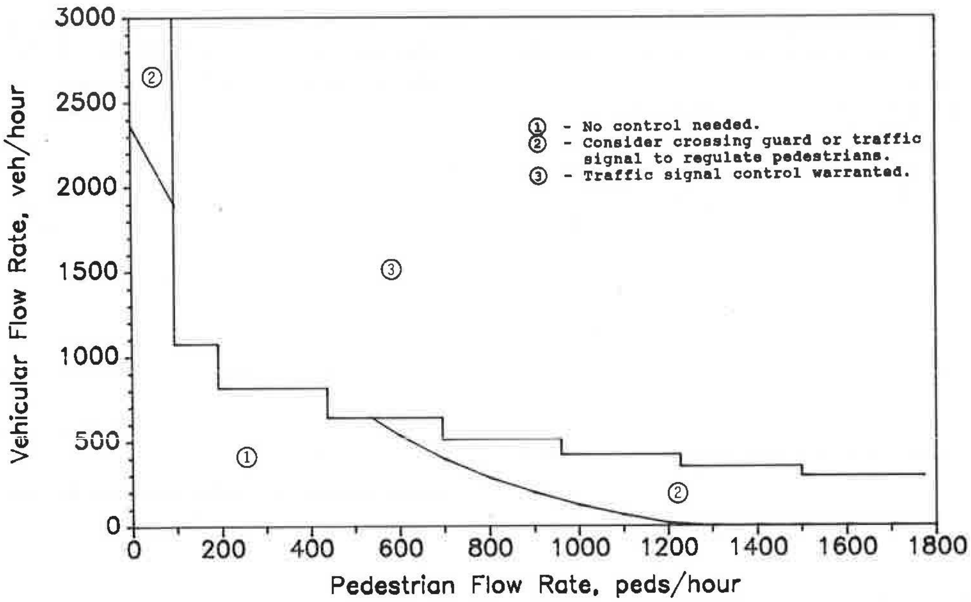

FIGURE 3 Pedestrian versus vehicular flow rate: 24-ft street width.

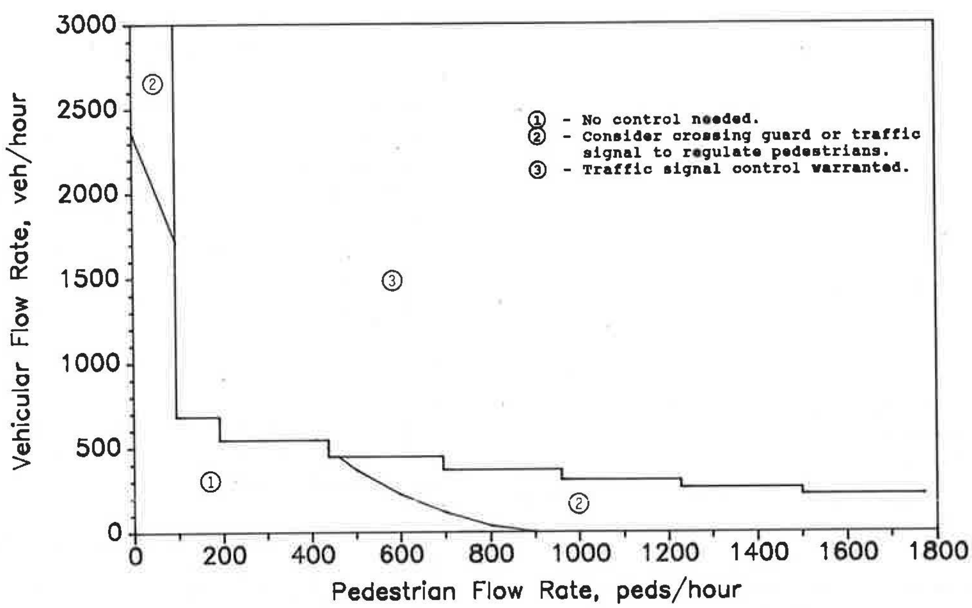

FIGURE 4 Pedestrian versus vehicular flow rate: 36-ft street width.

the proposed procedure. In particular, that portion of the procedure that theoretically determines the adequate gap size and the frequency of these gaps was compared with simulation results for similar demand conditions.

Before the results are discussed, it will be helpful to recall the objective of the ITE field study procedure (and the procedure proposed in this paper). This objective is to limit the time pedestrians must wait for an adequate gap. In an attempt to satisfy this objective, ITE proposed two field studies that are intended to ensure that 85 percent of all pedestrians can find an adequate gap in less than 1 minute, on average. Thus, in an indirect manner, the ITE procedure attempts to limit the delay experienced by 85 percent of the pedestrians to less than 60 seconds. It is this delay criterion that was used in the simulation results for comparative purposes.

The results of the simulation are shown in Figure 7. As alluded to in the previous paragraph, the simulation data reflect the threshold combination of demands that would cause 85 percent of all pedestrians to experience delays of 60 seconds or less. In general, the ITE and proposed procedures appear to yield results that are consistent with the delay criterion, which is not surprising since they were both formulated to do just that. In summary, these simulation results indicate that the proposed procedure can predict demand combinations that yield pedestrian delays of about 60 seconds or less; which is consistent with the procedure's original objective.

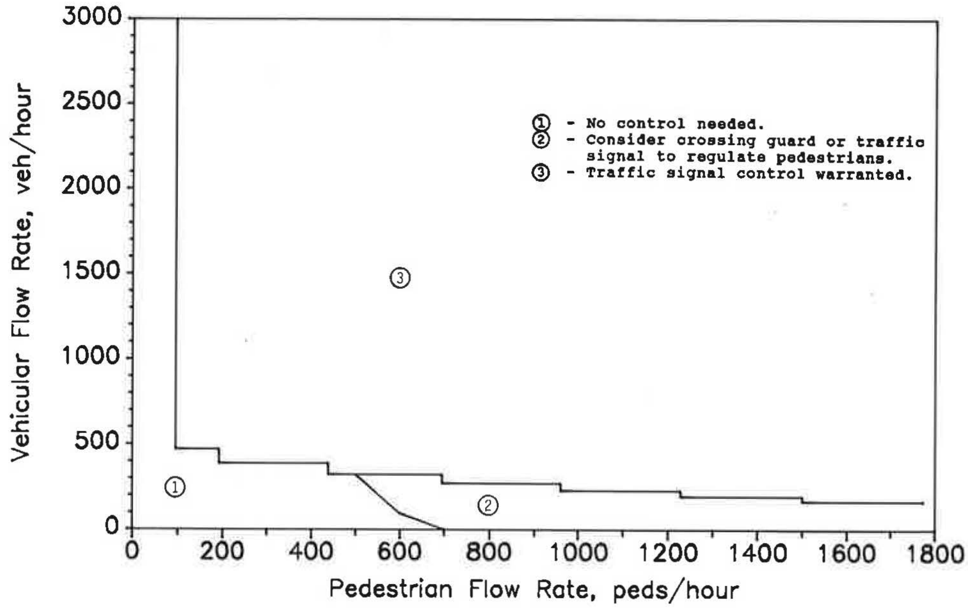

FIGURE 5 Pedestrian versus vehicular flow rate: 48-ft street width.

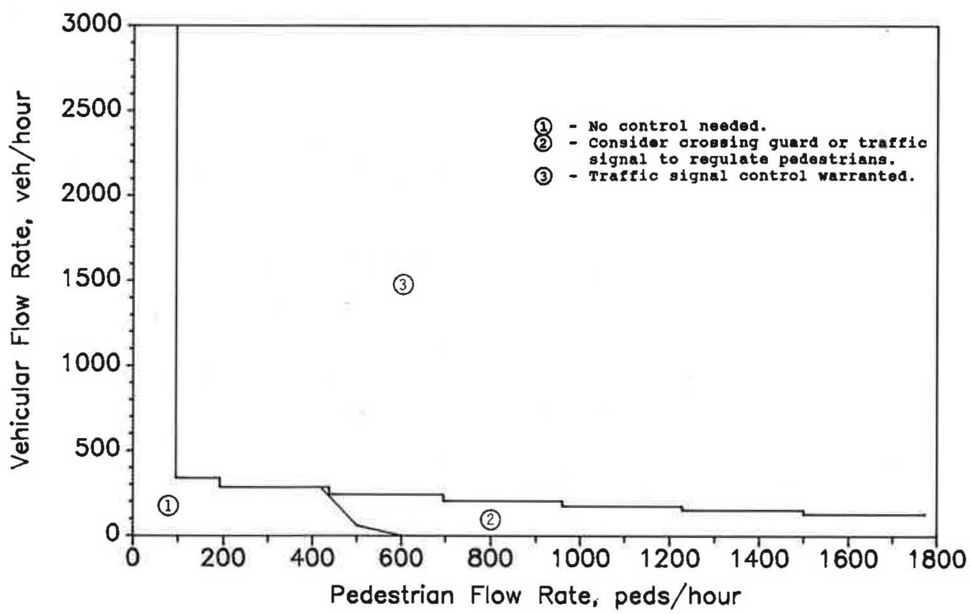

FIGURE 6 Pedestrian versus vehicular flow rate: 60-ft street width.

PROPOSED PROCEDURE

The following procedure is suggested for determining the need for traffic control at school crossings. It is intended as a substitute for the ITE field study procedure (*3*) and should be used only when the assumptions stated in the Statement of Problem section are valid. In any case, engineering judgment must be used to ultimately determine the need and type of traffic control to use at any school crossing.

The first step is to determine the period and duration of highest pedestrian activity (the minimum duration is 15 minutes) at the crossing of interest. This duration should be no longer than the period of pedestrian demand. Typical durations range from 30 minutes to 2 hours.

The second step is to determine the total daily pedestrian demand for the same crossing. If the minimum pedestrian flow rate and volume levels can be satisfied, then the analysis can proceed.

The third step is to identify the traffic volume that occurs during the same period previously identified (i.e., the period of highest pedestrian demand). Then, convert both the pedestrian and vehicular volumes to hourly flow rates and consult the figure (i.e., Figures 3–6) that most closely relates to the width of the roadway being crossed.

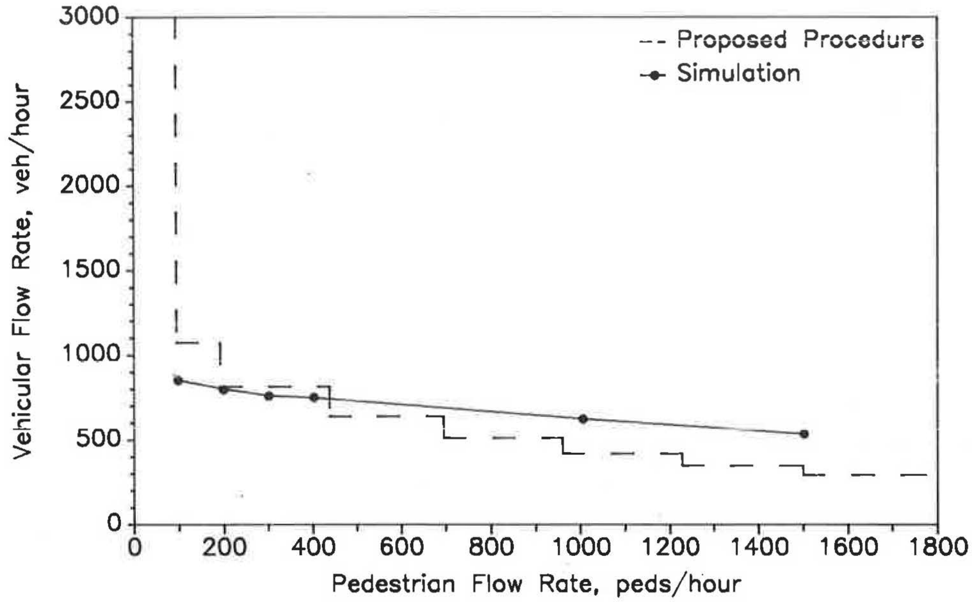

FIGURE 7 Procedure versus simulation results.

The last step is to find where the combination of flow rates intersect on the appropriate figure and determine the suggested traffic control for that location.

Other Considerations

Although this procedure is based on the assumption of random arrivals in the vehicular stream, this procedure can still be applied to streets with platooned arrivals. In general, platooned flows have longer, although less frequent, gaps for pedestrians to use. In most instances, these flows result in more opportunities for pedestrians to cross than would be suggested by an analysis based on random arrivals (*9*). As a result, the proposed procedure can also be used to determine traffic control needs on streets with vehicular progression—as long as it is recognized that the analysis is conservative.

Because the assumption of random arrivals is conservative, field studies are not necessary when the procedure indicates that no control is needed. Similarly, high vehicular and pedestrian volume combinations that fall well within the warrant region (i.e., region 3) would not justify a field study regardless of arrival distribution. The only time a field study should be considered is when the volume combinations just satisfy the proposed warrant for traffic control (based on an inadequate number of gaps) and it is known that the traffic stream has platooned arrivals. Under these circumstances, it is possible that an adequate number of traffic gaps are available in the platooned arrivals and that control is still not needed.

At locations where traffic signals exist within 300 ft along the street to be crossed, the relocation of the pedestrian crossing to the existing traffic signal should be considered before a pedestrian crossing is installed. If a traffic signal is recommended by this procedure, locating the signal at the nearest unsignalized intersection is preferred over a midblock location.

CONCLUSIONS

The purpose of this paper was to develop a procedure for selecting traffic signal control at school crossings. This procedure was developed to supplement and extend the procedure recommended by the MUTCD—Traffic Signal Warrant No. 4. Extensions to the MUTCD procedure include (a) the minimum daily pedestrian volume and flow rate criteria, and (b) a procedure for determining when pedestrian volumes are so high that they must be regulated to improve traffic flow.

The procedure presented in this paper is based on the current MUTCD warrant criteria for traffic control at school crossings. The ITE procedure used in evaluating this warrant requires field studies of pedestrian demand and traffic gap distribution, both of which can be time-consuming and expensive to conduct. In contrast, the proposed procedure eliminates the need for these complex field studies by using a probabilistic approach based on reasonable assumptions of vehicular and pedestrian arrival distributions. By using this procedure as a screening tool, local jurisdictions will be able to evaluate proposed school crossings in the office and eliminate those that would not satisfy the MUTCD warrant. This will enable jurisdictions to more efficiently allocate limited resources to other safety improvement programs by reducing the need for field study to just the "borderline" cases.

This paper has described the theoretical basis of the proposed procedure. Unfortunately, field verification and comparative assessment with the ITE procedure are beyond the scope of this paper. However, limited use with the proposed procedure combined with simulation results, indicate that the procedure will consistently yield results that limit pedestrian delays.

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Effects of Actuated Signal Settings and Detector Placement on Vehicle Delay

A. G. R. BULLEN

In this paper, the EVIPAS simulation and optimization model was used to analyze vehicle-actuated traffic signals. The variables studied were detector type and placement and the settings were for minimum green and vehicle extension. The evaluation criterion was minimum average vehicle delay. The study shows that the optimum design of a vehicle-actuated signal is specific for some variables but is relatively unaffected by others. The design is critical only for high traffic volumes. At low volumes, vehicle delay is relatively unaffected by the design parameters studied in this paper. The most critical variable is vehicle extension, particularly for passage detectors, where it should be at least 4.0 seconds regardless of detector placement and approach speed. For a presence detector, a short vehicle extension is recommended provided the detector is at least 60 ft in length. A length of 80 ft is preferred. For minimum green, the conventional design practice gives the best delay outputs.

It is generally accepted that a fully actuated traffic signal is almost always the most efficient form of signal control for an isolated intersection. The successful design of an actuated signal requires the specification of several critical parameters, including the type and placement of the detectors and the settings for the timing variables. Primarily, these factors are derived from considerations of driver behavior, vehicle characteristics, and safety. Within these constraints, however, there may be sufficient flexibility to allow traffic delay and associated vehicle operating costs to be considered.

This paper analyzes traffic delay at a fully actuated traffic signal as it relates to detector type and placement, and the settings for minimum green and vehicle extension. The analyses use the recently developed EVIPAS (*1*,*2*) simulation and optimization model for a vehicle-actuated traffic signal.

BACKGROUND

The design parameters for vehicle-actuated traffic signals have been extensively documented in many reports and publications. Summaries of these are available in the *Traffic Control Systems Handbook* (*3*), *NCHRP Report 233* (*4*), and the *Manual of Traffic Signal Design* (*5*).

The most commonly used detection arrangements are single passage and presence detectors. More sophisticated multiple detector arrangements for high-speed approaches are not the focus of this paper.

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Presence detectors usually range from 40 to 120 ft in length and are generally restricted to approach speeds of less than 30 mph. The minimum green is set from 0.0 to 4.0 seconds, and the vehicle extension is set from 0.0 to 3.0 seconds. Some literature provides a particular recommended minimum vehicle extension of 1.5 seconds, which ensures that drivers do not face an unexpected yellow when discharging.

Passage detector location is a function of the approach speed and the boundaries of the dilemma zone. Recommended distances range from 75 ft upward. The realized minimum green is set for the time required to evacuate the waiting vehicles within the detector distance. Generally this time is in the range of 12 to 14 seconds. The vehicle extension is the time for the vehicle to travel the distance from detector through the intersection. It can be a function of detector placement or can be set at a fixed value with detector placement being adjusted accordingly. A value of 3.5 seconds is frequently recommended.

The impact of these design values on signal performance and vehicle delay has been summarized in NCHRP 233 (*4*). Many of these results are based on somewhat simplified simulations and analytical procedures. Analyses of presence detectors show that a 60-ft length gives minimum delay, whereas for passage detectors a setback of 150 ft gives minimum delay.

The major timing variable related to vehicle delay is vehicle extension. For passage detectors, vehicle delay is shown to increase for very short or very long vehicle extensions. Generally, it is recommended that vehicle extensions should be kept as short as possible to give "snappy" signal operation and this recommendation is repeated in other literature. NCHRP 233 (*4*) point outs, however, the potential difficulty with early green termination because of variable queue discharge headways, the effects of which were not analyzed in that publication.

STUDY METHODOLOGY

The EVIPAS vehicle-actuated traffic signal model provides a mechanism for analyzing all of the factors described above in some detail. The effect on vehicle delay of variations in these parameters was studied for a variety of situations.

The EVIPAS model was field tested at ten vehicle-actuated traffic signals ranging from two to eight phases. Comparisons of stopped delay by approach lane were within 15 percent, which is within the statistical variation shown by traffic flow. The computer model also reproduced within 5 percent, the field observations of average cycle length and average phase length.

To keep the analyses fairly simple, only two phase signals on intersections with one- or two-lane approaches were studied. With one exception, only single detectors were considered.

Traffic volumes were varied from 350 vehicles per lane per approach per hour to 750 vehicles per lane per approach per hour. Passage detectors were located from 50 to 250 ft, and presence detectors of 40 to 120 ft in length were used. Approach speeds of the traffic ranged from 25 to 55 mph.

The minimum green values ranged from 0.0 to 5.0 seconds for presence detectors and from 5.0 to 20.0 seconds for passage detectors. The vehicle extensions ranged from 0.0 to 5.0 seconds for presence detectors and from 1.5 to 9.0 seconds for passage detectors. To minimize random variations, each simulation run simulated 4 hours of real time.

Whenever minimum green is discussed, it is the realized minimum green, i.e., the set value plus one vehicle extension for those controllers that operate in this mode.

STUDY RESULTS

Traffic Volume

The simulation results consistently showed that the design parameters are much more important at high volumes than at medium to low volumes. These results indicate that a signal designed to handle the peak period will be satisfactory for the off-peak periods. High volumes were considered to be 750 vehicles per hour per lane per phase, whereas low volumes were 350 vehicles per hour per lane per phase.

These characteristics regarding volume, which are detailed further below, indicate that as an intersection approaches capacity, the design of the actuated signal is critical. Slight design errors can lead to significant increases in delay and operating cost. At medium to low volumes, however, a considerable design variation can be tolerated and even a poorly designed signal can operate well.

Presence Detector Length

Provided that the timing variables were set correctly, the length of a presence detector can vary from 60 to 100 ft with little effect on delay at high volumes and low to moderate approach speeds. Figure 1 is an example of the variation of delay with detector length. At lengths below 60 ft, delay starts to increase significantly. This result is similar to that in NCHRP 233 (*4*) except that it specifically shows 60 ft to be the most efficient, whereas in this study 80 ft was found to be slightly more effective.

Passage Detector Location

Figure 2 shows the variation of delay with detector location. In the range of 100 to 200 ft, there is little effect on delay,

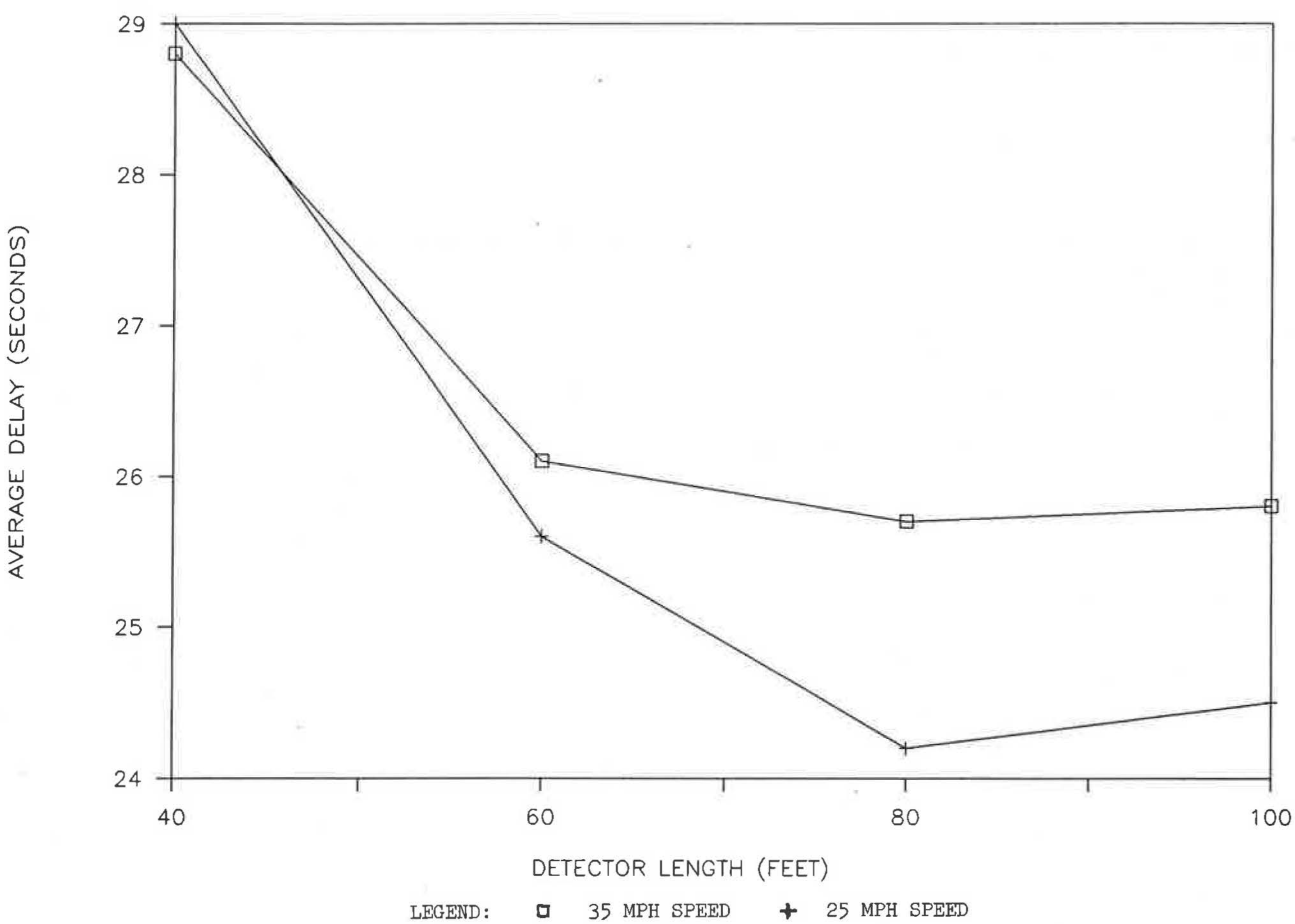

FIGURE 1 Presence detector length.

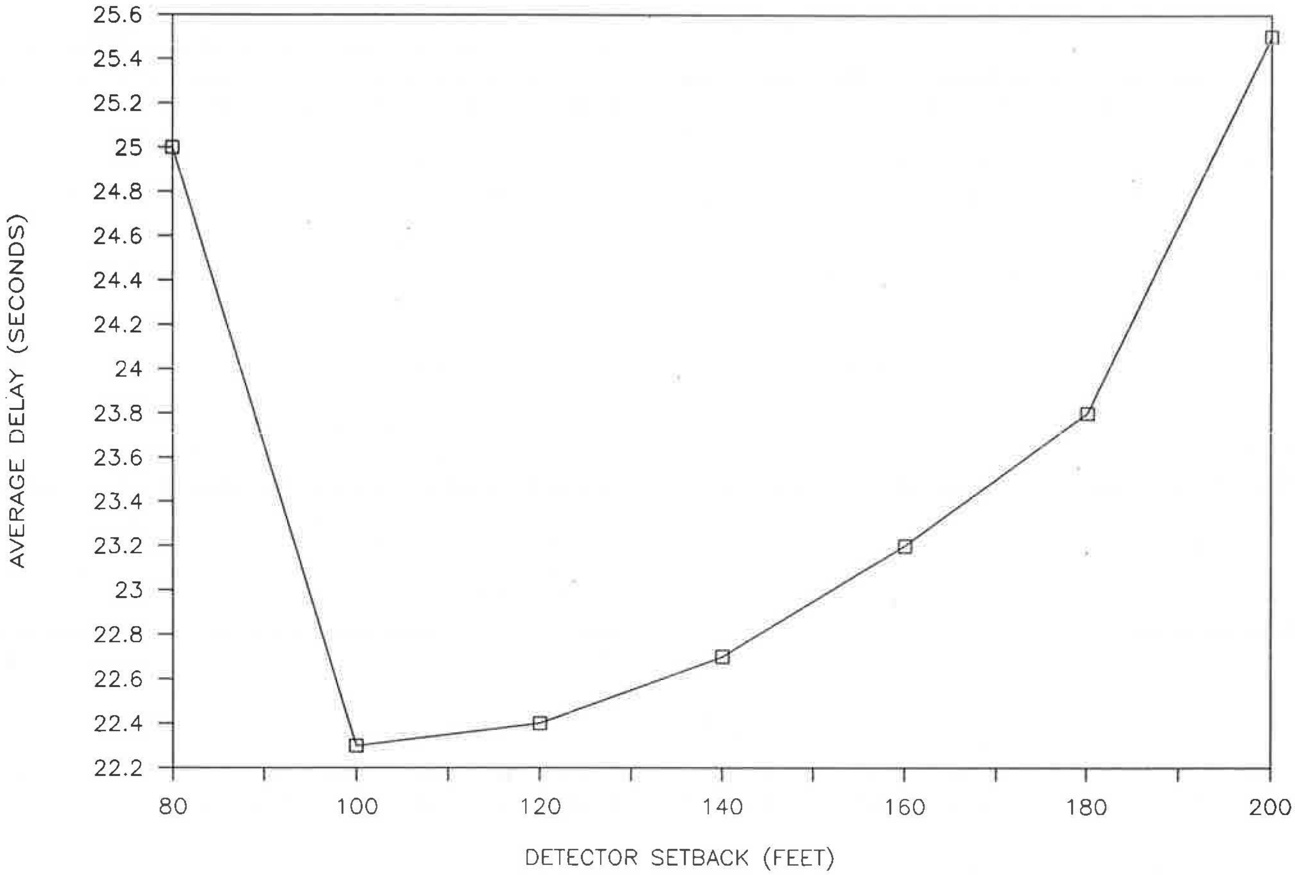

FIGURE 2 Passage detector setback.

provided the timing settings are optimized. Above 200 ft, the delay increases partly because the required minimum greens become large. Below 100 ft, the delay also starts to increase more rapidly. These results are somewhat more flexible than those from NCHRP 233 (*4*), in which a setback of 150 ft is specifically recommended for greatest efficiency.

The analysis of approach speed showed that detector location is not affected by this parameter. This conclusion, however, applies only to the minimum delay criteria and does not include driver behavior or safety.

Vehicle Extension: Presence Detectors

The study clearly shows that vehicle extension is the most critical variable for minimizing delay. For detectors greater than 60 ft in length, a vehicle extension of 0.0 seconds gives minimum delay, provided the minimum green is set correctly. Figure 3 shows the variation in delay with vehicle extension. For a 40-ft detector, which is not recommended, a vehicle extension of 1.5 to 2.0 seconds is required. Trucks in the traffic stream will modify these results by increasing the required vehicle extension.

Studies of delay for moderate and low traffic volumes indicated that efficiency is relatively insensitive to vehicle extension.

Vehicle Extension: Passage Detectors

The results of these studies of vehicle extension contradict to some extent the desires expressed in the literature for short snappy signal phases through short vehicle extensions. This difference is because the EVIPAS program has variable queue discharge headways and therefore models the penalty associated with the early green cutoff within a discharging queue.

The general relationship is illustrated in Figure 4. This figure shows delay as a function of vehicle extension for various volumes of passenger cars only. The pattern shows an optimum vehicle extension of 4.0 seconds. As the vehicle extensions increase, the increase in delay is moderate and the penalty for higher vehicle extensions is not severe even for high volumes. Below the optimum value, however, the increase in delay is sharp and rapidly becomes infinite. The penalty for a short vehicle extension is severe. Thus, the vehicle extension should always be a little on the high side rather than a little on the low side. This pattern of a delay curve that decreases rapidly and then increases slowly is typical of the relationship regardless of detector location, vehicle approach speed, or minimum green.

Figure 5 indicates an analysis in which two passage detectors are located in the lane at 75 and 150 ft. Although this arrangement should indicate a vehicle extension only half that of the

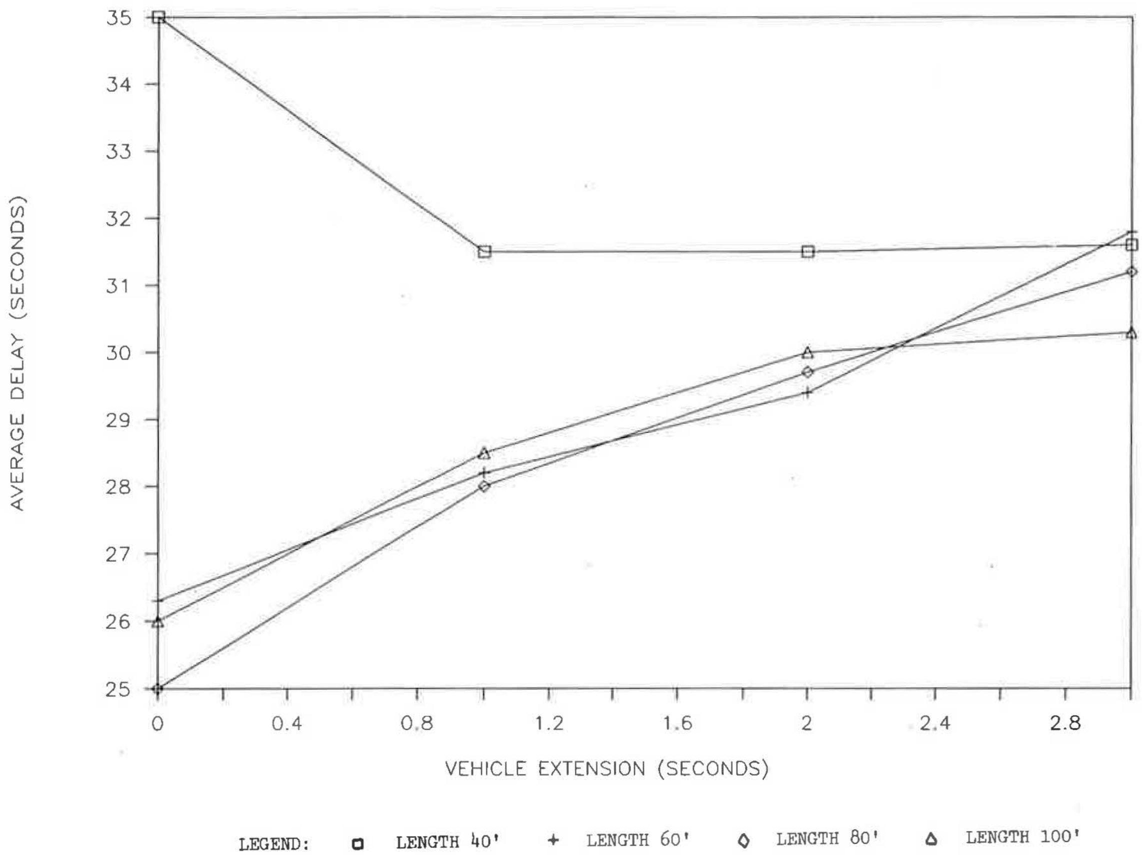

FIGURE 3 Presence detector extension.

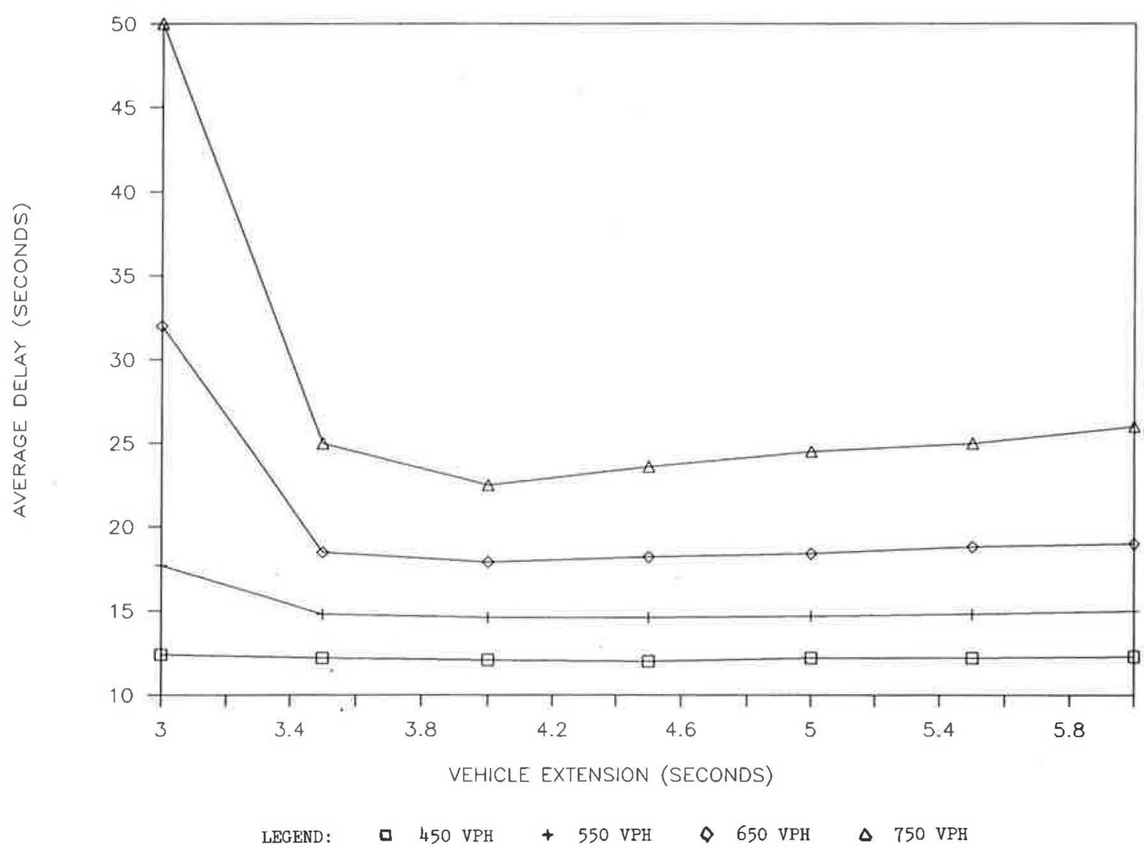

FIGURE 4 Vehicle extension and volume.

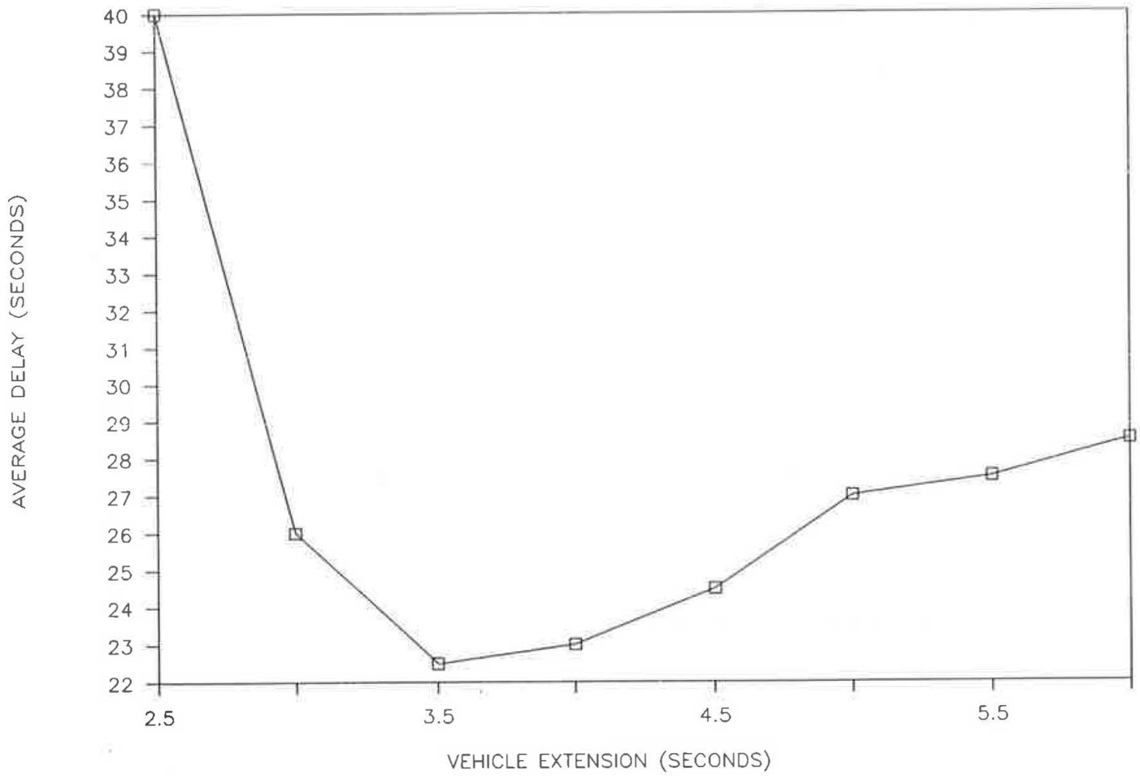

FIGURE 5 Two passage detectors.

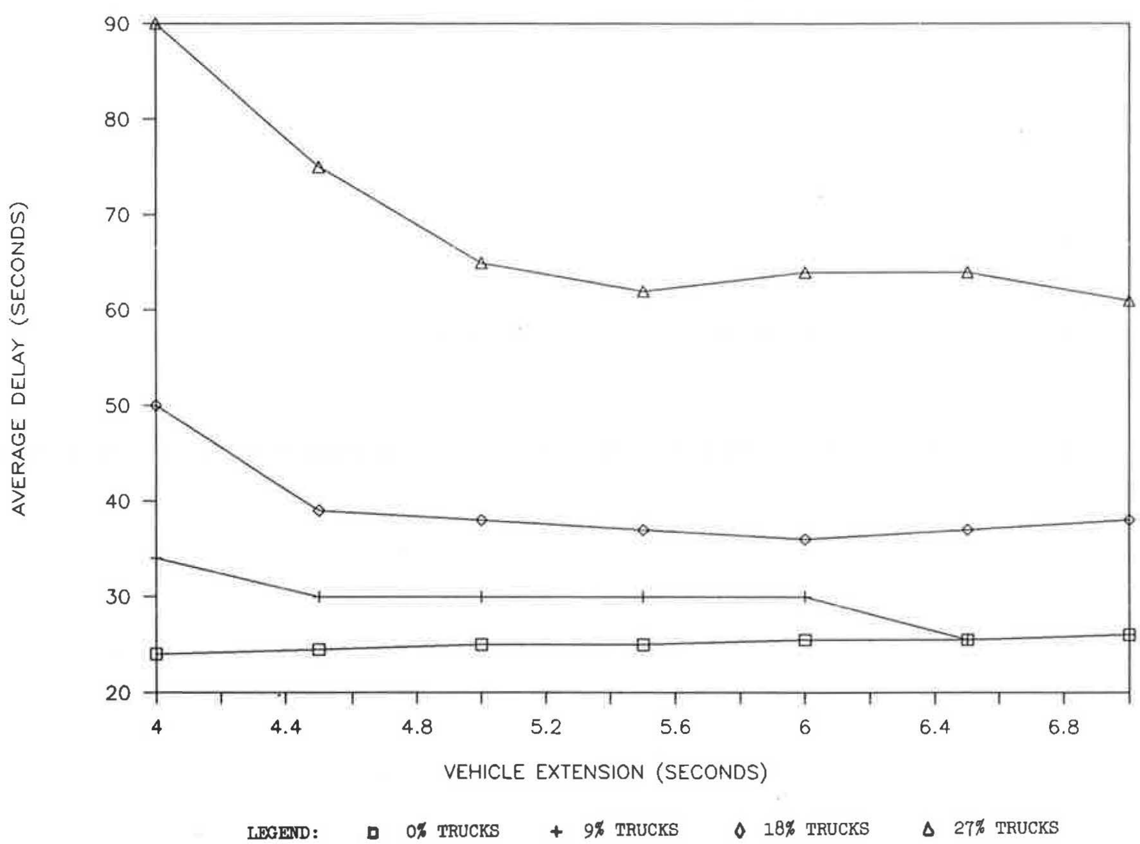

FIGURE 6 Effects of trucks.

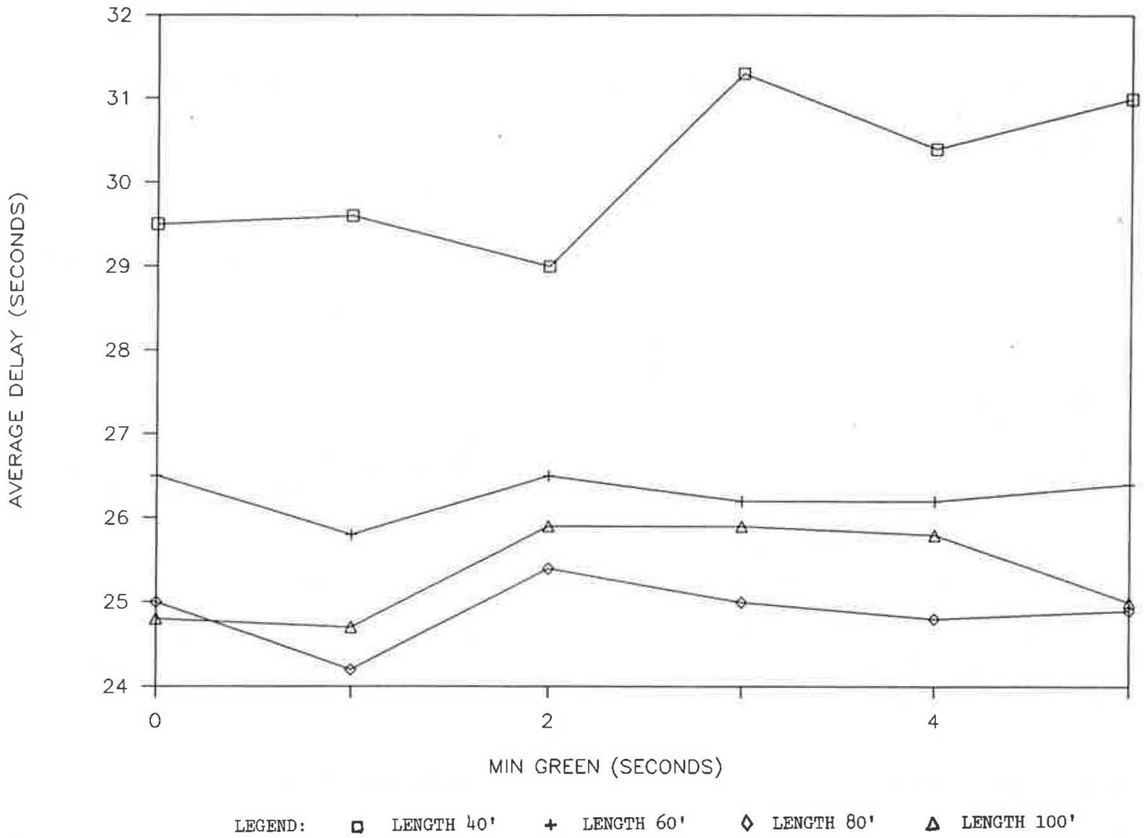

FIGURE 7 Presence detector, minimum green.

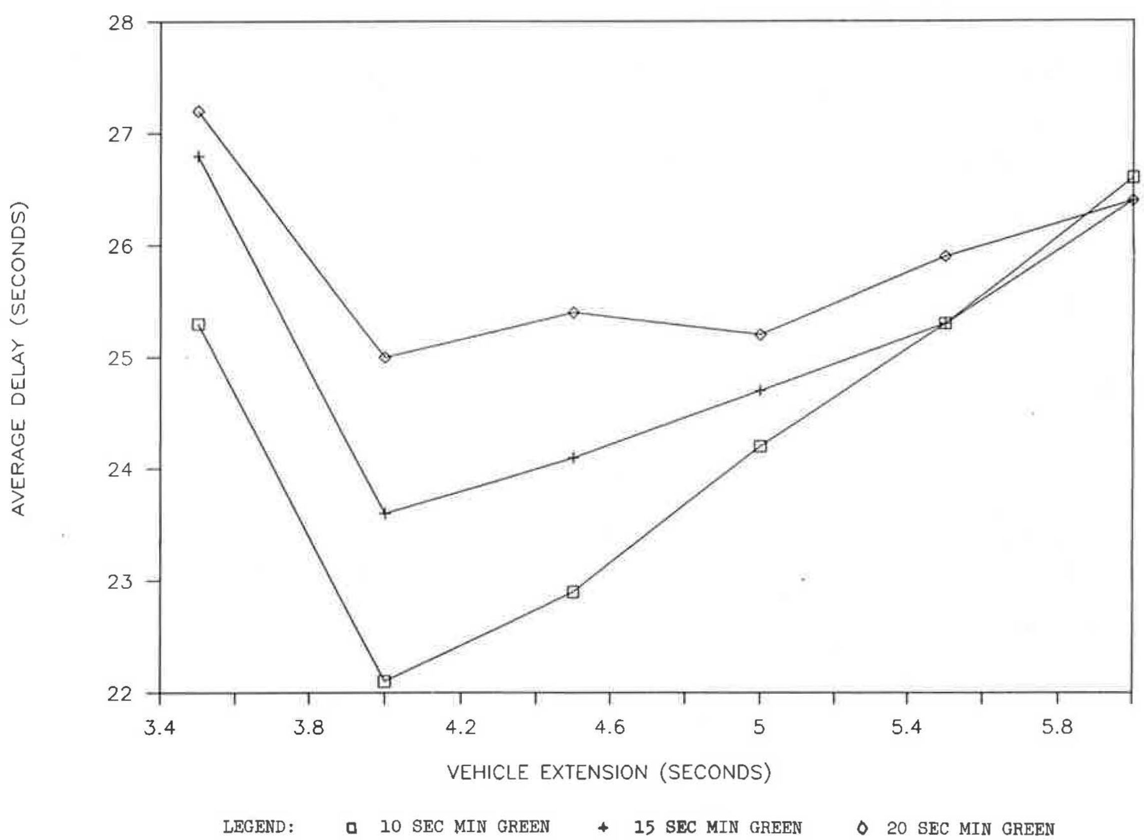

FIGURE 8 Passage detector, minimum green.

value for a single detector at 150 ft, the output is almost the same. An optimum vehicle extension of 3.5 seconds prevails with a similar delay curve structure. There is a severe penalty for short vehicle extensions. All of these results are critical only for high volumes. For lower traffic flows the vehicle extension, as with other design parameters, is relatively insensitive.

These general results for vehicle extension suggest that the efficiency of a vehicle-actuated signal is dominated by the distribution of arrival headways and the distribution of queue discharge headways. The problem of variable queue discharge headways grows when trucks are present because of their lower acceleration and longer vehicle length. The best vehicle extension should, therefore, be higher for higher truck percentages. This relationship is illustrated in Figure 6 in which the optimum vehicle extension increases as the trucks increase. Regardless of detector location, vehicle extensions of as much as 6.0 seconds may be needed for locations with heavy truck traffic.

The recommendation found in some literature to set the vehicle extension at 3.5 seconds and then to design the other parameters to this is somewhat low for delay optimization.

Minimum Green: Presence Detectors

Figure 7 shows how delay varies with minimum green. This variable is not very sensitive, although it appears that under most circumstances a minimum green of 1.0 to 3.0 seconds is appropriate.

Minimum Green: Passage Detectors

The minimum green usually is set to clear the vehicles waiting within the detectors, and for normal detector locations this setting seems to give satisfactory efficiency. This parameter does not have a significant effect on delay, although short minimums do lead to higher delays. Generally, in the recommended range for detector location of 100 to 200 ft, a normal minimum green calculation based on the detector distance gives a satisfactory result. Figure 8 shows the variation of delay with minimum green and vehicle extension.

CONCLUSIONS

The optimum design of a vehicle-actuated signal for the objective of minimizing vehicle delay is specific for some parameters but allows considerable flexibility for others.

The design and timing of the signal are critical only for high traffic flows, whereas considerable flexibility exists for low to moderate volumes. A signal designed and set to minimize delay for the peak hours, therefore, will be close to the optimum for off-peak periods. At a signal that has only moderate traffic flows (level of service B or better), the traffic delays will not be seriously affected by any reasonable detector locations or timing. In these circumstances the design criteria should be safety and driver characteristics.

For heavy traffic flows, the vehicle extension is the most critical variable affecting delay. For passage detectors the vehicle extension should be at least 4.0 seconds, but if trucks are present up to 6.0 seconds may be needed. Any variation on the low side will lead to rapidly increasing delays, whereas the penalty is less severe on the high side. For presence detectors, the vehicle extension should be as short as possible provided the detector is at least 60 ft long and there are few trucks.

The minimum green should be in the range of 1.0 to 3.0 seconds for presence detectors. For passage detectors, current design practice is satisfactory. Detector location does not have a significant effect on delay provided the locations are within 60 to 100 ft for presence detectors and 100 to 200 ft for passage detectors.

The location of detectors and the timing of key variables can be carried out with full considerations of traffic safety and driver characteristics and behavior. Within this framework there is still enough flexibility to achieve maximum operating efficiency.

ACKNOWLEDGMENTS

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Field Evaluation of a Four-Quadrant Gate System for Use at Railroad-Highway Grade Crossings

K. W. HEATHINGTON, DANIEL B. FAMBRO, AND STEPHEN H. RICHARDS

As part of research to identify and evaluate innovative active warning devices with the potential for improving safety at railroad-highway grade crossings, candidate devices were identified and developed, and the most promising devices were evaluated in detailed laboratory studies. Based on the results of the laboratory evaluation, three devices were evaluated in the field at actual crossings. One of the innovative active warning devices evaluated in the field was a four-quadrant gate and flashing light signal system with skirts. A before-and-after study approach was used to evaluate the four-quadrant gate system. Data were collected on measures of effectiveness (MOEs) at the existing crossing with the standard two-quadrant gate system and then again at the same crossing after the four-quadrant gate system had been installed to allow a direct comparison of the impact on the MOEs. With the installation of the four-quadrant gate system, MOEs such as speeds, perception-brake reaction times, and deceleration levels did not indicate a change in driver behavior. There were no measurable safety disadvantages to the four-quadrant gate system as measured by these MOEs. The four-quadrant gate system had no effect on the level of service at the crossing but had a positive effect on driver behavior at the crossing by eliminating risky and illegal behavior as well as violations at the crossing, thus producing superb improvements in safety MOEs. Such benefits are especially important at crossings with limited sight distance, high-speed trains, and multiple tracks.

During the 10-year period from 1977 through 1986, injuries and fatalities resulting from motor vehicle accidents at railroad-highway grade crossings have decreased from 4,452 and 846 to 2,227 and 507, respectively (*1*). Much of this safety improvement may be attributed to the availability of federal funds for grade crossing improvement projects (*2*). The majority of the federal funding has been used to upgrade passive crossings to active ones and has resulted in equipping in 1986 over one in four of the 192,454 public grade crossings with active warning devices. In 1986, 22,066 crossings (11.5 percent) were equipped with automatic gates and 32,778 crossings (17.0 percent) were equipped with flashing light signals (*1*).

Even with these improvements, over 50 percent of all car-train accidents in 1986 occurred at crossings with active warning devices, which represents only 28.5 percent of the crossings (*1*). Although this apparently high number of accidents may be a result of higher vehicle and train volumes and more complex railroad-highway geometrics at active crossings, it is likely that some of the accidents are caused by motorists either not seeing or not understanding the active warning devices presently used at railroad-highway grade crossings (*3*,*4*). Therefore, it seems that these active traffic control devices could be improved. Although research to improve safety at railroad-highway grade crossings has been going on for some 50 years, the traffic control devices used for warning motorists of impending danger at a crossing have not changed significantly. During this time, many innovative warning devices have been developed for use both at and in advance of crossings, yet field implementation of new concepts has been minimal.

Recognizing the need to fully address the issues and problems concerning active warning devices at railroad-highway grade crossings, the FHWA sponsored a research project to identify and evaluate innovative active warning devices with potential for improving safety at railroad-highway grade crossings. As part of the research, candidate devices were identified and developed, and the most promising devices were evaluated in detailed laboratory studies (*5*,*6*). Based on the results of the laboratory evaluation, three of the devices were evaluated in the field at actual crossings (*7*). One of the innovative active warning devices evaluated in the field was a four-quadrant gate and flashing light signal system with skirts. Before field evaluation of the four-quadrant gate and flashing light signal system with skirts, an on-site visit was made to several European installations of similar design. These systems were found to have operated satisfactorily for many years (*8*).

FIELD EVALUATION PLAN

Study Approach

A before-and-after study approach was used to evaluate the four-quadrant gate system. That is, performance data were collected at the existing crossing with the standard two-quadrant gate system and then again at the same crossing after the four-quadrant gate system had been installed. This approach allowed a direct comparison between the four-quadrant gate system and the two-quadrant gate system currently used at the crossing.

The first set of crossing studies on the two-quadrant gate system was conducted in the spring of 1985. The four-quadrant gate system was then installed in October 1985. After a 1- to 2-month familiarization period, the second set of studies was

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conducted in December 1985 and January 1986. The purpose of this delay was to ensure that the behavioral data being collected did not contain driver responses caused by unfamiliarity with the new device.

Measures of Effectiveness

Realistic field evaluation of the four-quadrant gate system was dependent on selection of suitable measures of effectiveness (MOEs). To avoid influencing drivers and hence influencing their responses, MOEs were selected that could be obtained with a minimum of interference and detection by drivers. In addition, only commonly accepted, safety-oriented driver performance measures were considered. As a result of these considerations, the MOEs selected for evaluation were as follows: (a) speed profiles, (b) perception-brake reaction times (PBRTs), (c) maximum deceleration levels, (d) violations, and (e) vehicles crossing.

The general hypotheses tested in the field studies were that when compared to the two-quadrant gate system, the four-quadrant gate system would result in: (a) quicker driver PBRTs, (b) fewer undesirable and uncomfortable decelerations, (c) fewer violations, and (d) fewer vehicles crossing in front of the train. Thus, the overall null hypothesis was that there was no difference in driver performance measures when comparing response to the two-quadrant gate system with response to the four-quadrant gate system.

Speed Profiles

Speed profile data were evaluated for the four-quadrant gate system and compared with similar data collected before installation of the device (i.e., under the existing conditions). In addition, a maximum deceleration level was computed from each individual speed profile. These values were then tabulated and plotted as a cumulative frequency distribution. The number of drivers accepting an undesirable level of deceleration (> 8 ft/sec^2) was also used for evaluation purposes. In each of the previously described comparisons, the Kolmogorov-Smirnov goodness of fit test was used to determine whether or not any observed differences in distributions were statistically significant (*9*).

Perception-Brake Reaction Time

PBRT was defined as the difference in time between activation of the warning device and activation of the vehicle's brake lights. This measure was computed by counting the number of frames on the videotape between these two points in time and dividing by 60 (60 frames per second). Only those vehicles whose brake lights were activated were included in the data set. Since the observations were not necessarily expected to be normally distributed, nonparametric techniques in the Statistical Analysis Systems program were used to ascertain whether or not observed differences were statistically significant (*10*).

Violations

Violations were compared for the two-quadrant and four-quadrant gate systems. Violations occurred whenever motorists either drove around the gate arm in the down position or collided with the gate arms as they were coming down. The number of violations that occurred for each train crossing were manually counted from videotapes. The analysis procedure for this measure was exactly the same as that for PBRTs.

Vehicles Crossing

The number of vehicles crossing was the final MOE used to evaluate the relative performance of the four-quadrant gate system. This measure was defined as the total number of vehicles crossing the tracks between activation of the warning device and the train's arrival at the crossing. The number of vehicles crossing were manually counted from the videotapes and then, for comparison purposes, subdivided into those that occurred within 10 and 20 seconds of the train's arrival at the crossing. Specifically, vehicles that crossed within 10 seconds of an oncoming train (CL10) were considered an indication of risky behavior as this represents a level of driver performance in which there is little, if any, room for error. This value was based on 2.5 seconds of perception reaction time, a 20-ft-long vehicle starting from a stop 20 ft away from the crossing, accelerating at a normal rate of 4.8 ft/sec^2, and clearing a point 20 ft on the far side of the crossing 2.5 seconds before the train's arrival. Vehicles that crossed within 20 seconds of an oncoming train (CL20) were considered an indication of aggressive behavior since this was thought to represent a level of driver performance in which there is some, but not much, room for driver, vehicle, or warning system error. The *Manual on Uniform Traffic Control Devices* (MUTCD) appears to address this point by requiring a minimum warning time of 20 seconds (*11*).

Data Collection and Reduction

The key to determining motorist response to the activation of an active warning device was to obtain accurate and pertinent data on driver behavior in the decision zone, i.e., that area in which the driver must decide to either stop or proceed through the crossing. Data were automatically recorded on portable video recorders whenever a train was approaching the crossing and partially reduced by an image processing and pattern recognition process.

Video Recording System

Three complete video recording systems were used for the field studies. Each system could be operated on rechargeable storage batteries or, with the appropriate adaptor, from either a 110-volt AC or 12-volt DC power source. The recorders were portable and used standard 0.5-in. T120 VHS cassettes and could operate in temperature and relative humidity ranges of approximately 32° to 104°F and 35 to 80 percent, respectively.

The video cameras used with the recorders were black and white closed circuit television cameras that weighed 2 lb each.

They used vidicon tubes with an automatic light range of 100,000 to 1, thus providing high-quality video under both day and night lighting conditions. The cameras operated on 12 volts DC and used the recorders as a power source; therefore, they were energized only when the recorders were activated. The cameras would operate from 0° to 140°F and 0 to 95 percent relative humidity.

Detection System

It was necessary to obtain a train presence signal in advance of the railroad's train detection signal to record the events immediately before the activation of the warning device. For this reason, a train detector was used that emitted an infrared light beam and detected its return from a reflector located across the tracks. When a train broke the beam, the detector transmitted an encoded camera activation signal followed by an audio timing signal. Detectors were located on each approach to the crossing such that the activation signal was transmitted at least ten seconds before the train activated the active warning device at the crossing.

Equipment Setup

Each camera was located at as high an elevation and as far from the centerline of the roadway as possible. Physical constraints limited the mounting height to about 20 ft and the lateral distance to about 60 ft; therefore, three 20-ft mounting poles were built. All three video recording systems and mounting poles were used at each field site. The first unit was located approximately 300 ft from the crossing, the second approximately 500 ft from the crossing, and the third approximately 700 ft from the crossing. The cameras were aimed toward the crossing and had overlapping fields of view.

FIELD SITE SELECTION AND STUDY PREPARATION

For a crossing to be considered for evaluating the four-quadrant gate system, it was necessary that it have a relatively high train and traffic volume, have a history of at least some accidents, and have a two-quadrant gate system already in place. Favorable response for use of a crossing with these characteristics in the Knoxville area was received from the Norfolk Southern Railroad.

The crossing (inventory number 730584K) selected for the four-quadrant gate system was located in the eastern part of Knoxville on Cherry Street. It was ranked as the 223rd most dangerous crossing in the state. As shown in Figure 1, the roadway was four lanes wide and straight and level on both approaches to the crossing. There was a building in the southwest quadrant that could obstruct a northbound driver's view of eastbound trains. The average daily traffic at this site was approximately 14,000 vehicles per day, and the average through-train volume was approximately 20 trains per day. The speed limit on Cherry Street was 30 mph, and train speeds at the crossing ranged from 20 to 40 mph. Although only one car-train accident had occurred at this location in the past 5 years, large numbers of motorists were observed driving around lowered gate arms at this site. This type of behavior made the Cherry Street crossing a potentially dangerous crossing.

Implementation of the four-quadrant gate system required the installation of an additional pole, electric motor, and gate arm support and counterweights in both the southwest and northeast quadrants. A 30-ft gate arm was then attached to each of the four support arms. Thus, the entire roadway was blocked in both directions whenever the gates were down. To avoid the possibility of trapping vehicles between the gates, a delay in the downward motion of the offside gate arm was incorporated into the system. In addition to the changes in the gate arms, railroad flashing light signals with 12-in. roundels were installed in all four quadrants. The existing bell and railroad advance warning signs were left as they were; however, the pavement markings were repainted so as to be more visible to approaching motorists. Figure 2 shows the four-quadrant gate system installed at the Cherry Street crossing.

RESULTS FROM FIELD EVALUATION

A large number of motorists disregarded the standard two-quadrant gate system at the Cherry Street crossing by driving around lowered gate arms. This behavior was not only illegal; it was dangerous. The primary change in driver performance that was expected as a result of the installation of the four-quadrant gate system was the elimination of this type of behavior. As a result of this expected change in behavior, the average clearance time between the last vehicle to cross and the train's arrival at the crossing should also increase. Both behavioral modifications have implied safety benefits because they provide greater spatio-temporal separation between trains and motor vehicles. The anticipated secondary change in driver

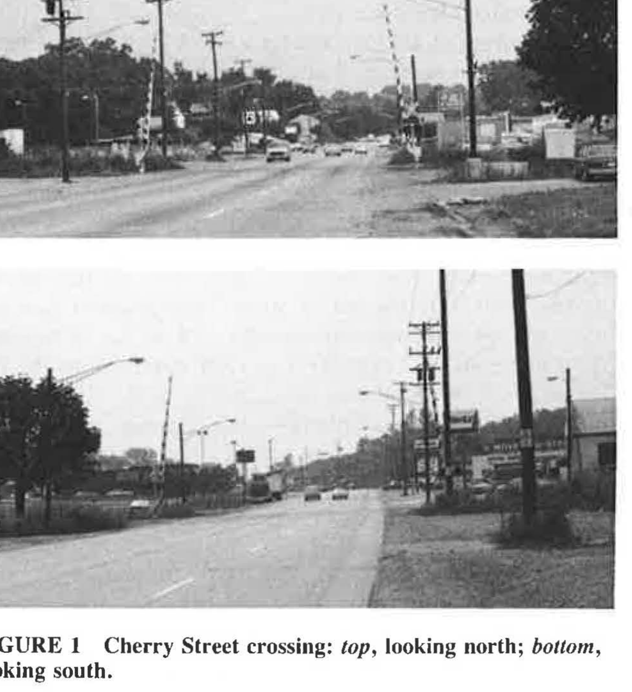

FIGURE 1 Cherry Street crossing: *top*, looking north; *bottom*, looking south.

FIGURE 2 Prototype of four-quadrant gate system installed at the Cherry Street Crossing: *top*, gate arms upright; *bottom*, gate arm and skirt assembly.

performance was better response to the new device (i.e., quicker PBRTs and lower deceleration levels) as a result of its greater conspicuity and more formidable appearance; however, differences in these performance measures were not expected to be as easy to quantify, and the related safety benefits were not as straightforward.

The four-quadrant gate system was installed at the Cherry Street railroad-highway grade crossing during the week of October 14, 1985. Before this time, the active warning device at the crossing was a standard two-quadrant gate system. Both train movement and driver behavior data were collected for approximately 2 months before (March and April 1985) and 2 months after (December 1985 and January 1986) the new devices were installed. During these two time periods, 169 train movements were observed. There were 105 train movements observed in the before study (two-quadrant gate system) and 64 train movements observed in the after study (four-quadrant gate system). For each observation, the following were recorded and subsequently analyzed: environmental and lighting conditions; train's direction of travel and warning time; and approaching vehicles' clearance times, speed profiles, and brake reaction times.

Crossing Measures

Warning Time

Warning time was defined as the difference in time between activation of the flashing light signals and the train's arrival at the crossing. It is the same as the maximum time a motorist would have to wait between activation of the flashing light signals and a train's arrival at the crossing. As there were no changes to the train detection system itself when the four-quadrant gate system was installed, there should have been no difference in the average warning time observed in the two studies. To verify this premise, the total data set from each study was first subdivided into observations that occurred during the day and observations that occurred during the night to ensure that similar train and traffic volume conditions were compared. These two subsets, together with the total data set, were then analyzed.

As shown in Table 1, the mean and standard deviation of the warning times from the data sets were numerically slightly less during the after study (four-quadrant gate system); however, the Mann-Whitney U test for two independent, continuously distributed populations indicated that these differences were not statistically significant at the 95 percent confidence level for either the day, night, or total data sets (*12*). This means that, as expected, installation of the four-quadrant gate system had no effect on the warning times at the crossing. The Mann-Whitney U test also indicated that there was not a statistically significant difference at the 95 percent confidence level between the day and night data sets from either of the two studies. Thus, warning times were not different during day and night operation for either the two-quadrant gate or the four-quadrant gate system.

It was hypothesized that the warning times observed at a railroad-highway grade crossing have a major influence on driver performance, i.e., the longer the warning times, the larger the number of drivers who will exhibit dangerous or illegal behavior. Unfortunately, there was no method in the literature for assessing the adequacy of the warning times at a railroad-highway grade crossing from the driver's perspective; however, level-of-service concepts have been well established in the highway field for the past 30 years. As a result, level-of-service criteria, similar to those for signalized intersections in the 1985 *Highway Capacity Manual*, were developed for active warning devices at grade crossings (*7,13*). The criteria developed are shown in Table 2 (*7*). The levels of service are based on the premise that a grade crossing is similar to a signalized intersection, albeit that one interrupts vehicular flow only a few times each day. This is not an unreasonable assumption given the fact that at both a signalized intersection and a railroad-highway grade crossing, drivers are primarily concerned with how long they have to wait.

As shown in Table 2, 20 seconds is the minimum warning time currently required by the MUTCD, and 60 seconds is defined by the 1985 *Highway Capacity Manual* as the limit of acceptable delay to most motorists (*11,13*). These two points clearly define the limits of adequate or acceptable motorist service, i.e., warning times less than 20 seconds are inadequate (as currently defined by the MUTCD), and warning times greater than 60 seconds are unacceptable and defined as level of service F. The 40-second range between these two limits was subdivided in 10-second increments so as to create four warning time categories for levels of service A, B, C, and D. As can be seen from Table 1, by using these definitions, the majority of the warning times observed in both studies could be classified as level of service D or better—65.7 percent in the before study (two-quadrant gate system) and 73.5 percent in the after study (four-quadrant gate sys-

TABLE 1 WARNING TIMES AT THE CHERRY STREET CROSSING[1]

	Two-Quadrant Gates			Four-Quadrant Gates with Skirts		
Summary Statistics	Day	Night	Total	Day	Night	Total
Sample Size	71	34	105	30	34	64
Mean (seconds)	55.81	61.49	57.65	51.64	60.06	56.11
Standard Deviation	14.05	19.80	16.26	8.58	16.16	13.73
Range (seconds)	30-106	29-118	29-118	38-75	30-94	30-94

	Two-Quadrant Gates			Four-Quadrant Gates with Skirts		
Warning Time (seconds)	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
<20	0	0.0	0.0	0	0.0	0.0
20-30	2	1.9	1.9	0	0.0	0.0
30-40	12	11.4	13.3	2	3.1	3.1
40-50	16	15.2	28.5	23	36.0	39.1
50-60	39	37.2	65.7	22	34.4	73.5
60-90	33	31.4	97.1	16	25.0	98.5
>90	3	2.9	100.0	1	1.5	100.0
Total	105			64		

[1]Time between activation of flashing lights and train's arrival at the crossing.

TABLE 2 PROPOSED LEVEL-OF-SERVICE CRITERIA FOR RAILROAD HIGHWAY GRADE CROSSINGS (*7*)

Level of Service	Warning Time Category	Time Before Train's Arrival[a] (sec)
—	Inadequate[b]	<20
A	Desirable	20–30
B	Marginal	30–40
C	Poor	40–50
D	Maximum	50–60
F	Unacceptable[c]	>60

[a]Average time (in seconds) between activation of the flashing light signals and the train's arrival at the crossing.
[b]20 seconds is the minimum warning time allowed by the MUTCD.
[c]60 seconds is the limit of acceptable delay to most motorists as defined by the 1985 *Highway Capacity Manual.*

tem). However, a much smaller percentage of the warning times observed could be classified as level of service C or better—28.5 percent in the before study and 39.1 percent in the after study. This relatively small percentage of warning times less than 40 seconds and the 34.3 percent of the warning times that were classified as level of service F (unacceptable) might explain why so many motorists drove around the lowered two-quadrant gate arms. In other words, the warning times were perceived as unacceptable (too long) and the motorists performed in an unacceptable (dangerous and illegal) manner by driving around the lowered two-quadrant gate arms.

Clearance Time

Clearance time was defined as the difference in time between the last vehicle to cross and the train's arrival at the crossing. Since the four-quadrant gate system prohibits driving around the gate arms by physically blocking the roadway, their installation should result in significantly longer clearance times. In other words, if motorists could drive around the gate arms, they could cross closer to the train's arrival at the crossing. This additional temporal separation between cars and trains is a definite safety benefit of the four-quadrant gate system.

Clearance times were recorded only for those arrivals in which a vehicle arrived at the crossing between the activation of the flashing light signals and the train's arrival at the crossing; there was an opportunity for a vehicle to cross in front of the train. Thus, the number of clearance times will always be less than or equal to the number of train arrivals. As shown in Table 3, 90 clearance times were observed in the before

TABLE 3 CLEARANCE TIMES AT THE CHERRY STREET CROSSING[1]

Summary Statistics	Two-Quadrant Gates Day	Night	Total	Four-Quadrant Gates with Skirts Day	Night	Total
Sample Size[2]	70	20	90	18	11	29
Mean (seconds)	23.96	26.62	24.55	44.39	56.27	48.90
Standard Deviation	11.18	17.23	12.71	9.10	16.27	13.39
Percent <20 Seconds	41.4	35.0	40.0	0.0	0.0	0.0
Percent <10 Seconds	5.7	5.0	5.6	0.0	0.0	0.0
Range (seconds)	6-62	4-72	4-72	34-68	34-81	34-81

Clearance Time (seconds)	Two-Quadrant Gates Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Four-Quadrant Gates with Skirts Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
<10	5	5.6	5.6	0	0.0	0.0
10-20	31	34.4	40.0	0	0.0	0.0
20-30	36	40.1	80.1	0	0.0	0.0
>30	18	19.9	100.1	29	100.0	100.0
Total	90			29		

[1]Time between last vehicle to cross and the trains arrival at the crossing.

[2]Includes only those observations in which vehicles were present before the train's arrival.

study (two-quadrant gate system) and 29 clearance times were observed in the after study (four-quadrant gate system). As with the warning time data set, the total data from each study were subdivided into observations that occurred during the day and observations that occurred during the night to ensure that similar train and traffic volume conditions were compared. These two subsets, together with the total data set, were then analyzed.

The mean and standard deviation of the clearance times from the day, night, and total data sets were noticeably longer during the after study, indicating greater temporal separation between vehicles and trains. Additionally, the Mann-Whitney U test for two independent, continuously distributed populations indicated that these differences were statistically significant at the 99 percent confidence level (*12*). This means that installation of the four-quadrant gate system significantly increased the average time between the last vehicle to cross and the train's arrival at the crossing (from 24.5 seconds to 48.9 seconds). In addition to being statistically significant, this change in driver performance was large enough to be considered meaningful from a practical point of view. The Mann-Whitney U test failed to indicate a statistically significant difference at the 95 percent confidence level between the day and night data sets from either of the two studies (*12*). This means that there was no evidence that suggested that clearance times were different between day and night operation for either the two-quadrant gate system or four-quadrant gate system.

It was hypothesized that even though warning times have a major influence on driver performance, a small percentage of drivers would exhibit undesirable (dangerous or illegal) behavior no matter how short the warning times were. This type of behavior is similar to that of those drivers who exceed properly set speed limits. In other words, there will always be a few drivers who will take risks at railroad-highway grade crossings just as there will always be a few drivers who take risks at regular intersections as well as on the open highway. The problem then becomes one of defining risky behavior. To solve this problem, four categories of driver performance and associated clearance times were defined as follows:

- Risky—less than 10 seconds;
- Aggressive—from 10 to 20 seconds;
- Normal—from 20 to 30 seconds; and
- Cautious—greater than 30 seconds.

Risky behavior represents a level of driver performance in which there is little, if any, room for error. A judgmental mistake by the driver or a mechanical failure by the vehicle will probably result in an accident. Aggressive behavior represents a level of driver performance in which there is some, but not much, room for error. A small misestimation of the train's arrival time at the crossing will probably still allow time for most drivers to clear safely; however, vehicles that stall or have poor acceleration characteristics may be involved in an accident. The MUTCD appears to address this point by currently requiring a minimum warning time of 20 seconds (*11*). Normal behavior represents a level of driver performance in which most reasonable and prudent drivers fall. Most minor judgmental mistakes and poorly accelerating vehicles will not result in an accident. Cautious behavior represents a level of driver performance in which drivers probably rely totally on the warning device and not on their own judgment of the train's arrival at the crossing.

According to the preceding definitions, 40.0 percent of the clearance times in the before study (see Table 3) were classified as either risky or aggressive, whereas in the after study, no clearance times were classified in these categories. In fact, all of the clearance times in the after study were classified as cautious; however, this finding is not a result of a different train or driver population. Instead, as stated previously, it is a result of the four-quadrant gate system prohibiting motorists from driving around the gate arms by completely blocking the road. Thus, all drivers, rather than just a few, were forced to rely on the warning device. In other words, the potential for drivers to make a judgment about whether it was safe to cross was removed from their possible set of options. Reliance on active warning devices is especially important at crossings with limited sight distance, high-speed trains, and multiple tracks because it is at these locations that drivers often make mistakes in judgment. However, to avoid unnecessarily delaying drivers at these crossings and to reduce risky or aggressive behavior, it is imperative that the warning devices operate reliably and at as high a level of service as possible.

Approach Measures

Speed Profiles

The average speed at which drivers approached the Cherry Street crossing whenever the warning devices were activated could or could not be different after the installation of the four-quadrant gate system. Hypothetically, the greater conspicuity and more imposing presence of the four-quadrant gate system should cause drivers to see them earlier and slow down sooner. Even if this behavior change occurred it may not be large enough to be statistically significant. If it is statistically significant, it still might not be large enough to be practically significant (i.e., a difference in speeds of 1 or 2 mph might be statistically significant because of a large sample size; however, from a practical standpoint, such a difference would be meaningless) (*14*).

To compare characteristics of similar vehicles, approach speed profiles for the first vehicle to stop at the crossing in both the before and after studies were plotted as shown in Figure 3. Each data point represents average speeds over 50-ft sections of roadway in advance of the stop bar at the crossing and are plotted at the midpoint of the section. Data in the range of 50 to 200 ft from the stop bar were obtained from camera 1, in the range of 250 ft to 450 ft from the stop bar from camera 2, and in the range from 500 to 700 ft in advance of the stop bar from camera 3. Unfortunately, there was such a small amount of available data from camera 3 that a significant number of average speeds could not be calculated at the far distances. Therefore, only data from the first two cameras are shown in Figure 3.

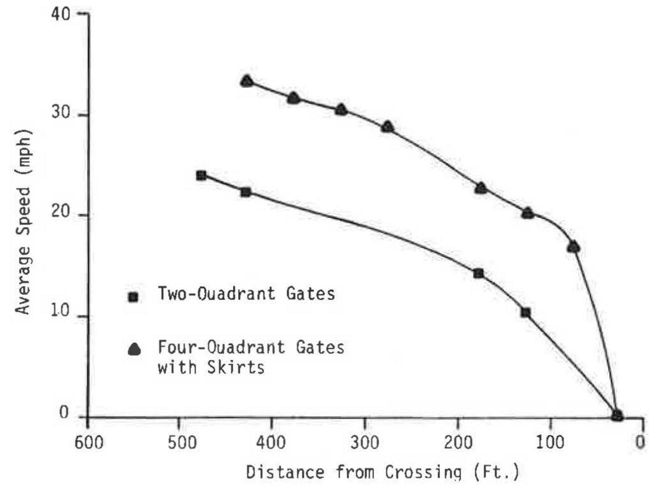

FIGURE 3 Average speed profiles for vehicles in advance of the Cherry Street crossing.

Several observations can be made concerning the average approach speed profiles in the before-and-after data sets. First, the average speeds in the after study (four-quadrant gate system) were about 10 mph faster than they were in the before study (two-quadrant gate system). This figure contradicted the initial premise that drivers slow down or at least maintain their speed in response to the four-quadrant gate system. As a result, an investigation was begun into why drivers speeded up. In the after study, the first vehicle to stop at the crossing did so because the four-quadrant gate system completely blocked the roadway. However, in the before study, visual observation of the videotapes indicated that the first vehicle to stop often followed a queue of slow-moving vehicles that were driving around the gate arms; thus, the first vehicle's speed was limited by the vehicles in front of it. In other words, approach speeds of the first vehicle to stop in the after study would be characterized as free-flow and approach speeds of the first vehicle to stop in the before study would be characterized as constrained.

Because of the unanticipated difference in stimuli and conditions, it was not surprising that the average approach speeds for the first vehicle to stop in the after study were faster than they were in the before study. Even with these unexpected results, several conclusions can be drawn from the approach speed profiles shown in Figure 4. First, in both studies, the first vehicle to stop began slowing about 450 ft from the stop bar. Second, stopping vehicles did so in a safe, gradual, and consistent manner. Finally, although installation of the four-quadrant gate system failed to cause the first stopping vehicle to begin slowing down sooner, the resultant speed profiles appeared to pose no safety problem for approaching motorists.

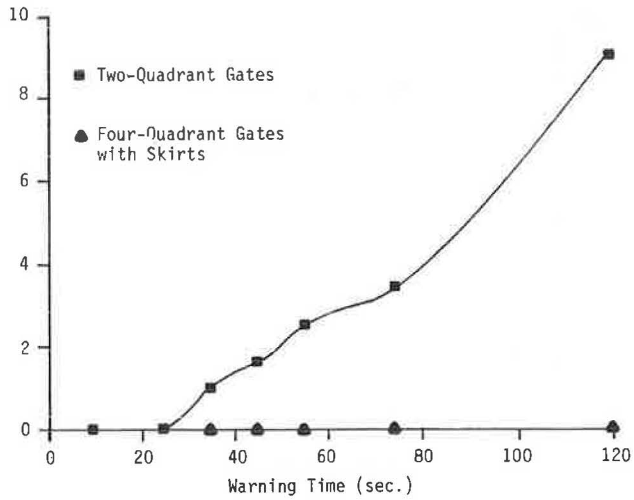

FIGURE 4 Average number of violations as a function of the warning times at the Cherry Street crossing.

PBRT and Deceleration

PBRT was defined as the difference in time between activation of the flashing light signals and the illumination of the vehicle's brake lights. It was expected that the greater conspicuity and more imposing presence of the four-quadrant gate system would cause motorists to brake sooner and, as a result, slow down more gradually. It was also expected that if these differences did exist, they would be small and very difficult to measure. To compound this problem, braking for a flashing light signal is an unexpected event and does not represent a pressure situation unless a train is also visible. Thus, driver response was expected to be highly variable.

Average PBRTs in response to the activation of the flashing light signals at the Cherry Street crossing were 18.4 seconds in the before study and 15.4 seconds in the after study. In both cases, the standard deviations were larger than the mean. These differences were small and, as indicated by the results of the Mann-Whitney U test, were not statistically significant at the 95 percent confidence level (*12*). These long reaction times confirm the premise that braking in response to a flashing light signal did not represent a pressure situation (short reaction times) and, because of this, was highly variable (large standard deviations). An additional complication with measuring PBRTs was the difficulty in determining whether the vehicle of interest was braking in response to the activation of the warning device, a slower moving vehicle ahead of it, or simply approaching a recognized railroad-highway grade crossing.

In terms of deceleration, the *Traffic Engineering Handbook* defines several deceleration categories as follows (*15*):

1. Emergency—> 20 ft/sec^2;
2. Very uncomfortable—14 to 20 ft/sec^2;
3. Uncomfortable—11 to 14 ft/sec^2;
4. Undesirable—8 to 11 ft/sec^2; and
5. Practical—< 8 ft/sec^2.

Previous studies have concluded that nearly all drivers approaching an activated flashing light signal decelerate to a stop at a practical level (*16,17*). The drivers approaching the Cherry Street crossing were no different. In the before study, only 5 percent of the vehicles exceeded a practical deceleration level while they were stopping, and in the after study, 12 percent of the vehicles did so. In both cases, none of the vehicles exceeded an undesirable deceleration. These differences are small and any differences that exist are probably the result of the differences in stimuli for the first vehicle that stopped in each of the two studies; in the after study, they may have stopped in response to the activation of the warning devices, whereas in the before study, they may have been traveling more slowly and stopping more gradually because of more slowly moving vehicles in front of them. However, in neither study did the maximum observed decelerations indicate a potential safety problem.

Safety Measures

Violations

At a crossing with gates, violations normally occur whenever motorists drive around the gate arms in the down position. As stated previously, many motorists drove around the lowered two-quadrant gate system at the Cherry Street crossing even though it was illegal to do so in Tennessee. Installation of the four-quadrant gate system was expected to eliminate this apparent disregard for the warning devices by completely blocking the roadway and making it physically impossible to drive around lowered gate arms.

Table 4 shows the number of violations observed at the Cherry Street crossing. As can be seen from Table 4, for those observations in which a motor vehicle was present before the train's arrival at the crossing, the average number of motorists per train arrival who drove around the gate arms went from 2.6 in the before study (two-quadrant gate system) to 0.0 in the after study (four-quadrant gate system). What was not expected was the high number of motorists who drove around the two-quadrant gate system—at least one in 83.9 percent of the train arrivals in which vehicles were present before the train's arrival, at least two in 62.4 percent of the train arrivals, and as many as 14 in a single train arrival. Clearly, driver performance in response to the two-quadrant gate system at Cherry Street was not good. Although it is fairly obvious that these differences were significant, a Pearson's chi-square statistic calculated from a 2-by-5 contingency table (two studies by five violation rate categories) indicated that these differences were statistically significant at the 99 percent confidence level (*12*).

One of the expected findings from the before study was that the average number of violations per train arrival would increase with an increase in warning time. These data are shown in Table 5 and illustrated in Figure 4. Notice that when the warning times were less than 40 seconds (level of service B or better), one or fewer motorists drove around the gate arms; however, when the warning times were between 40 and 60 seconds (levels of service C and D), two to three motorists drove around the gate arms, and when the warning times were longer than 60 seconds (level of service F), three or more motorists drove around the gate arms. Thus, a 40- to 50-second warning time might be considered as the threshold at which two or more motorists will drive around a gate arm,

TABLE 4 VIOLATIONS AT THE CHERRY STREET CROSSING[1]

	Two-Quadrant Gates			Four-Quadrant Gates with Skirts		
Summary Statistics	Day	Night	Total	Day	Night	Total
Sample Size[2]	71	22	93	28	25	53
Mean (vehicles)	2.76	2.09	2.60	0.00	0.00	0.00
Standard Deviation	2.40	2.29	2.38	0.00	0.00	0.00
Percent >0 Violations	87.3	72.7	83.9	0.0	0.0	0.0
Percent >1 Violation	67.6	45.5	62.4	0.0	0.0	0.0
Range (vehicles)	0-14	0-9	0-14	0-0	0-0	0-0

	Two-Quadrant Gates			Four-Quadrant Gates with Skirts		
Violations (vehicles)	Number of Observations	Percent of Observations	Cumulative Percentage	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
0	15	16.1	16.1	53	100.0	100.0
1	20	21.5	37.6	0	0.0	100.0
2	18	19.4	57.0	0	0.0	100.0
3	17	18.3	75.3	0	0.0	100.0
>3	23	24.7	100.0	0	0.0	100.0
Total	93			53		

[1]Vehicles driving around a lowered gate arm at the crossing.

[2]Includes only those observations in which vehicles were present before the train's arrival.

and a 60-second warning time as the threshold at which three or more motorists will drive around the gate arm. These observations support the premise that the longer the warning time, the larger the number of illegal and dangerous maneuvers that will take place.

The four-quadrant gate system simply eliminated all violations, as can be seen in Table 5. Obviously, this is a significant safety benefit.

Vehicles Crossing

The average number of vehicles crossing between activation of the flashing light signals and the train's arrival at the crossing is shown in Table 6. It should be noted that these numbers include not only the motorists who drove around the gate arms when they were in the down position (i.e., a violation), but also those motorists who drove through the crossing while the gate arms were descending. Installation of the four-quadrant gate system was expected to reduce the frequency of such behavior by completely blocking the roadway and making it physically impossible to drive around the lowered gate arms. Additionally, the more formidable appearance of the four-quadrant gate system with skirts may have discouraged some motorists from crossing while the gate arms were descending.

For the aforementioned reasons the average number of vehicles crossing per train arrival and the percentage of train arrivals with at least one vehicle crossing went from 4.01 and 96.8 in the before study (two-quadrant gate system) to 1.13 and 54.7 in the after study (four-quadrant gate system). As with the observed violations, it is fairly obvious that these differences were significant. This observation was verified by the results of the Mann-Whitney U test and a Pearson's chi-square statistic from a 2-by-6 contingency table (two studies by six crossing categories rate) that indicated that these differences were significant at the 99 percent confidence level (*12*). These findings support the premise that the four-quadrant gate system improved safety at the Cherry Street crossing by reducing the number of vehicles crossing in front of an oncoming train.

TABLE 5 EFFECTS OF WARNING TIMES ON VIOLATION RATES AT THE CHERRY STREET CROSSING

Study	Warning Time (Sec.)[1]	Observed Train Arrivals[2]	Average Violations (per Arrival)
Two-Quadrant Gates	<20	0	--
	20-30	2	0.00
	30-40	10	1.00
	40-50	15	1.64
	50-60	37	2.54
	60-90	27	3.44
	>90	2	9.00
	Total	93	2.60
Four-Quadrant Gates with Skirts	<20	1	0.00
	20-30	--	--
	30-40	1	0.00
	40-50	18	0.00
	50-60	21	0.00
	60-90	11	0.00
	>90	2	0.00
	Total	53	0.00

[1]Time between activation of flashing lights and train's arrivals at the crossing.

[2]Includes only those observations in which vehicles were present.

Crossings Less Than 20 Seconds (CL20)

Vehicles crossing within 20 seconds of a train's arrival at a crossing has previously been defined as an indication of aggressive behavior, i.e., there is some, but not much, room for driver or vehicular error. Because motorists had to drive around lowered gate arms to cross within 20 seconds, this behavior was illegal. Additionally, this measure represents those drivers who choose to cross within the 20-second minimum warning time presently required by the MUTCD (*11*). Installation of the four-quadrant gate system was expected to eliminate this type of behavior by completely blocking the roadway at least 20 seconds before the train's arrival at the crossing.

As shown in Table 7, the average number of vehicles crossing within 20 seconds of the train's arrival at the crossing went from 0.60 in the before study to 0.0 in the after study. Additionally over 40 percent of the observations in the before study resulted in at least one CL20, and over 10 percent of the observations resulted in multiple CL20s. Results from the Mann-Whitney U test indicated that these differences were significant at the 95 percent confidence level. Thus, as expected, installation of the four-quadrant gate system significantly decreased the CL20 rate (aggressive behavior) at the Cherry Street crossing.

A frequency distribution of the observed CL20s at the Cherry Street crossing is also shown in Table 7. In the before study there were 55 observations with zero CL20s, 27 observations with one CL20, six observations with two CL20s, and five observations with three or more CL20s. In the after study, there were no CL20s in any of the 53 observations. A Pearson's chi-square statistic calculated from a 2-by-4 contingency table substantiates the fact that these differences were significant at the 99 percent confidence level (*12*).

Crossings Less Than 10 Seconds (CL10)

Although it is illegal in Tennessee to drive around gate arms when they are in the down position (a violation), it also becomes extremely risky to do so whenever a train is in close proximity to the crossing. There was a portion of the data set that was also in potential conflict (at risk) with a train's arrival at the crossing. Clearance times that leave little room for either driver or vehicular error have previously been defined as crossing within 10 seconds of an oncoming train's arrival (CL10).

TABLE 6 VEHICLE CROSSINGS AT THE CHERRY STREET CROSSING[1]

	Two-Quadrant Gates			Four-Quadrant Gates with Skirts		
Summary Statistics	Day	Night	Total	Day	Night	Total
Sample Size[2]	71	22	93	28	25	53
Mean (vehicles)	4.32	2.95	4.01	1.50	0.72	1.13
Standard Deviation	2.93	2.87	2.96	1.50	0.98	1.33
Percent >0 Violations	98.6	90.9	96.8	64.3	44.0	54.7
Percent >1 Violation	88.7	63.6	82.8	42.9	20.0	32.1
Range (vehicles)	0-19	0-11	0-19	0-5	0-3	0-5

	Two-Quadrant Gates			Four-Quadrant Gates with Skirts		
Crossings (vehicles)	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
0	3	3.2	3.2	24	45.3	45.3
1	13	14.0	17.2	12	22.6	67.9
2	20	21.5	38.7	7	13.2	81.1
3	10	10.8	49.5	7	13.2	94.3
>4	47	50.5	100.0	3	5.7	100.0
Total	93			53		

[1]Vehicles driving around a lowered gate arm at the crossing.

[2]Includes only those observations in which vehicles were present before the train's arrival.

It was anticipated that installation of the four-quadrant gate system would eliminate this type of behavior by completely blocking the roadway at least 20 seconds before the train's arrival at the crossing.

As shown in Table 8, five CL10s (risky crossings) were observed at the Cherry Street crossing in the before study—four during the day and one during the night. Thus, five motorists drove around the gate arms and crossed the tracks within 10 seconds of the train's arrival. As expected, no similar behavior was observed with the four-quadrant gate system in the after study. A Pearson's chi-square statistic calculated from a 2-by-2 contingency table indicated that these differences were significant at the 95 percent confidence level for the day, night, and total data sets (*12*). Thus, it is obvious that installation of the four-quadrant gate system removed the possibility of risk-taking from the driver's set of options.

MAJOR FINDINGS AND CONCLUSIONS

Based on the field test results, the four-quadrant gate system outperformed the standard two-quadrant gate system on several key measures and proved to be operationally acceptable under a variety of conditions. Specific findings and conclusions for the four-quadrant gate system are summarized below:

1. Based on evaluation of the MOEs, the four-quadrant gate system substantially increased the safety of the crossing compared with the standard two-quadrant gate system.
2. With the two-quadrant gate system, one or more motor vehicles drove around the closed gates during 84 out of every 100 train arrivals. The four-quadrant gate system reduced the number of gate violations (number of vehicles crossing) from an average of 260 per 100 train arrivals to 0.
3. Compared with the standard two-quadrant gate system, the four-quadrant gate system reduced the CL20s (vehicles crossing less than 20 seconds before arrival of train) from 60 per 100 train arrivals to 0.
4. Compared with the standard two-quadrant gate system, the four-quadrant gate system reduced the CL10s (vehicles crossing less than 10 seconds before arrival of a train) from 5 per 100 trains to 0.
5. The four-quadrant gate system did not significantly affect PBRT or maximum deceleration levels at the test crossing.

TABLE 7 CL20S AT THE CHERRY STREET CROSSING[1]

Summary Statistics	Two-Quadrant Gates Day	Night	Total	Four-Quadrant Gates with Skirts Day	Night	Total
Sample Size[2]	71	22	93	28	25	53
Mean (vehicles)	0.65	0.45	0.60	0.0	0.0	0.0
Standard Deviation	0.97	0.67	0.91	0.0	0.0	0.0
Percent >0 Violations	42.3	36.4	40.9	0.0	0.0	0.0
Percent >1 Violation	12.7	9.1	11.8	0.0	0.0	0.0
Range (vehicles)	0-4	0-2	0-4	0-0	0-0	0-0

CL20s (vehicles)	Two-Quadrant Gates Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Four-Quadrant Gates with Skirts Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
0	55	59.1	59.1	53	100.0	100.0
1	27	29.0	88.1	0	0.0	100.0
2	6	6.5	94.6	0	0.0	100.0
3	3	3.2	97.8	0	0.0	100.0
>3	2	2.2	100.0	0	0.0	100.0
Total	93			53		

[1]Vehicles driving around a lowered gate arm at the crossing.

[2]Includes only those observations in which vehicles were present before the train's arrival.

TABLE 8 CL10S AT THE CHERRY STREET CROSSING[1]

Summary Statistics	Two-Quadrant Gates Day	Night	Total	Four-Quadrant Gates with Skirts Day	Night	Total
Sample Size[2]	71	22	93	28	25	53
Mean (vehicles)	0.06	0.05	0.05	0.0	0.0	0.0
Standard Deviation	0.23	0.21	0.23	0.0	0.0	0.0
Percent with Conflicts	5.6	4.6	5.4	0.0	0.0	0.0
Range (vehicles)	0-1	0-1	0-1	0-0	0-0	0-0
0 Conflicts/Arrival	67	21	89	28	25	53
1 Conflict/Arrival	4	1	5	0	0	0

[1]Vehicle's crossing within 10 seconds of the train's arrival.

[2]Includes only those observations in which vehicles were present prior to the train's arrival.

6. During the entire time that the four-quadrant gate system was in place at the test crossing, no motorists were trapped on the tracks. The four-quadrant gate system did not appear to increase the risk of a vehicle being trapped on the tracks, provided the lowering of the far-side gate arms was delayed by a few seconds to allow vehicle clearance.

7. The four-quadrant gate system did not interfere in any way with emergency vehicle operations at the test crossing during the field evaluation.

8. The four-quadrant gate system did not create unreasonable delays for motorists during the field evaluation.

9. No significant amount of traffic was diverted to other routes to avoid the four-quadrant gate system.

10. No public complaints were received concerning the use or operation of the four-quadrant gate system.

11. The wooden gate arms with skirts fabricated for the research performed adequately even under adverse weather conditions (e.g., high winds, heavy snow and ice).

12. Because of their simple design, the gate arms with skirts were too easily damaged when "brushed" by a vehicle. For long-term use, modifications should be made in the skirt assembly.

13. A standard two-quadrant gate system can be retrofitted easily to a four-quadrant gate system.

14. Worldwide experience with a four-quadrant gate system has been good and the need to provide for their use in the MUTCD is evident.

15. At a minimum, a four-quadrant gate system can be considered for the following types of crossings:

- Crossings on four-lane undivided roads;
- Multitrack crossings in which the distance between tracks is greater than the length of a motor vehicle;
- Crossings without train predictors in which train warning times are long and variable;
- Crossings that are frequented by trucks carrying hazardous materials, school buses, or high-speed passenger trains; and
- Crossings with consistent gate arm violations or continuing accident occurrences.

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Evaluation of Two Active Traffic Control Devices for Use at Railroad-Highway Grade Crossings

Daniel B. Fambro, K. W. Heathington, and Stephen H. Richards

Two active traffic control devices with the potential for improving safety at railroad-highway grade crossings were identified by a detailed laboratory evaluation as candidates for field testing under normal traffic conditions at actual crossings. Two crossings with active warning devices already in place were identified as potential study sites, and train and driver behavior data were collected both before and after the experimental traffic control devices were installed. The two devices evaluated for use at railroad-highway grade crossings were four-quadrant flashing light signals with overhead strobes and standard highway traffic signals. Based on the results of the field evaluation, there were no measurable differences in driver behavior between four-quadrant flashing light signals with overhead strobes and the standard two-quadrant flashing light signals. The warning system itself was operationally feasible and may have some limited application. The highway traffic signal proved to be both feasible and effective as a grade crossing traffic control device. Driver response to the highway traffic signal was excellent, with the traffic signal outperforming standard flashing light signals on several key safety and driver behavioral measures of effectiveness. Additional testing of this system is recommended.

Safety improvement at railroad-highway grade crossings over the past 10 years is well documented (*1*). Much of this improvement can be attributed to the availability of federal funds for grade crossing improvement projects (*2*), the majority of which involved upgrading passive crossings to active ones. As a result of the increase in these projects, over one in four of the 192,454 public grade crossings had active warning devices in 1986. Of concern, however, is the fact that over 50 percent of all car-train accidents occur at crossings with active warning devices (*3*), even though these crossings account for only 28 percent of all crossings. Although this high number of accidents may be a result of higher vehicle and train volumes and more complex railroad-highway geometrics at active crossings, it is likely that some of the accidents are caused by motorists either not seeing or not understanding the active warning devices presently used at railroad-highway grade crossings. For these reasons, it is clear that driver response to active warning devices at railroad-highway grade crossings could be improved.

Research aimed at improving driver response to active warning devices at railroad-highway grade crossings has been going on for some 50 years. Although there have been significant improvements in both electronic equipment and system components, the warning devices that are seen and responded to by the motorists have not changed significantly during this period of time. Even though many innovative warning devices have been developed for use both at and in advance of crossings, implementation of new devices during the past 50 years has been minimal.

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Recognizing the need to fully address more extensively the issues and problems concerning active warning devices at railroad-highway grade crossings, the FHWA sponsored a research project at the University of Tennessee to identify and evaluate innovative active warning devices with potential for improving safety at railroad-highway grade crossings. As part of this research, three innovative traffic control devices were identified as having potential for improving safety and were chosen for subsequent field evaluation under normal traffic conditions at existing crossings (*4*). The objective of this paper is to compare driver performance measures in response to the existing traffic control devices at two of the crossings with that of two innovative active traffic control devices at the same crossings.

PREVIOUS RESEARCH

There are two types of warning devices for use at railroad-highway grade crossings: passive devices and active devices. Passive devices provide static warning of a grade crossing's location and are required at virtually all at-grade crossings. Active devices supplement passive ones at locations where the accident potential is high to warn drivers of the approach or presence of a train. The active warning devices currently in use were developed over 50 years ago. Guidelines for their use and some practical interpretations are offered in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*5*) and the *Traffic Control Devices Handbook* (*6*); however, the general public does not fully understand the responsibilities various warning devices place on approaching drivers (*7,8*).

Driver performance measures are a means of assessing the adequacy of a traffic control system in meeting a driver's needs. The better those needs are met, the better the driver performs. The challenge lies in defining what constitutes good driver behavior. Surprisingly, few studies have attempted to quantify driver behavior at railroad-highway grade crossings. Those studies that have examined driver behavior have focused on such measures as looking behavior, speed profiles and changes, deceleration levels, conflicts, and violations. As a result of these studies, several interesting and somewhat unexpected conclusions were reached.

Looking behavior is a poor measure of driver performance because although drivers look, one does not know why or if they even see specific things in their field of view (*9*). In addition, looking behavior appears to be more related to past experience than the need to look, i.e., at various crossings, familiar drivers tend to look more when train volumes are high, and at the same crossing they tend to look less than unfamiliar drivers (*10*). Speed profiles of familiar drivers on the approach to a grade crossing are a function of the crossing surface, making it virtually impossible to compare various crossings; however, they are useful when comparing various warning systems at the same crossing.

When studying approach speed profiles, drivers should be grouped into categories of similar expected behavior based on the stimulus for stopping at the crossing (*11*). Basically, the greater the stimulus, the sooner and more gradually drivers will begin to slow down (*12*). Lowered gate arms result in the smoothest speed profiles and surprisingly, activated flashing lights result in speed profiles similar to those at passive crossings (*13*). As for speed changes of individual vehicles approaching the crossing, there are no apparent patterns other than the fact that their variance increases as the vehicles get closer to the crossing (*9*).

Extreme deceleration levels and large numbers of conflicts and violations are good indicators of potential grade crossing safety problems. Unfortunately, very few drivers exceed a practical deceleration level when stopping, thus requiring large data bases (*12*). Conflicts and violations are more common and easily observed. The key to their use is a clearly defined behavior that can be measured in the field.

Driver behavior at signalized intersections is different from that at railroad-highway grade crossings since changes in right-of-way are expected at intersections and unexpected at grade crossings; however, several research findings are worth noting. The 85th percentile perception-brake reaction time in response to a yellow signal has been estimated as 1.2 seconds (*14*). This value does not change with either distance from the intersection or day-night or wet-dry conditions. The 85th percentile deceleration level is 10.5 ft/sec^2, which is also unchanged for all conditions other than approach grade (*14*). As with grade crossings, few drivers select higher-than-practical deceleration levels when stopping. Ninety-five percent of the drivers that do not stop enter the intersection within 4.5 seconds of the onset of yellow regardless of their approach speed (*14*).

STUDY PROCEDURE

Two active warning devices for use at railroad-highway grade crossings were identified by a detailed laboratory evaluation process as candidates for field testing under normal conditions at actual crossings. Two crossings in the Knoxville area, Ebenezer Road and Cedar Drive, were identified as potential study sites, and driver behavior was studied before and after the new devices were installed. Both crossings, with standard railroad flashing light signals already in place, had relatively high train and traffic volumes and a history of at least some accidents. A four-quadrant flashing light signal system (with red strobe lights over the traffic lanes) was installed at the Ebenezer Road crossing and a highway traffic signal system (with white bar strobes in each red signal lens) was installed at the Cedar Drive crossing.

Data on driver behavior approaching and at the two crossings were collected using three pole-mounted video cameras, with each of the cameras covering approximately 300 ft of roadway with overlapping fields of view. The video recorders were automatically turned on before the activation of the warning devices and ran for approximately 2.5 to 3 minutes. For each study at a particular crossing, data were collected for a minimum of 30 trains. One existing and one improved condition study was conducted at each of the study sites.

Data tapes were taken to the university's computer lab for processing. The tapes were transferred to and played back on a high-quality video reproductive machine that could stop action and produce sequential scenes separated by $1/60$ of a second. Speed profiles were determined by using successive frames and noting the distances that the vehicle had traveled between frames. Since the cameras were fixed, any point on the vehicle moved on a surface dictated by the roadway. By use of an electronic cross hair, the coordinates of this reference point were calculated for successive frames and manually recorded. This information was used to construct each individual vehicle's speed-distance profile.

Other measures of driver performance that were recorded include perception-brake reaction times and violation and vehicle crossing rates in response to device activation. Statistical comparisons of these measures were made between both devices and conditions. The general hypotheses tested were that installation of these new devices improved the conspicuity of and compliance with active warning devices at railroad-highway grade crossings, thus providing for safer operations at the crossing.

Ebenezer Road

The four-quadrant flashing light signals with overhead strobes were installed at the Ebenezer Road crossing during the week of October 14, 1985. Before this time, active warning devices at the crossing were standard two-quadrant flashing light signals. Both train movement and driver behavior data were collected for approximately 2 months before (July and August 1985) and 2 months after (May and August 1986) the new devices were installed. During these two periods, a total of 226 train movements were observed. There were 157 trains observed in the before study (two-quadrant flashing light signals), and 79 trains observed in the after study (four-quadrant flashing light signals with overhead strobes). For each observation in the two studies, the environmental and lighting conditions, train's direction of travel and warning time, and approaching vehicle's clearance time, speed profile and brake reaction time were recorded and subsequently analyzed.

The approach roadway's horizontal and vertical alignments limit visibility of the Ebenezer Road crossing from both directions. Thus, the visibility of the standard two-quadrant flashing light signals at the crossing was also limited. The primary change in driver performance that was expected as a result of the installation of the four-quadrant flashing light signals with overhead strobes was an earlier reaction to the active warning devices. As a result of this change in behavior, the approach speeds were expected to be slower, the brake reaction times were expected to be quicker, and the deceleration levels were expected to be more gradual; however, differences in these driver performance measures were not expected to be easy to quantify and the related safety benefits are not straightforward.

Driver behavior at the crossing itself (i.e., clearance times, violation rates, and vehicle crossing rates) was not expected to change, since the new device neither changed the train detection system nor physically blocked the roadway. The only legal requirements placed on motorists approaching a flashing light signal are that they bring their vehicle to a stop in advance of the crossing and then proceed when it is safe to do so. Thus, violations at a crossing with flashing light signals were defined as the failure of drivers to reasonably stop in response to the warning device. Because of the difficulty in determining whether a vehicle came to a complete stop, violations were not counted for either of the two flashing light signal systems.

Cedar Drive

Highway traffic signals were installed at the Cedar Drive crossing during April 1986. Before this time, the active warning devices at the crossing were standard two-quadrant flashing light signals; however, because it was felt that long warning times at this crossing might lessen the traffic signal's credibility, predictors were installed during November 1985 to provide shorter and more consistent warning times. Both train movement and driver behavior data were collected for approximately 2 months before the highway traffic signals were installed (February and March 1986) and 2 months after the highway traffic signals were installed (July and August 1986).

During these two periods, a total of 142 train movements were observed. A total of 50 train movements were observed in the before study (flashing light signals with predictors) and 92 train movements were observed in the after study (highway traffic signals with predictors). For each observation, the environmental and lighting conditions, train's direction of travel and warning time, and approaching vehicle's clearance time, speed profile, and perception-brake reaction time were recorded and subsequently analyzed.

The Cedar Drive crossing had severe safety problems as evidenced by its high hazard ranking (31st most dangerous crossing in the state) and the three car-train accidents that occurred at this site during the past 5 years. It was hypothesized that these safety problems were a result of a combination of the relatively high train and traffic volumes, limited sight distance at the crossing, and long warning times, resulting in numerous motorists crossing in front of approaching trains. Because highway traffic signals have a relatively high level of driver credibility and respect, their installation at the Cedar Drive crossing should discourage most motorists from crossing in front of approaching trains.

Since the highway traffic signals legally rather than physically prohibit crossing, the average clearance time between the last vehicle to cross and the train's arrival at the crossing may or may not have increased. The average number of vehicles crossing per train arrival, however, was expected to decrease. These behavioral modifications have implied safety benefits since they provide greater spatiotemporal separation between trains and motor vehicles. The anticipated secondary change in driver performance was better response to the new devices (i.e., quicker perception-brake reaction times and lower deceleration levels) as a result of the greater conspicuity of the white bar strobes and credibility of the traffic signal. As noted previously, differences in these performance measures were not expected to be as easy to quantify, and the related safety benefits are not expected to be as straightforward.

RESULTANT MEASURES OF EFFECTIVENESS

Warning Time

Warning time was defined as the difference in time between activation of the flashing light signals and the train's arrival at the crossing. It is the same as the maximum time a motorist would have to wait between activation of the flashing light signals and a train's arrival at the crossing. Since there were no changes to the train detection system at either crossing when the new warning devices were installed, there should have been no difference in the average warning times observed in the before-and-after studies. To verify this premise, the total data set from each study was first subdivided into observations that occurred during the day and observations that occurred during the night to insure that similar train and traffic volume conditions were compared. These two subsets, together with the total data set, were then analyzed.

As shown in Table 1, the mean and standard deviation of the warning times from the two data sets at the Ebenezer Road Crossing were slightly shorter in the after study (flashing light signals with strobes); however, the Mann-Whitney U test for two independent, continuously distributed populations (*15*) indicated that these differences were not significantly different at the 95 percent confidence level for either the day, night, or total data sets. This means that, as expected, installation of the four-quadrant flashing light signals with overhead strobes had no effect on the warning times at the crossing. The Mann-Whitney test also indicated that there was not a statistically significant difference at the 95 percent confidence level between the day and night data sets from either of the two studies. Thus, warning times were not different during day and night operations for either the two-quadrant flashing light signals or the four-quadrant flashing light signals with overhead strobes.

The mean warning times from the two data sets at the Cedar Drive crossing (shown in Table 1) were also slightly shorter in the after study (highway traffic signals with predictors). In this case, however, the Mann-Whitney test for two or more independent, continuously distributed populations (*15*) indicated that these differences were statistically significant at the 99 percent confidence level. The Mann-Whitney test also indicated that there was a statistically significant difference at the 95 percent level between the night data sets from the two studies. These results were unexpected, since the train detection system did not change between the two studies; however, the results indicate that warning times were significantly shorter in the after study.

Clearance Time

Clearance time was defined as the difference in time between the last vehicle to cross and the train's arrival at the crossing.

TABLE 1 WARNING TIMES AT EBENEZER ROAD AND CEDAR DRIVE CROSSINGS[1]

EBENEZER ROAD CROSSING	Flashing Light Signals			Flashing Light Signals with strobes		
	Day	Night	Total	Day	Night	Total
Sample Size	106	51	157	60	19	79
Mean (seconds)	42.2	38.1	40.8	39.7	41.7	40.2
Standard Deviation	15.6	11.1	14.4	9.5	18.6	12.2
Range (seconds)	24-153	26-106	24-153	14-62	32-116	14-116
CEDAR DRIVE CROSSING	**Flashing Light Signals with Predictors**			**Highway Traffic Signals with Predictors**		
	Day	Night	Total	Day	Night	Total
Sample Size	22	28	50	67	25	92
Mean (seconds)	40.5	42.7	41.7	38.1	31.5	36.3
Standard Deviation	15.5	19.9	18.0	21.7	8.4	19.2
Range (seconds)	27-89	28-121	27-121	23-161	8-57	8-161

[1] Time between either activation of flashing lights or onset of yellow and the train's arrival at the crossing.

Since the four-quadrant flashing signals with overhead strobes changed nothing at the crossing itself, their installation was expected to have no effect on the observed clearance times. Thus, there was no expected increase in the temporal separation between cars and trains as a result of the installation of the new devices. Installation of the traffic signal, however, may give enough credibility to the warning devices to increase average clearance times at the Cedar Drive crossing. If, in fact, this does occur, the additional temporal separation between the cars and trains would be a definite safety benefit. This benefit is expected to be the result of the installation of both the predictors and the highway traffic signal at the Cedar Drive crossing.

Clearance times were recorded only for those train arrivals during which a vehicle arrived at the crossing between the activation of the flashing light signals and the train's arrival at the crossing, i.e., there was an opportunity for a vehicle to cross in front of the train while the signals were activated. Thus, the number of clearance times will always be less than or equal to the number of train arrivals. As with the warning time data set, the total data from each study was subdivided into observations that occurred during the day and observations that occurred during the night so as to insure that similar train traffic volume conditions were compared. The two subsets along with the total data set were then analyzed.

As shown in Table 2, the mean and standard deviation of the clearance times from all data sets at the Ebenezer Road Crossing were slightly shorter in the after study. The Mann-Whitney test (*15*), however, indicated that these differences were not statistically significant at the 95 percent confidence level for either the day, night, or total data sets. These values mean that installation of the four-quadrant flashing light signals with overhead strobes had no effect on the average time between the last vehicle to cross and the train's arrival at the crossing. This finding is shown clearly in the illustration of the frequency distribution of the clearance times from the two data sets in Figure 1. The Mann-Whitney test also failed to indicate a statistically significant difference at the 95 percent confidence level between the day and night data sets from either of the studies. These values mean that the clearance times were no different between day and night operation for either the two-quadrant flashing light signals or the four-quadrant flashing light signals with overhead strobes.

At the Cedar Drive crossing, the mean clearance times from the total data sets were approximately the same for both studies (see Table 2). The Mann-Whitney test for two or more independent, continuously distributed populations (*15*) confirmed that these differences were not statistically significant at the 95 percent confidence level. There was also no significant difference between the day and night data sets from either of the two studies, meaning that installation of the highway traffic signals in combination with the predictors had no effect on the clearance times observed at the Cedar Drive crossing.

Speed Profiles

The average speed at which drivers approach the two crossings whenever the warning devices were activated may or may not have been different after the installation of the new warning devices. Hypothetically, the greater conspicuity of the new

TABLE 2 CLEARANCE TIMES AT EBENEZER ROAD AND CEDAR DRIVE CROSSINGS[1]

EBENEZER ROAD CROSSING	Flashing Light Signals			Flashing Light Signals with Strobes		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	92	17	109	53	10	63
Mean (seconds)	19.1	27.9	20.5	15.6	24.8	17.1
Standard Deviation	9.9	20.5	12.4	6.8	10.3	8.1
Range (seconds)	7-64	8-99	7-99	5-36	4-38	4-38
CEDAR DRIVE CROSSING	**Flashing Light Signals with Predictors**			**Highway Traffic Signals with Predictors**		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	19	20	39	20	9	29
Mean (seconds)	16.2	26.3	21.4	20.7	21.4	20.9
Standard Deviation	5.8	18.9	14.9	8.4	10.5	8.9
Range (seconds)	7-28	6-96	6-96	5-34	10-45	5-45

[1] Time between the last vehicle to cross and the train's arrival at the crossing.

[2] Includes only those observations in which vehicles were present before the train's arrival.

warning devices, especially that of the overhead strobes, should cause drivers to see the warning devices earlier and slow down sooner. Even if this behavioral change occurs, however, it may not be large enough to be statistically significant, and even if it were, it still might not be large enough to be meaningful from a practical point of view (*16*). Also, the safety benefits of such a speed change are not easily quantified.

Although not illustrated in the paper, several observations can be made concerning the average approach speed profiles in the before-and-after data sets at the two crossings. First, the average speeds in the after studies were lower than the average speeds in the before studies. This finding indicates that the flashing signals with strobes and the highway traffic signals with strobes may have been visible farther from the crossing than the flashing light signals. Closer examination of the data, however, revealed that the average speeds in the before study and those in the after study were in fact relatively close to one another. Second, vehicles stopping in response to either the two-quadrant flashing light signals, the four-quadrant flashing light signals with overhead strobes, or the highway traffic signal did so in a safe, gradual, and consistent manner. As a result, the speed profiles appeared to pose no safety problems.

Perception-Brake Reaction Time

Perception-brake reaction time is defined as the difference in time between activation of the flashing light signals and the illumination of a vehicle's brake lights. It was expected that the greater conspicuity of the new traffic control devices would cause motorists to brake sooner and as a result decelerate more gradually. It was also expected that if these differences did exist, they would be very small and difficult to measure. To compound this problem, braking for a flashing light signal is an unexpected event and also does not represent a pressure situation unless a train is also visible. Thus, driver response can be relatively long and highly variable.

Average brake reaction times in response to the activation of the flashing light signals at the Ebenezer Road crossing were 15.6 seconds in the before study and 14.3 seconds in the after study. These differences were not large enough to be either statistically or practically significant, indicating that installation of the flashing light signals with overhead strobes had no measurable effect on the perception-brake reaction time of approaching motorists.

Average perception-brake reaction times in response to the activation of either the flashing light signals or the onset of the traffic signal's red indication were 17.1 seconds (before study), and 19.2 seconds (after study), respectively. In both cases the standard deviation was almost as large or larger than the mean. These differences also were not large enough to be either statistically or practically significant, indicating that installation of the highway traffic signal had no measurable effect on the perception-brake reaction time of approaching motorists.

These data confirm the premise that braking in response to either a flashing light signal or a highway traffic signal at

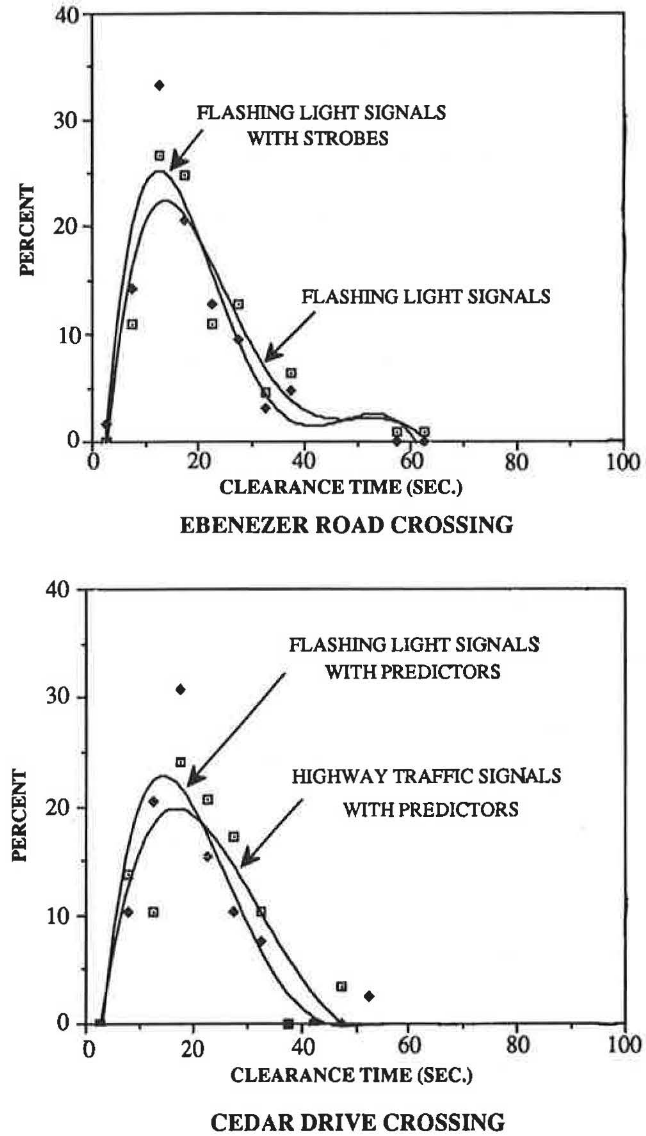


FIGURE 1 Frequency distribution of observed clearance times at Ebenezer Road and Cedar Drive crossings.

a railroad highway grade crossing did not represent a pressure situation (i.e., long reaction times) and, because of this, was highly variable. An additional complication with measuring brake reaction times was the difficulty in determining whether the vehicle of interest was braking in response to the activation of the warning device, a more slowly moving vehicle ahead of it, the horizontal alignment of the road, or simply because of the roughness of the crossing itself.

Deceleration Levels

In terms of deceleration, drivers approaching warning devices at the Ebenezer Road or Cedar Drive crossing were no different than those reported in the literature (*11,12*). None of the observed deceleration levels in either the before or the after studies exceeded a practical deceleration level, again indicating nonemergency stops. It could, however, also indicate that drivers had already slowed their vehicles because of the horizontal or vertical alignment of the road. If this were the case, continuance of this initial slowdown to a stop may have resulted in low decelerations. Whatever the reason, the maximum deceleration levels observed at the Ebenezer Road and Cedar Drive crossings did not indicate a potential safety problem for either the two-quadrant flashing light signals, the four-quadrant flashing light signals with overhead strobes, or the highway traffic signals.

Violations

At a crossing with flashing light signals, violations were defined as the failure of motorists to reasonably stop in response to the warning device. Because of the difficulty in determining whether a vehicle came to a complete stop, however, violations were not counted for the flashing light signal systems. Even if the number of violations had been counted, installation of the four-quadrant flashing light signals with overhead strobes was not expected to change the frequency of occurrence because there were no changes to either the train detection system or the crossing itself.

At a crossing with highway traffic signals, violations were defined as a motorist driving through the crossing while the signal displayed a red indication, i.e., a violation of the motor vehicle laws. Since the highway traffic signals did not physically block the roadway, their installation was not expected to eliminate violations at the Cedar Drive crossing; however, installation of the predictors in combination with the highway traffic signals may provide enough credibility in the warning devices to significantly reduce the number of violations at the crossing. Unfortunately, because of the different definitions, a direct comparison of the violation rates between the two conditions was not possible.

When the highway traffic signal was installed at the Cedar Drive crossing, the average and maximum number of motorists per train arrival who "ran the red" (illegal behavior) was 0.68 and 6, respectively. These statistics were based on the 78 observations in which vehicles were in the crossing area before the train's arrival. Of this total there were 50 observations in which no motorists behaved illegally, 15 observations in which one motorist behaved illegally, and 13 observations in which more than one motorist behaved illegally.

Vehicles Crossing

The average number of vehicles crossing between activation of the flashing light signals and the train's arrival at the Ebenezer Road crossing are shown in Table 3. As there was no statistically significant difference in the warning times observed during the two studies, there should have been no difference in the number of vehicles crossing. The results of the Mann-Whitney test (*15*) verified this premise at the 95 percent confidence level. Interestingly, almost 28 percent of the total observations resulted in five or more vehicles crossing after the flashing light signals were activated. This appears to be a clear indication that motorists will drive through a crossing while the signals are flashing as long as a train is not believed to be in close proximity.

The effects of warning times on the number of vehicles crossing while the flashing light signals were activated at the

TABLE 3 VEHICLES CROSSING AT EBENEZER ROAD AND CEDAR DRIVE CROSSINGS[1]

EBENEZER ROAD CROSSING	Flashing Light Signals			Flashing Light Signals with Strobes		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	101	22	123	58	11	69
Mean (vehicles)	3.83	1.59	3.43	3.84	2.18	3.58
Standard Deviation	3.41	1.37	3.26	2.61	1.47	2.53
Percent >0 Crossing	91.1	77.3	88.6	91.4	90.9	91.3
Percent >1 Crossing	74.3	45.5	69.1	81.0	54.6	76.8
Range (vehicles)	0-21	0-5	0-21	0-11	0-4	0-11
CEDAR DRIVE CROSSING	**Flashing Light Signals with Predictors**			**Highway Traffic Signals with Predictors**		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	21	24	45	59	19	78
Mean (vehicles)	3.86	2.92	3.35	0.80	0.53	0.73
Standard Deviation	3.34	2.50	2.92	1.47	0.61	1.32
Percent >0 Crossing	90.5	83.3	86.7	33.3	47.4	37.2
Percent >1 Crossing	71.4	62.5	66.7	20.3	5.3	16.7
Range (vehicles)	0-12	0-9	0-12	0-7	0-2	0-7

[1] Vehicles crossing after either activation of the flashing light signals or the traffic signal changing to yellow and the train's arrival at the crossing.

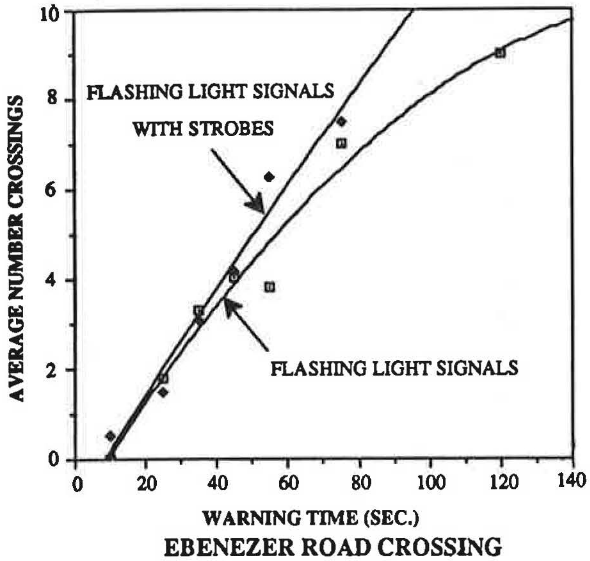
[2] Includes only those observations in which vehicles were present before the train's arrival.

Ebenezer Road crossing is shown in Figure 2. Even though the majority of the warning time observations were in the 30- to 40-second range, there is clearly an identifiable trend—the longer the warning time, the greater the number of vehicles that will cross while the signal is flashing. Note that if the warning time is less than 30 seconds, an average of one to two drivers will cross in front of the train; whereas, if the warning time is longer than 30 seconds, an average of three to four drivers will cross in front of the train. Although they were never in immediate danger, the more the drivers in these observations had to decide whether or not it was safe to cross, the greater the probability of one of them making the wrong decision.

The average number of vehicles crossing between activation of either the flashing light signals or the traffic signal's clearance interval and the train's arrival at the Cedar Drive crossing are also shown in Table 3. Because of the highway traffic signal's additional credibility, it was hypothesized that there would be a significant difference in the number of vehicles crossing. The Mann-Whitney test verified this premise at the 99 percent confidence level for the day, night, and total data sets, i.e., a significant reduction in the number of vehicles crossing was realized as a result of the installation of the highway traffic signals. The predictors in combination with the highway traffic signal reduced the average number of vehicles crossing per train arrival from 3.35 to 0.73. Thus, when predictors were installed in both systems, the additional credibility of the highway traffic signal reduced the average number of vehicles that crossed in front of an oncoming train by a factor of five (80 percent) compared with the flashing light signals.

The effects of warning times on the number of vehicles crossing while the flashing light signals were activated or the traffic signals were red at the Cedar Drive crossing is shown in the bottom of Figure 2. Even though the total observations are not distributed evenly throughout the warning time categories, there is clearly an identifiable trend, i.e., the longer the warning time, the greater the number of vehicles that will cross while the warning devices are activated. Note that if the warning time at the flashing light signals was less than 30 seconds, an average of one or two drivers will cross in front of the train, whereas if the warning time is greater than 30 seconds, an average of three to five drivers will cross in front of the train. If the warning device was a highway traffic signal, however, an average of only one driver crossed in front of the train no matter how long the warning time.

Interestingly, the average number of vehicles crossing at the Cedar Drive crossing compares favorably with the results



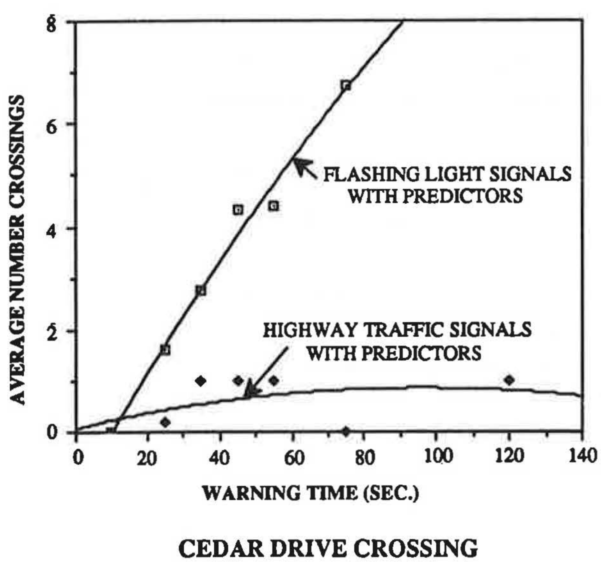


FIGURE 2 Average number of vehicles crossing as a function of warning time at Ebenezer Road and Cedar Drive crossings.

from the Ebenezer Road crossing—if the warning time is less than 30 seconds, an average of one driver will cross in front of an oncoming train, whereas if the warning time is as long as 50 seconds, an average of three to four vehicles will cross in front of the train. This result is not altogether surprising, since the active warning devices at both the Ebenezer Road crossing and the Cedar Drive crossing were exposed to similar traffic volumes, and both provided comparable average warning times. Thus, it appears that traffic volume and the average warning times may be a good indication of the average number of vehicles that will cross in front of an oncoming train.

Crossings Less Than 20 Seconds (CL20)

Vehicles crossing within 20 seconds of a train's arrival at the crossing were defined as an indication of aggressive behavior, i.e., there is some, but not much, room for driver and vehicular error. Although such behavior is not illegal, it represents those drivers that choose to cross within the 20-second minimum warning time presently required by the MUTCD (*7*). Installation of the four-quadrant flashing light signals with overhead strobes at the Ebenezer Road crossing should have no effect on this driver performance measure since nothing was changed at the crossing itself; however, installation of the highway traffic signals at the Cedar Drive crossing should have an effect because of the traffic signal's additional credibility.

As shown in Table 4, the average number of vehicles crossing within 20 seconds of a train's arrival at the Ebenezer Road crossing was not noticeably different for either the before or after study. Additionally, the Mann-Whitney test (*15*) indicated that there were no statistically significant differences at the 95 percent confidence level. Thus, as expected, installation of the four-quadrant flashing light signals with overhead strobes had no effect on the CL20 rate (i.e., aggressive behavior) at the Ebenezer Road Crossing. Surprisingly, over 55 percent of the observations in each study resulted in at least one CL20 and more than 30 percent of the observations in each study resulted in multiple CL20s.

Most of the observed warning times at the Ebenezer Road crossing were in the 30- to 50-second range. This left very few observations in the other warning time ranges and precluded any development of relationships between warning times and the CL20 rates. An additional complication in the development of relationships was the fact that the time available for CL20s to occur did not increase with an increase in warning time. It is interesting to note, however, that in the 30- to 40-second warning time range, there were approximately 1.3 CL20s per train arrival in the before study and 1.4 CL20s per train arrival in the after study.

As shown in Table 4, the average number of vehicles crossing within 20 seconds of the train's arrival at the Cedar Drive crossing was noticeably lower in the after study where the highway traffic signals were installed. The Mann-Whitney test (*15*) indicated that these reductions were statistically significant for both the day and total data sets at the 99 percent confidence level. Thus, installation of the highway traffic signals significantly reduced the number of CL20s at the crossing. There was no difference in the average CL20 rates for either of the nighttime data sets. The most effective warning device as far as preventing CL20s was the predictors in combination with the highway traffic signal—82 percent of the observations in the data set resulting in no CL20s.

The average CL20 rate was approximately 0.78 after predictors were installed, and 0.24 after both predictors and traffic signals were installed; however, there did not appear to be a relationship between warning time and CL20 rates. It should be noted that in the 30- to 40-second warning time range for the flashing light signal with predictor study, there were approximately 0.83 CL20s per train arrival, and whenever traffic signals were installed, the CL20 rate in this warning time range was approximately 0.33. This seems to indicate that highway traffic signals with predictors are more effective in reducing CL20s than are standard active warning devices currently found at railroad-highway grade crossings.

Crossings Less Than 10 Seconds (CL10)

Vehicles crossing within 10 seconds of a train's arrival at the crossing were defined as an indication of risky behavior, i.e.,

TABLE 4 CL20S AT EBENEZER ROAD AND CEDAR DRIVE CROSSINGS[1]

EBENEZER ROAD CROSSING	Flashing Light Signals			Flashing Light Signals with Strobes		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	101	22	123	58	11	69
Mean (vehicles)	1.30	0.41	1.14	1.44	0.45	1.29
Standard Deviation	1.50	0.67	1.43	1.39	0.69	1.34
Percent >0 Violations	70.3	31.8	55.3	70.6	36.4	65.2
Percent >1 Violation	36.6	4.6	30.9	41.4	9.1	36.2
Range (vehicles)	0-7	0-2	0-7	0-5	0-2	0-5
CEDAR DRIVE CROSSING	**Flashing Light Signals with Predictors**			**Highway Traffic Signals with Predictors**		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	21	24	45	59	19	78
Mean (vehicles)	0.95	0.63	0.78	0.24	0.26	0.24
Standard Deviation	0.86	1.10	1.00	0.63	0.45	0.59
Percent >0 Violations	66.7	41.7	53.3	15.3	26.3	18.0
Percent >1 Violation	33.8	8.3	15.5	6.8	0.0	5.2
Range (vehicles)	0-3	0-5	0-5	0-3	0-1	0-3

[1] Vehicles crossing within 20 seconds of the train's arrival at the crossing.

[2] Includes only those observations in which vehicles were present before the train's arrival.

there is little room for either driver or vehicular error. Although also not illegal, such behavior intuitively increases the likelihood of an accident occurring. It was expected that installation of the four-quadrant flashing light signals with overhead strobes at the Ebenezer Road crossing would have no effect on this driver performance measure since nothing was changed at the crossing itself; however, installation of the highway traffic signal at the Cedar Drive crossing might have an effect because of the traffic signal's additional credibility.

As shown in Table 5, 14 CL10s were observed at the Ebenezer Road crossing in the before study—13 during the day and 1 during the night, i.e., 14 motorists crossed the tracks within 10 seconds of the train's arrival. In fact, in at least one case, two motorists crossed the tracks within 10 seconds of a train's arrival. A total of 12 CL10s were observed in the after study—11 during the day and 1 during the night. A Pearson's chi-square statistic calculated from a two-by-two contingency table indicated that the observed CL10s in the before (i.e., two-quadrant flashing light signals) and after (i.e., four-quadrant flashing light signals with overhead strobes) data sets were not significantly different at the 95 percent confidence level. It is interesting to note, however, that 24 of the 26 observed CL10s occurred during the day. The obvious conclusion is that CL10s were more likely to occur during this period; however, the reasons why are not so clear. For example, it is not clear whether fewer drivers take risk at night because they have poorer visibility of approaching trains, or whether fewer drivers take risk at night because there are fewer of them in a position to take the risk, i.e., less exposure.

Unfortunately, there was such a small number of observed CL10s in the two studies at the Cedar Drive crossing (four in both the before and after studies) that meaningful statistical comparison could not be made between them. Therefore, the premise that the additional credibility of the highway traffic signal might further reduce the number of CL10s could not be tested.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the field evaluation, there were no significant differences in driver response or safety measures between the four-quadrant flashing signals with overhead strobe lights and the standard flashing light signals. This innovative traffic control system was found to be operationally feasible, and it may have some limited application. Specific conclusions

TABLE 5 CL10S AT EBENEZER ROAD AND CEDAR DRIVE CROSSINGS[1]

EBENEZER ROAD CROSSING	Flashing Light Signals			Flashing Light Signals with strobes		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	101	22	123	58	11	69
Mean (vehicles)	0.13	0.05	0.11	0.19	0.09	0.17
Standard Deviation	0.37	0.21	1.34	0.48	0.30	0.45
Percent >0 Conflicts	11.9	4.6	10.6	15.5	9.1	14.5
Percent >1 Conflict	1.0	0.0	0.8	3.5	0.0	2.9
Range (vehicles)	0-2	0-1	0-2	0-2	0-1	0-2
CEDAR DRIVE CROSSING	**Flashing Light Signals with Predictors**			**Highway Traffic Signals with Predictors**		
	Day	Night	Total	Day	Night	Total
Sample Size[2]	21	24	45	59	19	78
Mean (vehicles)	0.19	0.08	0.13	0.05	0.05	0.05
Standard Deviation	0.51	0.41	0.46	0.22	0.22	0.22
Percent >0 Conflicts	14.3	4.2	8.9	5.1	5.3	5.1
Percent >1 Conflict	4.8	4.2	4.4	0.0	0.0	0.0
Range (vehicles)	0-2	0-2	0-2	0-1	0-1	0-1

[1] Vehicles crossing within 10 seconds of the train's arrival at the crossing.

[2] Includes only those observations in which vehicles were present prior to the train's arrival.

and recommendations regarding four-quadrant flashing light signals with strobes are summarized below:

1. Four-quadrant flashing light signals with strobes offered no apparent driver response or safety advantages over standard two-quadrant flashing signals at the test crossing.
2. Four-quadrant flashing light signals with strobes did not significantly affect violations, clearance times, approach speed profiles, maximum deceleration levels or perception-brake reaction times at the test crossing.
3. There were no accidents, confusion, or motorist diversions while the four-quadrant flashing light signals with strobes were installed.
4. The overhead strobes performed adequately throughout the 12-month test period. Their alignment was not critical to visibility, and their brightness did not "wash out" other traffic control devices. They produced no known hypnotic effects on drivers.
5. Four-quadrant flashing light signals with strobes are generally not recommended as an enhancement of standard two-quadrant flashing light signals.
6. Four-quadrant flashing light signals with strobes may be considered for use at special problem crossings where visibility to the crossing is restricted; however, cantilever signals would probably be a better or equally effective alternative.

Based on the results of the field evaluation, the highway traffic signal proved to be both feasible and effective as a grade crossing traffic control device. Driver response to the highway traffic signal was excellent, with the highway traffic signal outperforming standard flashing light signals on several key safety and driver response measures. Specific conclusions and recommendations for the highway traffic signal results are summarized below:

1. Compared to flashing light signals with predictors, the highway traffic signal reduced the number of crossings per signal activation from 3.35 to 0.73.
2. Compared with flashing light signals with predictors, the highway traffic signal reduced the risky behavior per train arrival from 0.13 to 0.05. (Risky behavior refers to the number of vehicles crossing while the flashing light signals are activated and within 10 seconds of the train.)
3. The highway traffic signal did not significantly change drivers' approach speed profile, perception-brake reaction time, or maximum deceleration level at the test crossing.

4. During the entire time that the highway traffic signal was installed at the test crossing, there were no accidents, confusion, diversions, or unnecessary delays to motorists.

5. The highway traffic signal appeared to be well understood and respected at the test crossing by the overwhelming majority of motorists.

6. From limited observation and engineering experience, there was no evidence that the use of traffic signals at grade crossings would in any way diminish their effectiveness at highway intersections; however, there were no data collected to prove or disprove this fact.

7. Credibility problems would be expected if traffic signals were used at crossings where detector malfunctions were frequent or where train warning times were long and highly variable.

8. Highway traffic signals should be tested at additional crossing sites under varying conditions and in various parts of the country. Research is needed to evaluate the long-term performance of highway traffic signals.

9. Research should be undertaken to determine if the inherent fail-safe mode of highway signals is sufficient for grade crossing applications and, if it is, if back-up power requirements can be eliminated.

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Age Differences in a Visual Information Processing Capability Underlying Traffic Control Device Usage

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Three laboratory studies addressing the magnitude of age-related differences in visual performance and their effect on delineation recognition and sign word-message legibility were conducted by using a repeated-measures experimental design. A method of limits procedure using a Landolt-C detection task defined contrast sensitivity decrements among drivers aged 65 to 80 relative to drivers aged 18 to 49; the average threshold elevation factor for all older drivers tested was in the 2 to 2.5 range, and was as high as 20 for the poorest performers in the older driver test sample. Also, a self-selected sample of older drivers with unrepresentatively good visual performance capabilities was indicated through comparison of multiple older driver groups in this research. Significant age effects were observed in quantifying the required brightness (contrast) of pavement striping to discriminate a left- from a right-bearing curve at varying distances downstream on a two-lane roadway, as well as the required character size to read single words and complete (novel) messages on regulatory, warning, and guide sign stimuli. Correlations between measured contrast sensitivity for test subjects and their performance on the two subsequent tasks were calculated; maximum variance-accounted-for by this visual performance index in the delineation recognition task was under 11 percent and reached 27 percent for the legibility task. It was concluded that cognitive factors play a significant role in driving tasks previously hypothesized to rely principally on sensory capabilities, with implications for the design of traffic control element countermeasures to accommodate the older driver population.

The percentage of older drivers on America's highways will inevitably grow in the decades ahead, reflecting a sustained trend toward the aging of the population as a whole (*1*,*2*). This trend is further accentuated by indications that the proportion of licensed drivers aged 65 and older is increasing faster than the 65+ population itself (*3*). It is therefore prudent to anticipate ways in which the present system of traffic control devices (TCDs) may fail to accommodate the special needs of this group of motorists. If the most significant deficiencies with signs, markings, and other traffic control elements as now experienced by older drivers can be pinpointed, timely design changes that can improve future levels of safety and operational efficiency on the nation's roads will be permitted. A necessary first step in any redesign effort is to obtain relevant measures of age-related differences for the full range of sensory-perceptual, cognitive, and psychomotor (movement-to-control) functions that underlie safe and effective usage of TCDs. This report presents findings that address one important aspect of driver information processing: visual performance.

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RESEARCH METHODOLOGY

Specifically, a Landolt-C visual contrast sensitivity measure was initially performed by young–middle-aged and older drivers, followed by studies assessing the relative capabilities of these groups with respect to (a) the required contrast for pavement delineation (striping) at which downstream heading on a curved roadway can be discriminated without error, both with and without the presence of veiling luminance (glare) and (b) the required letter size (subtended visual angle) at which novel word combinations and complete messages can be read on regulatory, warning, and guide signs of varying luminance, both with and without the presence of glare.

The selection of the test sample received special attention in this research, given strong evidence from literature reviews (*4*) that this area of study is characterized by exaggerated variability among older subjects, suggesting that performance-oriented comparisons of both paid participants and volunteers often may be biased in the direction of an unrepresentatively capable segment of the overall older driver distribution. Consequently, the research design for the contrast sensitivity measure provided for 30 drivers in the age range 18 to 49 and 60 drivers in the age range 65 to 80. The 30 young–middle-aged drivers and half (30) of the older drivers were solicited as paid participants through newspaper ads and in-person presentations to local American Association of Retired Persons (AARP) chapters. These groups were designated as the "regular" test samples (Groups 1 and 2). Next, the additional "cross-validation" sample of the remaining 30 drivers aged 65 to 80 was selected (Group 3). This third group was obtained through visits to Pennsylvania photo license centers, where license renewal date-birth date (day of year) determines who among the driving public walks through the door on any given day. Although still not affording a completely random selection of research participants, the latter approach arguably produced a more representative sampling of older drivers.

It is critical to note that all reported differences between test (age) groups for the later, roadway heading discrimination and sign word-message legibility studies are restricted to comparisons involving the "regular" Group 1 and 2 test samples. The cross-validation sample, identified as Group 3 in the contrast sensitivity task, did *not* serve as test participants in the subsequent studies. Thus, to the extent that Group 2 versus Group 3 differences were demonstrated for the visual performance measure, it must be assumed that all age-related differences shown by the subsequent studies will be considerably exaggerated when generalizing to the wider range of capabilities observed in the entire older driving population.

Visual Contrast Sensitivity Task

Visual contrast sensitivity undeniably contributes to the detection and recognition of many traffic control devices, with painted roadway delineation being perhaps the most crucial, and has been shown to decline significantly with advancing age. The work of Blackwell, in particular, has indicated that a 60-year-old driver may be expected to require roughly 2.5 times the contrast as a 23-year-old driver to realize the same level of target visibility (*5*). Both the brightness (luminance) of a detection-recognition target and that of its background, or surrounding roadway environment, play a role in determining contrast level; since everyone's sensitivity to contrast falls off to some extent as background luminance is reduced, high contrast is required by all drivers to see signs, roadway striping, etc., when lighting levels are low. But, the age effect noted above suggests that disproportionate increases in target (TCD element) brightness may be necessary to accommodate the elderly under nighttime driving conditions. The present contrast sensitivity task was designed to describe differences within and across age groups in this test sample on this key index of visual performance.

The methodology used in this initial laboratory task used a Landolt-C as a target stimulus—actually a ring with a gap in it—in which the subject's task was to detect the orientation of the gap. On a given presentation of the test stimulus, the gap was randomly oriented in one of eight positions corresponding to the four cardinal compass directions, plus each intermediate position. The subject was seated 20 ft from the target; at that viewing distance, the overall target diameter described a visual angle of 20 minutes and the target gap and stroke width both were 4 minutes. Dark-adapted subjects viewed the target presentations at three background luminance levels (5° surround): 0.1, 1.7, and 100 cd/m^2, respectively. The target was presented for 0.2 second on a given test trial, and both ascending and descending target contrast trials were used in a method of limits to define each person's detection threshold at each background luminance level. Target contrast was varied by changing target brightness from trial to trial, with brightness controlled through the use of Kodak Wratten neutral density (0.1 log steps) gel filters.

Roadway Heading Discrimination Study

The roadway heading discrimination experiment investigated the differences in target (delineation) contrast required to discriminate a right-bearing from a left-bearing roadway under varying distance-to-curve and glare conditions for young–middle-aged versus older driver test groups. On all trials, subjects were told to wait to respond "right" or "left" for a given roadway stimulus until they had sufficient information to actually steer their vehicle in that direction, if viewing the same scene while driving. Four roadway scenes were used as test stimuli, including right- and left-bearing 7° horizontal curves beginning at apparent (scaled) distances of 100 and 200 ft (30.5 and 61 m, respectively) downstream. A tangent section of roadway was always shown in the scene's foreground. Each of the four roadway scenes presented in this study consisted of two slides; a background slide containing the sky, roadway surround, and pavement surface, and a second, over-projected slide containing the target delineation (pavement markings). The over-projection technique allowed for independent manipulation of pavement marking brightness.

Delineation brightness attenuation was accomplished by using 3-in. (7.6-cm)-square neutral density filters mounted side-by-side on horizontally rolling glass frames interposed between projector and screen. The frames permitted a combination of 40 attenuation levels ranging from no attenuation to 3.9 units of attenuation in 0.1 log units. All test stimuli were photometered by using a Spectra Pritchard 1980A.

The brightness of every over-projected, scaled-perspective roadway scene was further corrected to display a distribution of target and background luminances consistent with the isoluminance contours produced by No. 4656 (halogen) low-beam headlight illumination. The correction was accomplished by projecting each (background *and* target) image through a "headlight mask," a mosaic of 0.5 in. (1.3-cm) squares of neutral density filter material sandwiched between two glass plates to achieve the desired isoluminance contours. Figure 1 illustrates the experimental apparatus for this study.

Upon arrival to the laboratory, each participant was seated in a chair positioned 5.6 ft (1.7 m) from a slide projection screen, providing an eye height of 3.5 ft (1.06 m). The chair was positioned to preserve the perspective of a two-lane roadway (each lane being 12 ft wide) with the first segment of a dashed white center line perceived to begin 10 ft (3.04 m) from the subject's eye. Each participant was dark-adapted for at least 10 minutes while receiving instructions. As noted earlier, participants were told to respond left or right *only* when they were as sure about the roadway heading as they would need to be to steer their car in that direction if they saw the same scene through their windshield while driving at night

Delineation stimulus presentation was blocked at two levels of disability glare: no glare and glare that averaged 0.92 lx (SD, 0.11) across trials (i.e., some variability resulting from fluctuations in line voltage and bulb wear were observed). A 12-volt bulb affixed to the projection screen served as the glare source; the bulb was positioned 6 in. (15.2 cm) laterally from the point of road curvature in the scene, or approximately 6° off of the driver's forward line of sight. The 0.92-lx glare level is consistent with the intensity of an oncoming vehicle's low-beam halogen headlights seen from a distance of 100 ft, assuming 12-ft (3.66-m)-wide lanes on a two-lane roadway. Illuminance (glare source) measurements were recorded for each subject by using a Minolta model T-1 illuminance meter held at the subject's eye position during data collection.

All trials at the no-glare level were completed before the presentation of any glare trials to prevent transient adaptive effects from a glare trial from interfering with performance on a subsequent no-glare trial. Also, the different simulated

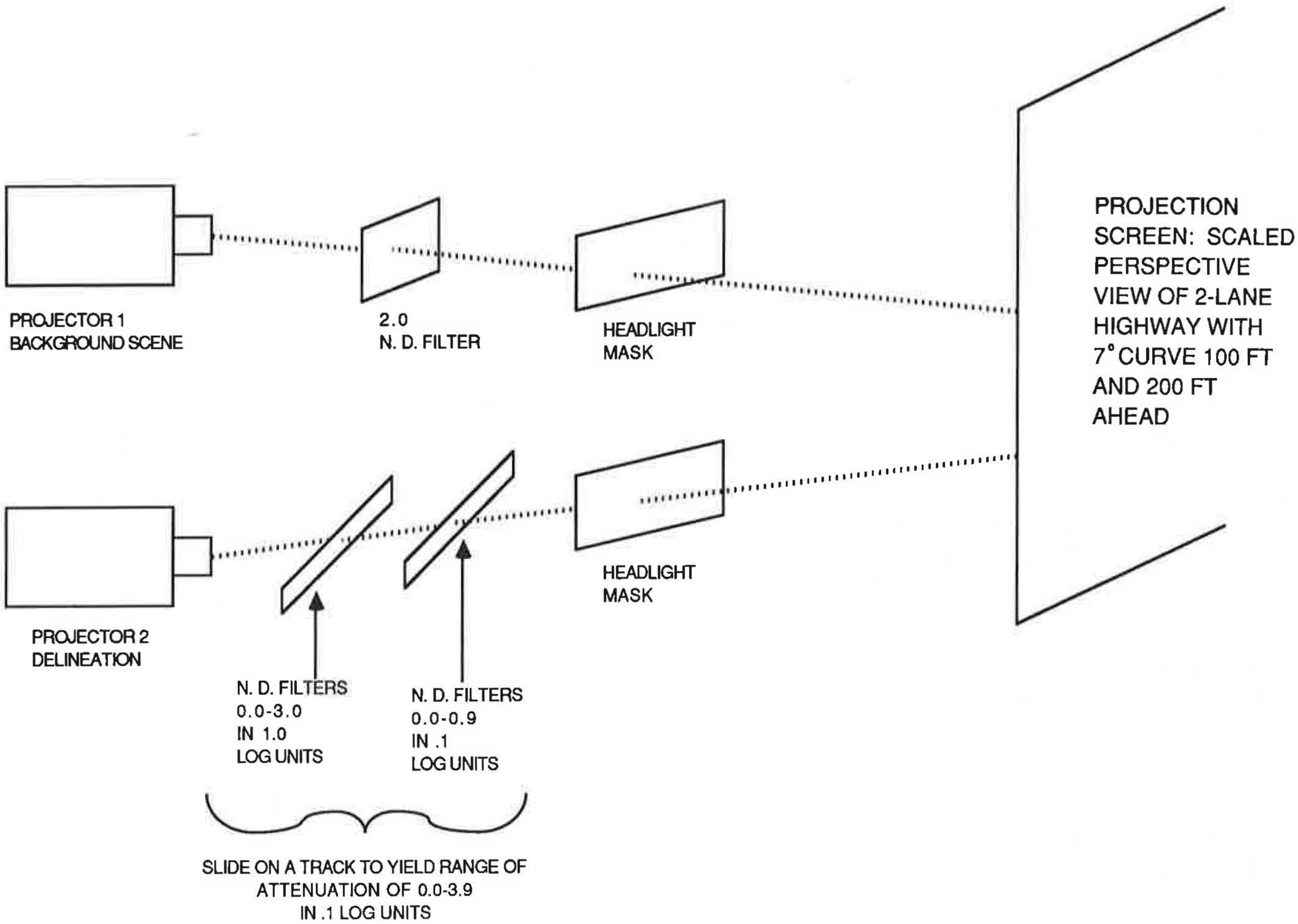

FIGURE 1 Arrangement of experimental apparatus.

observer-target separation distances for both left- and right-bearing curves were presented in random order. The duration of each slide presentation was 0.5 second; a trial consisted of a series of 0.5-second presentations of a given test stimulus in which each successive exposure used the neutral density filters to depict the delineation in either marginally higher or marginally lower contrast to the background roadway scene. The intertrial interval was 5 seconds.

Both ascending and descending brightness trials were presented in the method of limits to define each subject's heading determination threshold at each target separation distance. On ascending brightness trials, subjects were required to respond correctly three consecutive times before the next trial was presented. Descending brightness trials ended when a subject responded that he or she was uncertain of the roadway heading. Subjects were given at least two practice trials to make certain that they understood the test protocol. Subjects completed three replications of ascending brightness and three replications of descending brightness trials for each stimulus slide for each level of glare, for a total of 48 trials.

Sign Word-Message Legibility Study

After completing the delineation study, subjects were given a 10-minute break. They were then brought back into the laboratory, where they were seated in a chair positioned 20 ft (6.09 m) from the projection screen and were again dark-adapted while receiving instructions. The experimenter explained the task of trying to read a sign message at a glance, for (guide) signs with white lettering on a green background, (warning) signs with black lettering on a yellow background, and (regulatory) signs with black lettering on a white background. Subjects were advised that each sign would contain a four-word message constructed from common words combined into novel phrases, for example, NARROW BUSES MUST YIELD (regulatory), NEXT ROUGH DETOUR HILL (warning), and STATE BORDER ACCESS ROAD (guide).

The first sign projected on the screen was too small to read for all subjects, with sign size (and therefore letter size) increasing with each successive presentation. A tone was heard before each stimulus presentation, as a "ready" signal. Subjects were asked to respond verbally when they could first detect any individual words, and also when they could read the entire message.

Since the viewing distance was held constant at 20 ft, letter size was manipulated by varying the visual angle subtended by the letter at the subjects' eyes. Letter size ranged from the Snellen equivalent of 20/12.5 (visual angle = 0.625 minute) to 20/125 (visual angle = 6.25 minutes), in increments of ½-line Snellen acuity differences. (NOTE: Snellen letters are five times their stroke width in height; a stroke width that subtends 1 minute of visual angle at a viewing distance of 20 ft defines normal acuity of 20/20.)

Franklin Gothic lettering used for the upper case regulatory and warning sign letters and Helvetica Bold lettering used for the lower case letters on the guide sign stimuli closely resembled the Series D font used on traffic signs. The lettering was placed on clear acetate following the spacing guidelines specified in the Standard Highway Signs manual. The messages were then overlaid on 3M engineering grade sheeting of the desired color and photographed. Variation in letter size for the test stimuli was achieved by progressively zooming in while photographing a projected image of each sign.

Each stimulus was presented for a 0.5-second duration, with a trial consisting of a series of 0.5-second presentations of successive letter size increments for a particular test stimulus in which a legibility response was required for each presentation. The experimenter recorded the letter size at which the subject could first correctly detect any word, and the letter size at which the entire message could be read. A trial ended when the subject could correctly read the message or when the letter size corresponding to 20/125 was presented without a correct legibility response, whichever occurred first.

The mean message length across all signs was 19.94 letters, and ranged from 18 to 22 letters per message. Messages were constructed from four-, five-, and six-letter words. It was important to hold message length constant to avoid confounding message length with message legibility. In all, 54 four-word combinations were devised for this study, divided into three sets of 18 each regulatory, warning, and guide sign stimuli. Overall, the test conditions for this study permitted three novel message replications for every combination of sign type and luminance level and glare condition.

Stimulus presentation was blocked at two levels of glare: no glare, and glare that averaged 1.26 lx (SD = 0.33) across trials. As in the earlier study, some variability across trials resulted from fluctuations in line voltage and bulb wear. A 12-volt bulb again served as the glare source but located in this study approximately 6° off the subject's forward line of sight on a stand near the subject's seating position. Illuminance (glare source) measurements were recorded for each subject by using a Minolta model T-1 illuminance meter held at the subject's eye position during data collection. All no-glare trials were presented before presentation of any glare trials to prevent transient adaptive effects from a glare trial from influencing performance on a subsequent no-glare trial.

Stimulus luminance was also varied in this study, to simulate the effect of viewing real-world signs set back at increasing distances from the roadway edge (and therefore at lower luminances). Three luminance levels were employed for each of the three sign categories of interest. Specific luminance values in candelas per square meter (cd/m^2) for regulatory sign stimuli at levels L_1, L_2, and L_3 were 0.126, 0.080, and 0.050 for targets (letters) and 3.44, 2.17, and 1.37 for the corresponding background (sign panels). For warning sign stimuli, actual target luminance values tested were 0.099, 0.063, and 0.031, with corresponding background values of 2.14, 1.35, and 0.677. For guide sign stimuli, L_1, L_2, and L_3 for the letters equaled 1.10, 0.754, and 0.475, with values of 0.089, 0.060, and 0.038 for the corresponding sign panels. As before, all luminance measures were obtained by using a Spectra Pritchard 1980A photometer.

Contrast values remained constant within each sign category, as both target and background elements were attenuated by using common neutral density filter factors for luminance conditions L_2 and L_3, respectively, relative to the highest (L_1) luminance condition. Calculated contrast values $[(L_t - L_b)/L_b]$ for the regulatory, warning, and guide sign stimuli were -0.96, -0.95, and 11.5, respectively.

RESULTS AND CONCLUSIONS

Participants actually completing data collection requirements for the contrast sensitivity task included 14 males and 16 females in the young–middle-aged test group (Group 1), with an overall age range of 19 to 49 and a median age of 35; 16 males and 15 females in the "regular" older sample (Group 2), with an overall age range of 65 to 80 and a median age of 69; and 10 males and 9 females in the "cross-validation" older sample (Group 3), with an overall age range of 65 to 77 and a median age of 69. Only Group 3 experienced any attrition, with six individuals dropping out due to fatigue or lack of interest, and five others excused because of equipment malfunction in the laboratory.

For the roadway heading discrimination study, the participants from the young–middle-aged test sample completing data collection requirements included 14 males and 15 females with an overall age range of 19 to 49 and a median age of 35.5; and 15 males and 15 females from the regular older driver sample, with an overall age range of 65 to 80 and a median age of 69. Finally, participants from the young–middle-aged test sample completing data collection requirements for the sign word-message legibility study included 12 males and 16 females with an overall age range of 19 to 49 and a median age of 35; and 15 males and 15 females from the regular older driver sample, with an overall age range of 65 to 80 and a median age of 69.5.

Visual Contrast Sensitivity Test

The contrast sensitivity data were blocked for analysis at each of the three included background luminance (L_b) levels for comparison of the detection thresholds of the driver groups tested in this research. Within each L_b level, each subject's threshold was determined by translating the neutral density setting at which five out of eight correct responses were obtained (to compensate for guessing) to a target luminance value, then substituting into the expression $(L_t - L_b)/L_b$ to result in a contrast value (C).

Mean and median contrast values at threshold, plus standard deviations, are presented in Table 1 for the younger and older regular samples and the older cross-validation sample tested in this research, designated as study Groups 1, 2, and 3, respectively, in this report. As shown in this table, the mean and standard deviation threshold contrast values suggest dramatic differences between groups under the lowest lighting condition; under progressively higher background luminance levels, the pattern of differences remains constant, but the percent change from one group to another becomes less pronounced. The shift in median performance levels across lighting conditions is much more stable, reflecting the presence of extreme data points distributed among Group 2 and, especially, Group 3.

A series of six t-tests were planned to evaluate the obtained differences between Groups 1 and 2 and between Groups 2

TABLE 1 CONTRAST SENSITIVITY (4′ LANDOLT-C TARGET) THRESHOLDS FOR INCLUDED STUDY GROUPS

	Threshold Contrast (C)								
	@L_{b1} (0.1 cd/m^2)			@L_{b2} (1.7 cd/m^2)			@L_{b3} (100 cd/m^2)		
	$\bar{X}$	med.	σ	$\bar{X}$	med.	σ	$\bar{X}$	med.	σ
Group 1 (n=30)	1.05	0.85	0.70	0.21	0.17	0.12	0.11	0.08	0.09
Group 2 (n=31)	4.35	1.83	9.40	0.45	0.35	0.28	0.15	0.14	0.11
Group 3 (n=22)	20.91	2.48	69.28	2.52	0.45	4.82	0.30	0.17	0.34

NOTE: To convert cd/m^2 to fL, multiply by 0.292

TABLE 2 MEAN AND STANDARD DEVIATION CONTRAST REQUIREMENTS FOR DELINEATION RECOGNITION/ROADWAY HEADING DISCRIMINATION BY TEST (AGE) GROUP, DISTANCE TO CURVE, AND GLARE CONDITION

Test (age) group	Distance to curve							
	100 ft				200 ft			
	No glare		Glare		No glare		Glare	
	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ	$\bar{X}$	σ
young/middle-aged	1.20	.36	2.42	1.69	1.23	.29	2.35	1.16
elderly	1.27	.54	2.88	2.06	1.32	.51	3.25	3.05

and 3 at each background luminance level. After preliminary F_{max} tests led to rejection of a hypothesis of homogeneity of variance for the comparisons of Group 1 and Group 2 at L_{b1}, and Group 2 and Group 3 also at L_{b1}, a log transformation was applied to these data before conducting the *t*-tests.

The outcomes of the *t*-tests indicated statistically significant or marginally significant differences for five out of six comparisons; only the comparison between the performance of Groups 1 and 2 at L_{b3}, the brightest background condition, did not approach statistical significance. For the Group 1-Group 2 comparisons at L_{b1} and L_{b2}, respectively, $t = 5.40$ ($P < .001$) and $t = 4.24$ ($P < .001$), for degrees of freedom (d.f.) = 59. For the Group 2-Group 3 comparisons at L_{b1}, L_{b2}, and L_{b3}, respectively, $t = 1.60$ ($P < .06$), $t = 2.39$ ($P < .02$), and $t = 2.35$ ($P < .02$), for d.f. = 51.

These findings lead to at least two important conclusions. First, the spread between the visual capabilities of young–middle-aged drivers (Group 1) and a self-selected sample of older motorists (Group 2) is consistent with the substantial differences contained within a standard reference summarizing results of Blackwell and others (*5*). More seriously, there is evidence of a pronounced selection bias in these data, such that a large proportion of active, older drivers may in fact suffer far greater visual performance deficits than are typically detected in psychophysical studies of this nature.

Roadway Heading Discrimination Study

Results of the roadway heading discrimination study are described by the more detailed data summary presented in Table 2, which contrasts the mean and standard deviation performance of older versus young–middle-aged drivers according to glare condition and distance-to-curve condition in this study. It is apparent from reviewing the data in this table that, although the introduction of glare affected both test (age) groups, the effect of the range of reduced target (task detail) size associated with increasing distance-to-curve in this study was limited to a performance decrement among the older driver group only. Further, when the data are collapsed across glare and distance conditions to calculate an overall effect of test (age) group, the older test sample required a level of contrast 20 percent greater than that for the young–middle-aged group to achieve the discrimination task in this study.

Statistical tests conducted on the data in this study included a three-way analysis of variance (ANOVA) to examine the main effects and possible interactions of the variables test (age) group, glare (present versus Table 2 absent), and distance-to-curve. The direction of curvature was not included as a variable in data analysis, having been introduced as a stimulus condition to define the discrimination task (right versus left), with the order of right versus left presentations randomized across all trials in the study.

The findings of the ANOVA included the hypothesized main effect of test (age) group ($F = 6.77$; d.f. = 1, 440; $P < .01$); an even stronger effect of glare ($F = 103.7$; d.f. = 1, 440; $P < .001$); and, a significant test (age) group-by-glare interaction ($F = 4.18$; d.f. = 1, 440; $P < .04$). A Scheffe post-hoc test indicated that both variables made a significant ($P < .05$) contribution to this interaction, even though the magnitude of the main effect associated with glare condition was considerably larger than that associated with the test (age) group.

An additional, one-way ANOVA was conducted to test for a main effect of gender on performance in this study. Although an exaggerated decrement in performance was noted for older females versus older males, just the opposite finding was observed among the young–middle-aged test group. Overall, the differences attributable to this factor were shown to be not statistically significant.

Further analysis of these data consisted of Pearson product-moment correlations between the contrast at threshold for the present discrimination task, versus the tested contrast sensitivity for each subject as measured earlier. Of course, for the present set of correlations, only the contrast sensitivity measures for those subjects actually completing this study could be used.

The results of this analysis are presented in Table 3. Interestingly, the strongest correlation—and greatest amount of variance accounted for in this study—is demonstrated for subjects' tested contrast sensitivity at a background luminance of 1.7 cd/m^2. This finding is consistent with the mesopic conditions that frequently characterize nighttime driving. More surprising, none of the correlations are high in an absolute sense; thus, an important conclusion implied by these results is that nonsensory factors play a prominent role in driver discriminations of downstream roadway heading, given the visual cues available to test subjects in this study.

TABLE 3 PEARSON PRODUCT-MOMENT CORRELATIONS BETWEEN MEASURED CONTRAST SENSITIVITY AT VARYING BACKGROUND LUMINANCES AND MEAN CONTRAST REQUIREMENTS FOR DELINEATION RECOGNITION/ROADWAY HEADING DISCRIMINATION (GLARE ABSENT) WITH CALCULATED r^2

Background (adaptation) luminance	Correlation, r	Variance-accounted-for, r^2
$L_b = 0.1$ cd/m^2	.099	1.0%
$L_b = 1.7$ cd/m^2	.328	10.8%
$L_b = 100$ cd/m^2	.261	6.8%

Sign Word-Message Legibility Study

Results of the sign word-message legibility study are presented beginning with summary descriptive statistics documenting the overall effect of test (age) group on the two performance measures of interest in this study, as shown in Table 4. For the reader's convenience, the dependent measure is reported both in terms of minutes of visual angle of character stroke width, as well as in terms of an equivalent Snellen fraction denominator. Apparent trends in this summary of data include a consistently superior performance for young–middle-aged and older test groups alike on the negative versus positive contrast stimuli; also, for both groups, the letter size required for complete message legibility was consistently larger than that required to discern individual words on a sign. In comparisons between groups, however, the older driver sample without exception demonstrated a need for larger mean letter sizes, plus elevated standard deviations, relative to the younger test sample.

When performance in this study is broken down by one additional level to examine separately the conditions of glare versus no-glare, there is no evidence of any clear effect. The responses of the young–middle-aged group actually showed a marginal improvement when glare was present, although

TABLE 4 MEAN AND STANDARD DEVIATION CHARACTER SIZE EXPRESSED IN MINUTES OF VISUAL ANGLE (WITH SNELLEN FRACTION DENOMINATOR EQUIVALENT) REQUIRED FOR REGULATORY, WARNING, AND GUIDE SIGN WORD AND MESSAGE LEGIBILITY AS A FUNCTION OF TEST (AGE) GROUP ONLY

Test (age) group	Sign Type	Character size (Snellen denominator)			
		Word		Message	
		$\overline{x}$	σ	$\overline{x}$	σ
young/middle-aged	regulatory	1.33 (26.6)	.45 (9.0)	1.92 (38.4)	.57 (11.4)
	warning	1.29 (25.8)	.46 (9.2)	1.92 (38.4)	.63 (12.6)
	guide	1.79 (35.8)	.57 (11.4)	2.68 (53.6)	.86 (17.2)
elderly	regulatory	1.82 (36.4)	.53 (10.6)	2.57 (51.4)	.75 (15.0)
	warning	1.84 (36.8)	.59 (11.8)	2.71 (54.2)	.89 (17.8)
	guide	2.52 (50.4)	.82 (16.4)	3.78 (75.6)	1.31 (26.2)

TABLE 5 MEAN AND STANDARD DEVIATION CHARACTER SIZE EXPRESSED IN MINUTES OF VISUAL ANGLE (WITH SNELLEN FRACTION DENOMINATOR EQUIVALENT) REQUIRED FOR REGULATORY, WARNING, AND GUIDE SIGN WORD AND MESSAGE LEGIBILITY AS A FUNCTION OF TEST (AGE) GROUP AND STIMULUS LUMINANCE LEVEL (NO-GLARE TRIALS ONLY)

a. word legibility

test (age) group	sign type	Character size (Snellen denominator)					
		L_1		L_2		L_3	
		$\bar{x}$	σ	$\bar{x}$	σ	$\bar{x}$	σ
young/middle-aged	regulatory	1.36 (27.2)	.33 (6.6)	1.35 (27.0)	.37 (7.4)	1.34 (26.8)	.50 (10.0)
	warning	1.26 (25.2)	.42 (8.4)	1.35 (27.0)	.53 (10.6)	1.32 (26.4)	.46 (9.2)
	guide	1.86 (37.2)	.51 (10.2)	1.92 (38.4)	.61 (12.2)	1.81 (36.2)	.60 (12.0)
elderly	regulatory	1.75 (35.0)	.42 (8.4)	1.77 (35.4)	.49 (9.8)	1.83 (36.6)	.57 (11.4)
	warning	1.72 (34.4)	.42 (8.4)	1.79 (35.8)	.54 (10.8)	1.84 (36.8)	.55 (11.0)
	guide	2.31 (46.2)	.66 (13.2)	2.60 (52.0)	.73 (14.6)	2.58 (51.6)	.81 (16.2)

b. message legibility

test (age) group	sign type	Character size (Snellen denominator)					
		L_1		L_2		L_3	
		$\bar{x}$	σ	$\bar{x}$	σ	$\bar{x}$	σ
young/middle-aged	regulatory	1.98 (39.6)	.46 (9.2)	1.98 (39.6)	.53 (10.6)	1.93 (38.6)	.65 (13.0)
	warning	1.95 (39.0)	.57 (11.4)	1.96 (39.2)	.74 (14.8)	1.96 (39.2)	.58 (11.6)
	guide	2.87 (57.4)	.81 (16.2)	2.78 (55.6)	.87 (17.4)	2.72 (54.4)	.93 (18.6)
elderly	regulatory	2.48 (49.6)	.56 (11.2)	2.50 (50.0)	.69 (13.8)	2.58 (51.6)	.84 (16.8)
	warning	2.59 (51.8)	.70 (14.0)	2.57 (51.4)	.78 (15.6)	2.73 (54.6)	.86 (17.2)
	guide	3.77 (75.4)	1.06 (21.2)	3.89 (77.8)	1.23 (24.6)	3.94 (78.8)	1.25 (25.0)

the older group generally demonstrated the expected performance decrement for the comparisons of glare versus no-glare. In all cases, however, the apparent differences attributable to glare were very slight, described by at most a 10 percent shift in required target size for word and message legibility between glare and no-glare conditions.

This finding is in sharp contrast to the highly significant effect of this factor in the roadway heading discrimination study. A likely explanation for this outcome follows from the placement of the glare source in the respective studies. In the previous study the glare source was attached to the projection screen and thus remained in a constant, fixed position with respect to the (projected) test stimuli; in this study, the glare source was attached to a stand and positioned only a few feet in front of the subject at the desired angle of eccentricity. With the latter arrangement, a very slight leaning of the subject's head to one side at stimulus onset—a gesture that would have been difficult to observe from the experimenter's station in this study—could have substantially attenuated the veiling luminance experienced by the subject.

Finally, Table 5 presents a further breakdown of performance (for no-glare trials only) according to stimulus luminance level (L_1, L_2, and L_3 as defined earlier). Patterns in these data may be described that again indicate the need for larger letter sizes, accompanied by larger standard deviations, for the older versus the younger test (age) group, across both glare conditions. Within the test group, however, the older subjects typically demonstrated their best performance (i.e., smallest visual angles of stimuli required for legibility) under the highest luminance condition (L_1), whereas the younger subjects performed at levels that were roughly constant across luminance conditions or were superior at L_2 or L_3, as opposed to L_1. For both groups, it remains apparent that the positive contrast (guide) signs were consistently the most difficult to read across all luminance conditions.

Statistical tests conducted on these data included, first, a set of two-way ANOVA blocked according to sign type and glare condition, which separately examined word and message legibility performance differences as a function of test (age) group, stimulus luminance level, and the group-by-luminance level interaction. In general, only differences between test (age) groups were shown to be statistically significant; a main effect of stimulus luminance level was demonstrated only for guide signs.

Specifically, for regulatory signs, the main effect of test (age) group was significant at $P < .001$ for word legibility under both glare and no-glare conditions ($F = 45.9$; d.f. = 1, 168 and $F = 39.0$; d.f. = 1, 168, respectively); with message legibility as the dependent measure, main effects were similarly demonstrated at $P < .001$ ($F = 47.6$; d.f. = 1, 168 with glare present and $F = 33.7$; d.f. = 1, 168 with glare absent). No main effects of luminance level or interactions of test (age) group with stimulus luminance were shown for regulatory signs in this study, for either word or message legibility, under either glare or no-glare conditions.

An identical pattern of results was demonstrated for warning signs. The main effect of test (age) group was significant at $P < .001$ for the word legibility measure with glare present ($F = 54.6$; d.f. = 1, 167) and with glare absent ($F = 39.6$; d.f. = 1, 169), and also at $P < .001$ for the message legibility measure with glare present ($F = 49.0$; d.f. = 1, 167) and with glare absent ($F = 39.0$; d.f. = 1, 169). Again, no main effects of stimulus luminance level or interactions between test (age) group and stimulus luminance were shown for either word or message legibility either with or without glare.

For guide signs, the expected main effect of test (age) group was significant at $P < .001$ for the word legibility measure with glare present ($F = 54.7$; d.f. = 1, 168) and with glare absent ($F = 39.0$; d.f. = 1, 168), and also at $P < .001$ for the message legibility measure with glare present ($F = 41.1$; d.f. = 1, 168) and with glare absent ($F = 46.0$; d.f. = 1, 168). As noted above, main effects of stimulus luminance level were also demonstrated for this sign type, though only with glare present and more significantly for the message than for the single-word legibility measure. With the complete message response requirement, the effect of stimulus luminance level was significant at $P < .01$ under the glare condition ($F = 4.2$; d.f. = 2, 168) but did not approach significance when glare was absent. When the response requirement was to read a single word as opposed to the entire message on a guide sign, the significance of the effect of stimulus luminance was marginal with glare present ($F = 2.7$; d.f. = 2, 168; $P < .07$) and again negligible under the no-glare condition. No significant interactions of test (age) group and stimulus level were noted for either the word or message legibility measures under either glare or no-glare conditions.

An additional one-way ANOVA was conducted to test for main effects of subjects' gender on the word and message legibility measures for each sign type. For regulatory signs, the effect of sex did not approach significance for either word legibility or message legibility performance measures. Likewise, the warning sign data showed no significant differences due to the gender of subjects, for either word legibility or message legibility. The performance differences between males and females with respect only to word legibility on guide signs, although not significant, did approach the conventional 0.05 cutoff ($F = 3.35$; d.f. = 1, 344; $P < .07$); in terms of the absolute magnitude of differences between sexes on this single response measure, females averaged 6.7 percent better than males across all test conditions. When performance on the message—as opposed to word—legibility measure for guide signs was analyzed, the effect of gender was diminished and did not approach statistical significance.

As in the roadway heading discrimination study, Pearson product-moment correlations were calculated between subjects' measured contrast sensitivity, at three background (adaptation) luminance levels, and their performance on the dependent measures in this study. Table 6 displays the results of this analysis by sign type and stimulus luminance level. As shown in this table, the measured contrast sensitivity of subjects at the lower (0.1 and 1.7 cd/m^2) background luminances were correlated most strongly with performance on both the word and message legibility measures, across all three stimulus (sign) luminance levels tested in the laboratory. This is not surprising, since the range of sign luminances presented to subjects fell roughly between 0.03 and 3.0 cd/m^2.

Again, the magnitudes of the variance-accounted-for figures in the correlational analysis are most interesting. Accounting for 25 percent and more of the variance in a realistic driving performance measure on the basis of a single psychophysical indicator is potentially a useful finding. The increased magnitudes of the obtained r^2 values in this study relative to the previous effort also deserve mention; arguably, the sensory (visual screening) data are a stronger predictor of task performance when the subject is performing a feature-matching response such as letter-word legibility than when performing the more ambiguous delineation recognition task.

GENERAL DISCUSSION

This report suggests a special concern regarding one indication of the decline in visual performance capability among aged adults. A critical first step in a driver's processing of the information provided by TCDs is access to the full range of sensory inputs afforded by a normal, healthy visual system. Because of increased light absorption and scattering in the crystalline lens (*6*), however, the eyes of older drivers require a markedly higher level of contrast for objects in the roadway environment to perform as safely and effectively as younger drivers. Specifically, the present findings suggest that at night roughly 2 to 2.5 times more contrast is needed by the median or 50th percentile older driver, whereas individuals representative of the lowest quartile of visual performance among this age group—including persons who do report driving at night, at least occasionally—may require 10 to 20 times more contrast than an average younger driver.

This diminished visual capability was hypothesized to have the strongest impact on the use of various pavement markings and on sign legibility in this research. In fact, older drivers participating in focus groups earlier in this project (*7*) complained vigorously about missing or faded edgelines, about undelineated lanes at the "aim points" when completing left turns at intersections, and, to a lesser extent, about difficulty in reading road signs. Also, these motorists reported associated problems including hesitation and erratic driving

TABLE 6 PEARSON PRODUCT-MOMENT CORRELATIONS (r) BETWEEN MEASURED CONTRAST SENSITIVITY AT VARYING BACKGROUND LUMINANCES AND MEAN CHARACTER SIZE REQUIREMENT FOR WORD AND MESSAGE LEGIBILITY (GLARE ABSENT) WITH CALCULATED r^2 (VARIANCE-ACCOUNTED-FOR)

Background (adaptation) luminance	Legibility performance measure	Sign type	Stimulus luminance level L_1 r	L_1 r^2	L_2 r	L_2 r^2	L_3 r	L_3 r^2
$L_b = 0.1\ cd/m^2$	word	regulatory	.482	23.2%	.446	19.9%	.454	20.6%
		warning	.346	12.0%	.386	14.9%	.483	23.3%
		guide	.517	26.7%	.404	16.3%	.422	17.8%
	message	regulatory	.493	24.3%	.471	22.2%	.438	19.2%
		warning	.466	21.7%	.394	15.5%	.469	22.0%
		guide	.508	25.8%	.442	19.5%	.404	16.3%
$L_b = 1.7\ cd/m^2$	word	regulatory	.528	27.9%	.446	19.9%	.459	21.1%
		warning	.473	22.4%	.424	18.0%	.473	22.4%
		guide	.490	24.0%	.465	21.6%	.460	21.2%
	message	regulatory	.483	23.3%	.513	26.3%	.476	22.7%
		warning	.500	25.0%	.418	17.5%	.489	23.9%
		guide	.492	24.2%	.520	27.0%	.424	18.0%
$L_b = 100\ cd/m^2$	word	regulatory	.220	4.8%	.126	1.6%	.150	2.3%
		warning	.212	4.5%	.213	4.5%	.215	4.6%
		guide	.238	5.7%	.195	3.8%	.169	2.9%
	message	regulatory	.288	8.3%	.258	6.7%	.232	5.4%
		warning	.253	6.4%	.265	7.0%	.245	6.0%
		guide	.298	8.9%	.273	7.5%	.183	3.3%

behaviors as they seek the additional information needed to accomplish intended vehicle maneuvers.

Despite the magnitude of differences observed in the contrast sensitivity measure, however, a relatively small percentage of variance was accounted for in drivers' responses, particularly on the delineation recognition task. Possible explanations suggested by the technical literature (*4*) include hypothesized deficits in selective attention or pattern recognition-integration or, more generally, a fundamental difference in strategy where older drivers required greater certainty before responding. In any event, the apparent contribution of cognitive factors to the present results suggest that design guidelines for retroreflective traffic control elements should take note of driver performance variables over and above those "purely sensory" deficits long recognized to accompany advancing age.

Probably the single most important outcome to emphasize in this discussion is the tremendous increase in variability of performance among older drivers. This aspect of behavior poses the greatest challenge to TCD redesign efforts to accommodate older drivers, given a population in which the most capable individuals can meet and often exceed performance expectations for any age group. Also, the magnitudes of variability observed among the two groups of older drivers participating in this research indicate a substantial self-selection bias, in which unrealistically high estimates of the older driver population's capabilities were produced by those individuals recruited through responses to newspaper advertisements and solicitations at AARP chapters. Clearly, it is essential to exercise special care in sampling the older driver population when deriving estimates of performance capabilities. As work continues to investigate the relationship between age and traffic control device use, researchers and policymakers must aggressively challenge the credibility of findings generated by volunteer or otherwise unrepresentative test samples of older drivers.

As a final note, the role of complementary efforts at the state level to develop assessment and qualifications programs to identify diminished capability drivers deserves mention. Such screening not only has the potential to moderate the demand for changes in the current system of TCDs; if equitably administered it more properly focuses attention on a driver's abilities instead of on his or her age per se.

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