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Driver Information Needs and Visibility of Traffic Control Devices

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Abridgment

Applicability of Guidance and Navigation Systems

GERHART F. KING

ABSTRACT

Vehicle location, guidance, and navigation systems of varying degrees of complexity are receiving increasing attention in an attempt to reduce adverse energy consumption safety, and environmental effects of excess travel as a result of navigational failures. The various systems advocated or proposed apply to different types of travel as described by trip length, trip purpose, driver demographic characteristics, and other factors. The National Personal Transportation Study data base was analyzed to determine the distribution of a number of pertinent variables. These data are presented and examples are provided of how the results of this analysis can be used to determine the applicability of specific system configurations.

Driver information systems may apply to all or only some of the drivers exposed to the information sources. Most regulatory signing falls into the first category; directional signing is a good example of the second type. Information system design must include consideration of constraints imposed by characteristics of the specific user population. Economic analyses of potential improvements in these systems must also include consideration of the number of drivers and the number and characteristics of vehicle trips to which they apply. This paper contains applications of these concepts to one particular class of information systems--systems that deal with one or more aspects of route planning, navigation, and route guidance. One function of these systems is to minimize excess travel (i.e., the difference between actual highway travel and the amount that would occur if each motor vehicle trip used an optimum route). Excess travel results from the interaction of deficiencies in route selection criteria and in route planning and following.

A number of previous research efforts, both in the United States and abroad (1-3) have attempted to quantify excess travel. Excess travel was found to be a significant component of total travel with one study finding this proportion to exceed 40 percent. A synthesis of all studies of this type made in England led to the conclusions that between 2 and 4 percent of all travel represented waste that might be eliminated, and that for certain trip purpose-trip length combinations, this proportion of waste increased to 20 percent (4,5).

REMEDIAL MEASURES

Remedial measures that have been conceived, advocated, designed, or tried cover a great range of techniques, devices, or systems that include

1. Improved trip planning and map reading skills;
2. Improved availability, accuracy, and legibility of highway maps;
3. Computer-assisted trip planning;
4. Improvements in the highway information system;
5. Improved radio broadcast traffic condition advisory; and
6. Improvements in vehicle location and navigation systems.

To make an economic assessment of the measures, it is necessary to determine not only the effectiveness of the individual remedial measure to abate the excess travel problem, but also the specific characteristics of the trips for which such measures are applicable. The applicability of a given type of remedial measure and the benefits to be derived therefrom are functions of both trip and driver characteristics. In this context, the term applicability addresses whether a system can be used as well as whether it will be used.

Trip characteristics of interest include length and purpose. With increased trip length, there will be a consequent increase in the probability of entering relatively unfamiliar territory, in the number of alternate routes available, and in the number of decision points. Consequently, the probability of error is likely to increase. Applicability of a given remedial measure can thus be expected only if trips exceed some minimum length.

Trip purpose interacts with trip length and also serves as a rough indication of route familiarity. Furthermore, trip purpose is also correlated with trip urgency, that is, with the driver's desire to optimize his route and with the economic effects of departures from such an optimum.

All potential remedial measures affect the perceptual and cognitive demands placed on the driver. Different information sources must be detected, recognized, and sampled, and the information so obtained must be processed. The process of information acquisition and utilization must be time-shared with the preeminent perceptual and cognitive demands of operating a vehicle in a traffic stream. In some cases (e.g., consulting a detailed, small-scale map) such time-sharing is impossible if safe vehicle operating conditions are to be maintained. The presence of another person in the vehicle, who can act as a navigator, would eliminate this problem.

TRIP AND DRIVER CHARACTERISTICS

A complete assessment of the applicability of cost-effectiveness of remedial measures for navigational waste thus requires data on the distribution of trip and driver characteristics. These data were obtained from the National Personal Transportation Study (NPTS) (6).

Previous analysis of the problem has shown that the applicability of various navigation and guidance

TABLE 1 Mileage by Trip Purpose *(6)*

	Minimum Trip Length								
	1 Mi			5 Mi			9 Mi		
Trip Purpose	Male	Female	Total	Male	Female	Total	Male	Female	Total
A Earning a living[a]	30.5	9.5	40.0	32.4	9.5	41.9	33.9	9.0	42.8
B Family and personal business	7.2	6.0	13.2	7.0	5.3	12.3	6.8	4.8	11.6
C Civic, educational, or religious	1.6	1.2	2.8	1.5	1.1	2.6	1.4	1.0	2.4
D Social and recreational	10.0	4.1	14.1	10.2	4.1	14.3	10.7	4.1	14.8
E Return home[a]	14.9	8.3	23.2	15.0	7.5	22.5	15.3	7.1	22.3
F Other[a]	3.9	2.8	6.7	3.8	2.5	6.3	3.8	2.2	6.0
Total	68.0	32.0	100.0	69.9	30.1	100.0	71.8	28.2	100.0

[a]The basic NPTS Trip Purpose Classification includes all trips whose destination is "home" into aggregate class "other" (including work-to-home trips).

TABLE 2 Driver Median Age *(6,7)*

Trip Purpose	Day Male	Day Female	Day All	Night Male	Night Female	Night All	All Male	All Female	All All
A	39	32	37	38	35	37	39	33	37
B	40	37	38	35	35	35	38	37	37
C	28	30	30	39	34	36	30	30	30
D	34	34	34	28	28	28	34	32	33
E	40	36	37	34	33	34	36	35	36
F	35	33	33	31	32	32	33	33	33
Total	38	34	37	35	33	34	37	34	36
U.S. Population, 16 and older							37	40	38

techniques is a function of trip length and purpose while the usability of these techniques depends on such driver characteristics as age, sex, and education, and on the presence of a navigator.

Trip length by trip purpose is shown in Table 1. A tabular presentation of the role played by longer trips follows.

Minimum Trip Length (mi)	Vehicle Miles Traveled	Trips
5	.87	.45
9	.72	.26
14	.57	.15
24	.38	.06

No data are available concerning driving experience; however, this variable is highly correlated with age. Table 2 shows the median driver age for each trip purpose and for day and night conditions. Table 3 shows that proportionately less driving is done at each tail of the distribution of driver age.

The educational level of drivers is higher than that of the population as a whole as can be seen by comparing proportions of drivers who have completed 12 or more years of school such as high school graduates (6).

Population Group	Percent of Individuals Who Have Completed 12 or More Years of School (%)
Total U.S. population	64.9
Drivers, all trips	80.8
Drivers, trips over 4 mi	81.2
Drivers, trips over 8 mi	81.5

The disparity is actually greater than shown because an appreciable proportion of young drivers have not yet completed their education.

TABLE 3 Driver Median Age—Tails of the Distribution *(6,7)*

Age	Male U.S. Population (%)	Male Miles Driven	Female U.S. Population (%)	Female Miles Driven	All U.S. Population (%)	All Miles Driven
21 and younger	16.3	10.4	14.9	13.9	15.8	11.6
65 and older	12.5	10.3	16.6	4.1	14.7	8.3

Tables 4 and 5 give a summary of navigator status by trip purpose and length. It can be seen that the availability of a potential navigator is highly dependent on trip purpose and that this proportion increases for longer trips. It is also worth noting that for all trip purposes, the availability of a potential navigator is considerably smaller for female drivers than for male drivers.

DISCUSSION

These data can be used to determine the potential applicability, as previously defined, of different configurations of remedial measures for the problem of navigational waste. These determinations involve the following steps:

1. Estimate the prerequisites, in terms of driver age, sex, education, and other demographic attributes for the usability of a proposed remedial measure.
2. Estimate the proportion of all trips, stratified by trip length and trip purpose, for which the remedial measure would be applicable.
3. Estimate whether these factors would be affected by the presence of a potential navigator.

TABLE 4 Percent of Total Mileage by Navigator Status and Trip Purpose—All Trips *(6)*

		Trip Classification								
Navigator Status	Driver Sex	A	B	C	D	E	F	All	5 Mi	9 Mi
1. Alone	Male	96.1	55.7	76.8	48.0	58.4	51.5	74.7	74.1	72.9
	Female	94.1	60.5	74.5	57.2	61.4	47.0	70.6	70.3	69.4
	Overall avg	95.6	57.8	75.9	50.7	59.5	49.4	73.4	73.0	71.9
2. Child only	Male	0.4	4.0	2.1	3.1	2.7	5.6	1.9	1.8	1.7
	Female	4.1	23.3	13.3	21.9	21.9	34.3	17.1	16.6	16.1
	Overall avg	1.3	12.5	6.9	8.7	9.6	18.9	6.8	6.2	5.8
3. 1 + 2	Male	96.5	59.7	79.0	51.1	61.1	57.1	76.6	75.9	74.6
	Female	98.2	83.9	87.7	79.1	83.4	81.3	87.7	86.8	85.5
	Overall avg	96.9	70.3	82.8	59.4	69.1	68.3	80.2	79.2	77.7
4. Navigator available	Male	3.5	40.3	21.0	48.9	38.9	42.9	23.4	24.1	25.4
	Female	1.8	16.2	12.3	20.8	16.6	18.7	12.2	13.2	14.5
	Overall avg	3.1	29.7	17.2	40.5	30.9	31.7	19.8	20.8	22.3

TABLE 5 Percent of Total Mileage by Navigator Status and Trip Length *(6)*

Navigator Status	Minimum Trip Length (mi)							
	0	5	11	15	21	31	41	51
Male	23.4	24.5	26.4	27.8	32.7	40.1	45.7	50.4
Female	12.2	13.0	15.4	16.3	19.8	26.5	29.4	32.0

4. Compute the proportion of all U.S. highway travel characterized by these combinations of driver and trip characteristics.
5. Estimate the proportion of navigational waste within each of these classes that is likely to be eliminated by application of the proposed remedial measure. These estimates are based on empirical data on the distribution by type and consequence of navigational errors and on data concerning route familiarity within trip classes.
6. Compute the amount of navigational waste that will be eliminated by application of the proposed remedial measure.

The results of a number of hypothetical examples of this process follow with an outline of assumptions.

1. Highway Information System--Improvements in the highway information system such as elimination of ambiguous or confusing messages, location information, and advance warning of decision points will affect navigational waste that results from route-following difficulties. Assumptions include (a) No work trips, (b) Applicability ranges from 50 to 90 percent as function of trip length, and (c) No pertinent driver characteristics.
2. Highway Maps--Improvements in highway maps including clarity, inclusiveness, legibility, availability, and correspondence to ground truth will affect navigational waste resulting from both trip planning and route-following deficiencies. Such improvements may affect navigational waste as a result of trip planning deficiencies. Assumptions include (a) Applicability ranges from 25 to 90 percent as function of trip length, and (b) Usability ranges from 10 to 90 percent as joint function of education and sex.
3. Navigation Systems--Computer-controlled, real time navigation and guidance systems will, theoretically, alleviate the need for trip planning and eliminate the possibility of route-following errors. It can be estimated that real time navigation systems will, 5 years after implementation, lead to a reduction of 17 percent in navigational waste. This proportion will increase rapidly as the number of adequately equipped vehicles increases. Assumptions include (a) Entire arterial system would be instrumented; (b) System would be installed in 50 percent of all new vehicles--there would be no after-installation; (c) Equipped vehicles would accrue 120 percent of average mileage; and (d) Computer routings would be accepted and followed in 85 percent of all cases.

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Abridgment

Traffic Characteristics During Signal Change Intervals

ROBERT H. WORTMAN, JAMES M. WITKOWSKI, and THOMAS C. FOX

ABSTRACT

Driver and traffic characteristics associated with change intervals were studied at five intersections in the Tucson metropolitan area. The field studies were conducted to document the influence of (a) the variation in the duration of the yellow interval, (b) the effect of enforcement, and (c) intersection approach grades. Before and after studies were used at two of the intersections to evaluate the influence of extending the duration of the yellow interval. Generally, the major difference that resulted from a 4-sec yellow interval rather than 3 sec was the reduction in the percentage of the last vehicles through the intersection that entered on the red signal indication. The effect of enforcement was tested with a police vehicle located at the intersection approach. Although there was a reduction in the percentage of vehicles that entered the intersection on the red signal interval, other measures of driver and traffic characteristics generally showed no significant difference.

In 1981, a study entitled An Evaluation of Driver Behavior at Signalized Intersections was undertaken by the University of Arizona and Arizona State University for the Arizona Department of Transportation. The results of that study were published in January 1983 by the Arizona Department of Transportation in a report entitled An Evaluation of Driver Behavior at Signalized Intersections. A summary of that study was published in a paper by Wortman and Matthias (1).

Based on this previous work, additional research was proposed that would be undertaken in two phases. Phase I involved additional field studies of traffic characteristics and driver behavior that involved conditions and situations not included in the earlier research. The intent of the field studies was to provide information on the influence of (a) the variation in the duration of the yellow interval, (b) the effect of enforcement, and (c) intersection approach grades. Phase II of the research will focus on the development of guidelines for signal change intervals on the basis of the current status of knowledge and available information. This paper summarizes the findings of the Phase I portion of the research project.

DATA COLLECTION

Time-lapse photography techniques were utilized to record the driver behavior and traffic characteristics at selected intersections. Vehicles were filmed before the onset of the yellow signal interval, during the change interval, and until the vehicle either cleared the intersection or stopped. The camera was located so that it was possible to record the intersection and the signal indication as well as the operation of the approaching vehicles within about 350 to 400 ft of the intersection. Given the onset of the yellow signal indication, the study focused on the last vehicle to pass through the intersection and the first vehicle to stop.

DESCRIPTION OF STUDY INTERSECTIONS

Four intersections in the Tucson area were included in the field studies. At each of the intersections, one approach was observed and the study approach was located on the first street listed for each of the intersections. The following list provides detailed information for each of the intersections:

1. First Avenue and Roger Road (North Approach)
   a. Change interval: base condition--3 sec of yellow plus 2 sec of all-red; extended Yellow Condition--4 sec of yellow plus 2 sec of all-red.
   b. Approach configuration: two through lanes with an exclusive left-turn lane.
   c. Left-turn signalization: left turns permitted on a permissive basis during the through movement.
2. Wilmot Road and Broadway Boulevard (North Approach)
   a. Change interval: base condition--3 sec of yellow plus 2 sec of all-red; extended yellow condition--4 sec of yellow plus 2 sec of all-red.
   b. Approach configuration: three through-lanes with exclusive right- and left-turn lanes.
   c. Left-turn signalization: exclusive leading left-turn phase and permissive left turns during the through movement.
3. Swan Road and River Road (North Approach)
   a. Change interval: 5 sec of yellow plus 2 sec of all-red.
   b. Approach configuration: two through lanes plus a left-turn lane.
   c. Left-turn signalization: left turns on a permissive basis during the through movement.
4. Oracle Road and River Road (North Approach)
   a. Change interval: 4.5 sec of yellow plus 2 sec of all-red.
   b. Approach configuration: three through-lanes plus a left-turn lane.
   c. Left-turn signalization: turns permitted during an exclusive leading turn phase.

The First Avenue site was used to test the influence of enforcement. This intersection was selected for this part of the study primarily because previous observations at this site from the earlier study (1) revealed a relatively high percentage of drivers who entered the intersection during the all-red portion of the change interval. For this part of the research effort, a before and after approach was utilized. Following completion of the initial data collection, observations were made with a police vehicle parked along the side of the intersection approach.

Before and after studies were also used to examine the influence of the extension of the yellow interval. These studies were undertaken at the First Avenue and Wilmot Road sites. At each of these locations, the existing change interval consisted of a 3-sec yellow interval plus a 2-sec all-red phase. Field data were collected for the base conditions, and then the yellow interval was extended to 4 sec. The all-red phase was not changed.

The Swan Road and Oracle Road intersections were utilized to provide data about driver behavior where significant downgrades were involved. The intersection approach on Swan Road was approximately 2.0 percent, and the Oracle Road approach was about 2.6 percent.

DATA REDUCTION

For each of the vehicles that were the first to stop after the beginning of the yellow interval, the following information was extracted from the film:

1. The distance from the intersection at the beginning of the yellow interval,
2. The location of the vehicle when the brakes were applied (as indicated by the brake lights),
3. The location of the vehicle when it stopped,
4. The time required for the vehicle to stop,
5. The response time (determined as the time between the beginning of the yellow interval and the application of the brakes), and
6. Type of vehicle if other than a passenger car or light truck.

In addition, the behavior of the last vehicle to pass through the intersection after the beginning of the yellow was determined by making the following observations:

1. The location of the vehicle at the beginning of the yellow interval,
2. The time elapsed from the onset of the yellow interval until the vehicle entered the intersection,
3. The type of vehicle if other than a passenger car or light truck, and
4. If the vehicle entered the intersection on the red signal indication.

RESULTS

The results that are reported generally represent the descriptive statistics that describe the observed traffic and driver characteristics for each aspect of the data collection. Some limited comparative analyses were undertaken with respect to the influence of enforcement and the extension of the yellow interval. Further analyses will be undertaken in Phase II of the project.

OBSERVED TRAFFIC AND DRIVER CHARACTERISTICS

For each of the intersection approaches and the various studies conducted at a particular approach, descriptive statistics were computed for

1. Approach speeds,
2. Distance from the intersection at the beginning of the yellow interval,
3. Response time,
4. Deceleration rate, and
5. Percent of vehicles entering on the red signal indication.

Table 1 contains a summary of the observed values for each of these parameters at the study sites.

An analysis of the influence of the distance from the intersection, approach speed, or deceleration rate on the response time supported the findings of earlier work. As was found in the earlier work by Wortman and Matthias (1), there is little relationship between the response time and these individual variables.

Although the percentage of vehicles entering the intersection during the red signal decreased when the police vehicle was present, the extension of the yellow interval was a more effective treatment. At both sites where the yellow interval was extended, the percentage of vehicles entering on the red indication was significantly reduced.

THE EFFECT OF ENFORCEMENT

The effect of enforcement was analyzed by comparing the traffic and driver characteristics at the First

TABLE 1 Observed Driver and Traffic Characteristics

	Last Vehicle Through the Intersection				First Vehicle to Stop			
Intersection Approach	Posted Speed Limit (mph)	Mean Approach Speed (mph)	Mean Distance at Beginning of Yellow (ft)	Percent Entering on Red	Mean Approach Speed (mph)	Mean Distance at Beginning of Yellow (ft)	Mean Response Time (sec)	Mean Deceleration Rate (ft/sec^2)
First Avenue								
Base condition	45	38.5	131	18.4	37.9	265	1.3	11.9
With police car	45	38.3	115	8.6	37.7	260	1.4	12.5
Before extension	45	39.8	136	15.6	38.2	247	1.2	12.9
After extension (dry)	45	40.3	118	1.5	42.1	238	1.1	12.1
After extension (wet)	45	40.3	119	0.0	35.9	242	1.3	11.0
Wilmot Road								
Before extension	40	35.1	106	9.8	35.8	240	1.4	13.2
After extension	40	32.1	101	3.0	32.9	223	1.3	12.0
Swan Road	45	46.3	205	3.6	47.5	309	1.0	8.3
Oracle Road	50	42.9	146	1.4	41.3	274	1.1	10.1

Avenue site by using the base condition and condition with the police vehicle at the site. There were no significant differences in the parameters except that the distance from the intersection at the onset of the yellow signal indication for the last vehicle through the intersection decreases when the police vehicle was at the site.

THE EFFECT OF EXTENDING THE YELLOW INTERVAL

At the two intersections where the yellow interval was extended, the data obtained from the before-and-after studies yielded mixed results. For the First Avenue site, the mean speed of the first vehicle to stop was significantly higher during the after condition. Also, the mean distance from the intersection at the onset of the yellow interval was significantly less for the last vehicles through the intersection. These significant differences could be the result of differences in the traffic stream rather than the effect of the extension of the yellow interval. At this site, all other differences were not significant. In contrast, the mean approach speeds, response time, and deceleration rate at the Wilmot Road site were significantly lower in the sample taken after the extension of the yellow interval.

As has been indicated previously, the extension of the yellow did reduce the percentage of vehicles entering on the red signal indication. These results tend to support the findings and conclusions of Stimpson, et al. (2). In their study, it was found that extensions of the yellow duration substantially reduced the frequency of potential intersection conflicts.

THE EFFECT OF WET PAVEMENT

Although the sample size for the wet conditions was quite small, a comparative analysis of the wet and dry conditions at the First Avenue site was made. For this analysis, only the observations made after the extension of the yellow were made. There were no significant differences except that the approach speeds of the first vehicles to stop were significantly lower.

CONCLUSIONS

Although further analysis of the data base will be undertaken in a later phase of the project, several conclusions can be drawn based on this part of the study. The conclusions can be summarized as follows:

1. The presence of the police vehicle at the study site generally did not affect the measured traffic characteristics and driver behavior. With the police vehicle at the site, the percentage of vehicles entering the intersection during the red signal indication was reduced. The extension of the duration of the yellow signal interval, however, was more effective in reducing this percentage.
2. An analysis of traffic characteristics and driver behavior at the two study sites revealed somewhat mixed results when the duration of the yellow interval was extended. At both sites, however, the percentage of vehicles entering during the red signal indication was reduced following the extension of the yellow interval.
3. Data collected at the sites with significant downgrades did indicate lower deceleration rates and response times when compared with the other sites.
4. Based on the limited observations at the one site, the wet condition did not have a significant effect except that the approach speed of the stopping vehicles was lower with the wet pavement.
5. No relationship was found between response time and the distance from the intersection at the onset of the yellow interval, approach speed, or deceleration rate. Further analysis will be required to determine if response time is a combined function of several of these variables.

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Accident Effects of Centerline Markings on Low-Volume Rural Roads

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ABSTRACT

Several accident comparisons were made for low-volume rural roads that were either unmarked, marked with a dashed centerline only, or marked with both a dashed centerline and no-passing zone stripes. These analyses made use of the Federal Highway Administration data base collected during the Pavement Marking Demonstration Program. The analyses indicated that the Pavement Marking Demonstration Program as a whole was not effective in reducing highway accidents. However, the analyses indicated that the safety effects that resulted from adding combined centerlines may be beneficial for pavement widths of 20 or more feet and traffic volumes of 500 or more vehicles per day.

In designing and operating highways, the highway agency is interested in providing maximum traffic safety and efficiency. Maximum safety requires wide roadways and shoulders, gentle alignment, clear roadsides, and high quality traffic control devices.

When considering low-volume rural roads, however, the highway agency is faced with an apparent dilemma. On the one hand, the agency would like to provide each individual motorist with the same degree of safety experienced on the modern Interstate system. On the other hand, the cost of providing this degree of safety often conflicts with the agency's philosophy of economic expediency. The way to solve this apparent dilemma is to gain knowledge of the safety effects of each highway design and traffic control element so the application of criteria can be established through the principles of cost-effectiveness.

The use of centerline and no-passing zone markings is one area where the cost-effectiveness is unclear. For example, the Manual on Uniform Traffic Control Devices (1) does not give a guideline on the minimum traffic volume level for the application of centerline markings. For no-passing zone markings, the manual mandates them on all highways where centerlines are used.

In a report published by the NCHRP, probability analyses and assumptions about accident reduction were used to conclude that centerline markings are not cost-effective below 300 vehicles per day (vpd) (2). What is needed is a more definitive empirical study that either substantiates or modifies these findings. The objective of this research was to collect and evaluate accident data for the purpose of verifying or modifying the warrants for centerline and no-passing zone markings suggested in the NCHRP Report.

EFFECTIVENESS OF ADDING CENTERLINE AND NO-PASSING ZONE MARKINGS TO UNMARKED HIGHWAYS

Of primary interest in this project was the determination of any accident benefits associated with the placement of centerline and no-passing zone markings on low-volume rural roads. A review of published literature revealed the lack of any descriptive data. The 1981 and 1982 editions of the Federal Highway Administration (FHWA) report on highway safety stewardship (3,4) did provide a general overview of the Pavement Marking Demonstration Program (PMDP) that was established by the Federal Highway Act of 1973. Table 1, which is extrapolated from these documents, contains data on the net accident effect on the application of centerline and no-passing zone markings on previously unmarked highways. As can be seen from this table, the general accountability of the PMDP is a significant increase in injury accidents, a significant decrease in property-damage only accidents (PDO), and no significant change in total accidents.

TABLE 1 Accident Reduction Effectiveness for the PMDP Application of Centerline and No-Passing Zone Markings *(3,4)*

				Reduction in Accidents[a] (%)			
Year	No. of States	No. of Miles	Total Cost (million $)	Fatal	Injury	Property Damage Only	Total
1980	14	11,475	4.416	-8	-8[b]	2[b]	-1
1981	15	12,673	5.039	-3	-6[b]	4[b]	1

[a]Minus sign denotes increase.
[b]Significant change at 95 percent level of confidence.

In an attempt to find more descriptive data regarding the evaluation of pavement marking effectiveness, several unpublished documents were found. Most helpful of these was a Federal Highway Administration (FHWA) report by Lee (5), which contained an evaluation of 225 pavement marking projects in six states. A brief summary of this evaluation is given in Table 2. Although the statistical significance of these evaluations was not given, all pavement marking categories showed an increase in accident rate. On request, the FHWA supplied the original data base for the Lee report. In addition to before and after accident data for each project, the data base also included highway information on project length, before and after study periods, average daily traffic (ADT), lane width, shoulder width, terrain, and speed limit.

Additional data was obtained from several states. However, some of these data sets were not descrip-

TABLE 2 Summary of PMDP Effectiveness Report by Lee (5)

Improvement Type	No. of Sites	No. of Miles	Accident Rate (A/MVM)[a] Before	After	Change (%)
Centerline striping	48	382	4.18	4.28	+2
Edgeline striping added	94	721	2.88	2.93	+2
Centerline and edgeline	83	943	1.97	2.55	+29
Total	225	2,046	2.64	2.99	+13

[a]A/MVM = accidents per million vehicle miles.

tive enough for additional analysis, and other data sets were from states already included in the FHWA data base. As a result, only data sets from Ohio and Missouri were used in additional analyses. The data from Missouri provide the only available analysis of the effectiveness of adding dashed centerlines only to unmarked highways and the effectiveness of adding no-passing zone markings to highways marked with either dashed centerlines only or with dashed centerlines and edgelines.

Table 3 contains results of a general analysis of the effectiveness of adding centerline and no-passing zone markings to previously unmarked highways. This table includes the five states with this kind of project in the FHWA data base as well as Ohio. This table shows somewhat mixed results. The data for indicated significant increase in accidents and the data for Ohio indicated a significant decrease in accidents, while the other four states showed nonsignificant differences. These six states may, however, have different reporting levels, ADT distributions, and road design characteristics in their samples of projects.

The significance test used in Table 3 and all subsequent tables is a one-sample t-test using a normal approximation to a binomial distribution. In essence, it tests whether the proportion of before or after accidents to total accidents is significantly different than the proportion of before of after vehicle-miles to total vehicle-miles. The statistic is as follows:

$$t = A_B M_A - A_A M_B / [(A_B + A_A) M_A M_B]^{1/2} \quad (1)$$

where

A_B = number of accidents in before period,

TABLE 3 Summary of Before and After Accident Statistics for Projects Where Centerline and No-Passing Markings Were Added to Previously Unmarked Highways

State	No. of Sites	No. of Miles	Before Period: Vehicle Miles (millions)	Before Period: No. of Accidents	Before Period: Accident Rate	After Period: Vehicle Miles (millions)	After Period: No. of Accidents	After Period: Accident Rate	Significant Difference[a]
FHWA Data Base									
Missouri	4	27.80	13.140	45	3.42	11.308	33	2.92	N.S.
Montana	2	20.30	3.482	10	2.87	2.449	24	9.80	S(+)
North Carolina	13	96.90	29.775	127	4.27	30.457	150	4.92	N.S.
Virginia	22	168.11	30.628	151	4.93	33.180	168	5.06	N.S.
West Virginia	7	68.70	36.321	144	3.96	39.233	141	3.59	N.S.
Total	48	381.81	113.346	477	4.21	116.627	516	4.42	N.S.
Ohio Data Base									
Ohio	N/A	468.24	92.870	153	1.65	94.360	106	1.12	S(-)
Grand total	N/A	850.05	206.216	630	3.06	210.987	622	2.95	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 4 Summary of Before-After Injury Plus Fatal Accident Statistics for Projects Where Centerline and No-Passing Markings Were Added

State	No. of Sites	No. of Miles	Before Period: Vehicle Miles (millions)	Before Period: No. of Fatal and Injury Accidents	Before Period: Fatal and Injury Accident Rate	After Period: Vehicle Miles (millions)	After Period: No. of Fatal and Injury Accidents	After Period: Fatal and Injury Accident Rate	Significant Difference[a]
FHWA Data Base									
Missouri	4	27.80	13.140	20	1.52	11.308	15	1.33	N.S.
Montana	2	20.30	3.482	8	2.30	2.449	11	4.49	N.S.
North Carolina	13	96.90	29.775	48	1.61	30.457	52	1.71	N.S.
Virginia	22	168.11	30.628	52	1.70	33.180	66	1.99	N.S.
West Virginia	7	68.70	36.321	42	1.16	39.233	47	1.20	N.S.
Total	48	381.81	113.346	170	1.50	116.627	191	1.65	N.S.
Ohio Data Base									
Ohio	N/A	468.24	92.870	59	0.64	94.360	42	0.45	S(-)
Grand total	N/A	850.05	206.216	229	1.11	210.987	233	1.10	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tailed t-test.

A_A = number of accidents in after period,
M_B = number of vehicle-miles in before period, and
M_A = number of vehicle-miles in after period.

In Table 4, the same data as in Table 3 are used to show before-after comparisons for fatal plus injury accidents. In this case, Ohio is the only state that shows a significant change, a decrease in severe accidents.

In an attempt to understand some of the variances shown in Tables 1 through 4, several analyses were conducted on the FHWA data base, where information was available on highway characteristics for each project. These analyses showed state, ADT, and lane width to be the only interesting stratification variables. Also, total accident comparisons and fatal plus injury accident comparisons showed similar results, so only the total accident comparisons are shown.

The data in Table 5 give a summary of the FHWA data base stratified by ADT. In this table, the data for Montana indicate a significant increase in accident rate for ADTs of 0-500 vpd, and the data for North Carolina indicate a significant increase in accident rate for ADTs of 501-1,000 vpd. The totals for each ADT category are nonsignificant but show a trend toward accident benefits with higher ADTs.

The data in Table 6 give a summary of the FHWA data base stratified by lane width. In this table, Montana shows a significant increase in accident rate for lane widths of between 10 and 11 ft. All other comparisons indicate no significant differences.

In an attempt to find a more discerning relationship for the accident effectiveness of centerline

TABLE 5 Summary of Before-After Accident Statistics Stratified by ADT for 48 Projects Where Centerline and No-Passing Zone Markings Were Added (FHWA data base)

		Before Period			After Period			
State	No. of Projects	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
ADT = 0-500 VPD								
Missouri	1	1.205	9	7.47	1.009	3	2.97	N.S.
Montana	2	3.482	10	2.87	2.449	24	9.80	S(+)
North Carolina	9	13.347	53	3.97	13.661	50	3.66	N.S.
Virginia	15	15.831	73	4.61	16.400	88	5.37	N.S.
Total	27	33.865	145	4.28	33.519	165	4.92	N.S.
ADT = 501-1,000 VPD								
Missouri	1	1.900	15	7.89	1.900	10	5.26	N.S.
North Carolina	4	16.428	74	4.50	16.796	100	5.95	S(+)
Virginia	4	5.981	31	5.18	6.402	31	4.84	N.S.
West Virginia	3	4.031	14	3.47	4.059	23	5.67	N.S.
Total	12	28.340	134	4.73	29.157	164	5.62	N.S.
ADT = >1,000 VPD								
Missouri	2	10.035	21	2.09	8.399	20	2.38	N.S.
Virginia	3	8.816	47	5.33	10.378	49	4.72	N.S.
West Virginia	4	32.290	130	4.03	35.174	118	3.35	N.S.
Total	9	51.141	198	3.87	53.951	187	3.47	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tail t-test.

TABLE 6 Summary of Before-After Accident Statistics by Lane Width for 48 Projects Where Centerline and No-Passing Markings were Added (FHWA data base)

		Before Period			After Period			
State	No. of Projects	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
Lane Width of 8-9 ft								
North Carolina	11	23.436	87	3.71	23.983	110	4.59	N.S.
Virginia	19	24.776	114	4.60	26.360	128	4.86	N.S.
West Virginia	7	36.321	144	3.96	39.233	141	3.59	N.S.
Total	37	84.533	345	4.08	89.576	379	4.23	N.S.
Lane Width of 10-11 ft								
Missouri	4	13.140	45	3.42	11.308	33	2.92	N.S.
Montana	2	3.482	10	2.87	2.449	24	9.80	S(+)
North Carolina	2	6.339	40	6.31	6.474	40	6.18	N.S.
Virginia	3	5.852	37	6.32	6.820	40	5.87	N.S.
Total	11	28.813	132	4.58	27.051	137	5.06	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tail t-test.

and no-passing zone markings, the FHWA data were stratified by both ADT and lane width. For this purpose, two separate analyses were undertaken. The first analysis used the 48 before-after sites shown in previous tables. The second analysis, which is not entirely a before-after comparison, used portions of data from all 225 projects in the FHWA data base where either no markings were present or centerline and no-passing zone markings were present.

The data in Tables 7 and 8 give the 48 before-after projects stratified by ADT and lane width. These tables show significant increases in accident rate for highways with up to 500 vpd, a lane width of between 10 and 11 ft, and for highways with between 501 and 1,000 vpd and a lane width of between 8 and 9 ft. Although all other categories are nonsignificant, there does appear to be a decided trend toward accident rate decrease with higher ADTs and wider lanes.

The data in Tables 9 and 10 give a comparison from the FHWA data base of all sites with no markings in the before period to all sites with centerline and no-passing zone markings in either the before or after period. These tables also show significant increases in accident rate for highways with up to 500 vpd and a lane width of between 10 and 11 ft, and for highways with between 501 and 1,000 vpd and a lane width of between 8 and 9 ft.

TABLE 7 Comparison of Before-After Accident Statistics by ADT for Sites with 10-11 ft Lanes Where Centerline and No-Passing Zone Markings were Added (FHWA data base)

		Before Period			After Period			
ADT by State	No. of Sites	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
ADT = 0-500 VPD								
Missouri	1	1.205	9	7.47	1.009	3	2.97	N.S.
Montana	2	3.482	10	2.87	2.449	24	9.80	S(+)
Virginia	2	2.056	17	8.27	2.180	22	10.09	N.S.
Total	5	6.743	36	5.34	5.638	49	8.69	S(+)
ADT = 501-1,000 VPD								
North Carolina	2	6.339	40	6.31	6.474	40	6.18	N.S.
Missouri	1	1.900	15	7.89	1.900	10	5.26	N.S.
Total	3	8.239	55	6.68	8.374	50	5.97	N.S.
ADT = >1,000 VPD								
Missouri	2	10.035	21	2.09	8.399	20	2.38	N.S.
Virginia	1	3.796	20	5.27	4.640	18	3.88	N.S.
Total	3	13.831	41	2.96	13.039	38	2.91	N.S.
Grand total	11	28.813	132	4.58	27.051	137	5.06	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 8 Comparison of Before-After Accident Statistics by ADT for Sites with 8-9 ft Lanes Where Centerline and No-Passing Zone Marking were Added (FHWA data base)

		Before Period			After Period			
State	No. of Sites	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
ADT = 0-500 VPD								
North Carolina	9	13.347	53	3.97	13.661	50	3.66	N.S.
Virginia	13	13.775	56	4.07	14.220	66	4.64	N.S.
Total	22	27.122	109	4.02	27.881	116	4.16	N.S.
ADT = 501-1,000 VPD								
North Carolina	2	10.089	34	3.37	10.322	60	5.81	S(+)
Virginia	4	5.981	31	5.18	6.402	31	4.84	N.S.
West Virginia	3	4.031	14	3.47	4.059	23	5.67	N.S.
Total	9	20.101	79	3.93	20.783	114	5.49	S(+)
ADT = >1,000 VPD								
Virginia	2	5.020	27	5.38	5.738	31	5.40	N.S.
West Virginia	4	32.290	130	4.03	35.174	118	3.35	N.S.
Total	6	37.310	157	4.21	40.912	149	3.64	N.S.
Grand total	37	84.533	345	4.08	89.576	379	4.23	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 11 Summary of Before-After Accident Statistics for Projects in Missouri Where Dashed Centerlines Only Were Added

	Before Period			After Period			
Accident Parameters	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
Total accidents	12.426	32	2.58	12.752	40	3.14	No
Injury and fatal accidents	12.426	14	1.13	12.752	19	1.49	No

Note: The number of projects was 9, the number of miles was 58.45, the ADT range was 88-512 vpd, and the lane widths were 10-11 ft.
[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 12 Summary of Before-After Statistics by ADT for 20 Projects in Missouri Where No-Passing Zone Markings Were Added to Existing Centerline Markings (10-11 ft lanes)

			Before Period			After Period			
ADT (vpd)	No. of Projects	No. of Miles	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
501-1,000	7	66.10	44.263	103	2.33	45.949	80	1.74	S(–)
>1,000	13	130.59	139.011	281	2.02	141.279	220	1.56	S(–)
Total	20	196.69	183.374	384	2.09	187.228	300	1.60	S(–)

[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 13 Summary of Before-After Statistics by ADT for 33 Projects in Missouri Where No-Passing Zone Markings Were Added to Existing Centerline and Edgeline Markings

			Before Period			After Period			
ADT (vpd)	No. of Projects	No. of Miles	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
0-500	4	35.59	11.740	36	3.07	14.364	36	2.51	None
501-1,000	17	355.01	192.841	496	2.57	202.557	503	2.48	None
>1,000	12	241.61	244.649	689	2.82	240.415	708	2.94	None
Total	33	632.21	449.230	1,221	2.72	457.336	1,247	2.73	None

[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 14 Change in Accident Rates Associated with Center Markings *(3,4)*

	Lane Widths					
ADT (vpd)	8-9 ft	Change (%)	10-11 ft	Change (%)	All Lane Widths	Change (%)
0-500	FHWA Data Base 1[a]	+3	FHWA Data Base 1	+63[c]	FHWA Data Base 1	+15
	FHWA Data Base 2[b]	+15	FHWA Data Base 2	+55[c]	FHWA Data Base 2	+15[c]
	Missouri (dashed centerline only added)	+32				
500-1,000	FHWA Data Base 1	+40[c]	FHWA Data Base 1	–11	FHWA Data Base 1	+19
	FHWA Data Base 2	+47[c]	FHWA Data Base 2	+7	FHWA Data Base 2	+29[c]
			Missouri (no-passing stripe added)	–25[c]		
Greater than 1,000	FHWA Data Base 1	–14	FHWA Data Base 1	–2	FHWA Data Base 1	–10
	FHWA Data Base 2	–5	FHWA Data Base 2	–16	FHWA Data Base 2	–9
			Missouri (no-passing stripe added)	–23[c]		
All ADTs	FHWA Data Base 1	+4	FHWA Data Base 1	+10	1981 FHWA Stewardship Report	+1
	FHWA Data Base 2	+15[c]	FHWA Data Base 2	+1	1982 FHWA Stewardship Report	–1
					FHWA Data Base 1	+5
					FHWA Data Base 2	+13[c]
					Ohio Data Base	–35[c]

[a]FHWA Data Base 1 includes 48 before-after site comparisons between no markings and combined centerline and no-passing zone markings (5 states).
[b]FHWA Data Base 2 includes 87 sites with no markings and 111 sites with both centerline and no-passing zone markings (5 states).
[c]Statistically significant change at 90 percent confidence level.

TABLE 9 Comparison of Accident Rates for Sites with no Markings to Sites with Centerline and No-Passing Zone Markings (FHWA total data base)

	No Lines				Centerline and No Passing Zone				
State	No. of Sites	Vehicle Miles (millions)	No. of Accidents	Accident Rate	No. of Sites	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
ADT = 0-500 VPD/lane width = 8-9 ft									
North Carolina	21	32.758	117	3.57	59	82.133	344	4.19	N.S.
Virginia	13	13.775	56	4.07	13	14.220	66	4.64	N.S.
Total	34	46.533	173	3.72	72	96.353	410	4.26	N.S.
ADT = 501-1,000 VPD/lane width = 8-9 ft									
North Carolina	3	14.930	51	3.42	4	16.405	84	5.12	S(+)
Virginia	4	5.981	31	5.18	5	9.177	44	4.79	N.S.
West Virginia	5	9.122	22	2.41	3	4.059	23	5.67	S(+)
Total	12	30.033	104	3.46	12	29.641	151	5.09	S(+)
ADT = > 1,000 VPD/lane width = 8-9 ft									
Virginia	2	5.020	27	5.38	2	5.738	31	5.40	N.S.
West Virginia	6	42.175	154	3.65	4	35.174	118	3.35	N.S.
Total	8	47.195	181	3.84	6	40.912	149	3.64	N.S.
Grand total	54	123.761	458	3.70	90	166.906	710	4.25	S(+)

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tailed t-test.

TABLE 10 Comparison of Accident Rates for Sites with no Markings to Sites with Centerline and No-Passing Zone Markings (FHWA data base)

	No Lines				Centerline and No Passing Zone				
State	No. of Sites	Vehicle Miles (millions)	No. of Accidents	Accident Rate	No. of Sites	Vehicle Miles (millions)	No. of Accidents	Accident Rate	Significant Difference[a]
ADT: 0-500 VPD/lane width = 10-11 ft									
Missouri	8	11.304	46	4.40	2	4.186	13	3.11	N.S.
Montana	3	4.963	16	3.22	2	2.449	24	9.80	S(+)
Virginia	2	2.056	17	8.27	2	2.180	22	10.09	N.S.
Total	13	18.323	79	4.31	6	8.815	59	6.69	S(+)
ADT: 501-1,000 VPD/lane width = 10-11 ft									
Missouri	11	31.129	104	3.34	2	4.180	15	3.59	N.S.
North Carolina	2	6.339	40	6.31	4	14.526	62	4.27	S(-)
Total	13	37.468	144	3.84	6	18.706	77	4.12	N.S.
ADT = > 1,000 VPD/lane width = 10-11 ft									
Missouri	6	43.703	116	2.65	2	8.399	20	2.38	N.S.
Virginia	1	3.796	20	5.27	7	27.751	67	2.41	S(-)
Total	7	47.499	136	2.86	9	36.150	87	2.41	N.S.
Grand total	33	103.290	359	3.48	21	63.671	223	3.50	N.S.

Note: N.S. = nonsignificant.

[a]Significance at 90 percent confidence level using two-tailed t-test.

These tables also show a trend toward rate reduction with higher ADTs.

EFFECTIVENESS OF ADDING ONLY DASHED CENTERLINE MARKINGS TO UNMARKED HIGHWAYS

The study indicated that Missouri was the only state that made extensive use of a dashed centerline without a nonpassing stripe. The practice is to use this treatment on unnumbered state highways with less than 1,000 vpd. These highways are basically local rural access roads. The data in Table 11 give an evaluation of nine projects where dashed centerlines only were added to previously unmarked highways with between 88 and 512 vpd and a lane width of between 10 and 11 ft. This analysis shows a 22 percent nonsignificant increase in accident rates.

EFFECTIVENESS OF ADDING NO-PASSING ZONE MARKINGS

The data further indicated that Missouri also provided for evaluating the effectiveness of adding no-passing stripes to highways previously marked with dashed centerlines only. The data in Table 12 give an evaluation of 20 projects where no-passing zone markings were added to highways with lane widths of between 10 and 11 ft previously marked with dashed centerline only. This evaluation shows significant decreases in accident rates for both ADT levels of between 501 and 1,000 vpd and greater than 1,000 vpd.

The data in Table 13 give an evaluation of 33 projects where no-passing zone markings were added to highways previously marked with dashed centerline only and edgelines. This evaluation shows no significant differences in accident rates for any ADT category.

SUMMARY OF ACCIDENT COMPARISONS

Several sources of data were gathered to analyze the potential safety effectiveness of the application of centerline and no-passing zone markings. Most of the data represented before and after comparisons made as a part of the PMDP. The data in Table 14 give a summary of the sources, accident reduction effectiveness, and statistical significance of comparisons. The results of Table 14 are somewhat conflicting but do indicate that:

1. Widespread application of center pavement markings to all paved roads with no existing markings is not likely to produce accident reduction benefits;
2. Center markings applied to roads with 500 or less vpd appear to produce increased accident rates;
3. Center markings applied to roads with less than 10-ft lane widths and fewer than 1,000 vpd appear to produce increased accident rates; and
4. Accident reduction benefits may be generally associated with wider roads and higher ADTs.

CONCLUSIONS

The nationwide application of center markings on previously unmarked two-lane rural roads under the federal PMDP does not appear to have produced any reduction in accident rates. In fact, the before-after results for hundreds of center marking projects in 15 states indicate a significant increase in injury accident rates (3,4).

Despite this seemingly negative result, a more detailed analysis of available data indicates potential accident benefits for wider roads that carry higher traffic volumes. This result was evident both for adding centerline and no-passing zone markings to previously unmarked roads and for adding no-passing zone marking to roads previously marked with only a dashed centerline.

Although the data were not sufficient for determining specific road width and ADT warrants based on a precise breakpoint of cost-effectiveness, they do seem to indicate lower boundaries for these warrants based on omitting center markings where they appear to produce significant increases in accident rates. By using this basis, the following tentative warrants seem reasonable for the application of both dashed centerline and no-passing zone markings:

Road Width (ft)	Minimum ADT (vpd)
Less than 16	Not Applicable
16-18	1,000
20 or greater	500

This concept of road width and minimum ADT warrants is generally consistent with current state department of transportation practice in those states that have high portions of low-volume road mileage (6). The practice of not marking low-volume rural roads is also prevalent among local rural jurisdictions. Perhaps the lack of markings on these roads, which tend to have lower design standards, provides the driver a greater ability to distinguish the need for a more cautious driving behavior than is required on higher-volume roads with better design standards.

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Abridgment

Evaluation of Freeway Crash Cushion Delineation Treatments

ROBERT C. WUNDERLICH

ABSTRACT

Presented in this paper are the results of a study conducted to evaluate the effectiveness of improved delineation techniques in reducing accident frequency at freeway gore areas with crash cushions. Four delineation treatments were developed. Three treatments included only reflective static elements whereas the fourth consisted of both static elements and flashing lights. Each treatment was installed at two sites and two additional sites where no changes were made were used as controls. The repair history of each site was used as an indicator of the accident frequency. Treatment effectiveness was assessed by comparing the repair rate before treatment installation to the repair rate after treatment installation. The analysis revealed that the treatment that combined static elements with flashing lights did reduce the frequency of repairs at sites with high initial repair rates (9 to 12 per year). In general, treatments with only static elements had little effect on the repair rate at sites with moderate initial repair rates (3 to 6 repairs per year). However, one static treatment was more effective than the group when the group was considered as a whole.

The severity of accidents at highway gore areas has been greatly reduced by the use of crash cushions although their use does not reduce accident frequency (1). Even though the potential for serious injury has been lessened by their use, collisions with crash cushions can still result in personal injury and property damage. Another consequence of these accidents is the cost of the repair work required to restore a damaged cushion's effectiveness. Attempts to recover the repair costs are made by billing the person who damaged the cushion; however, in many cases, the costs cannot be recovered and are consequently absorbed by the highway department that is responsible for crash cushion maintenance.

Another important aspect of the repair work is the hazard presented to the motoring public and work crews while the damaged crash cushion is being repaired. Gore areas are often located at the interchange of two high-speed, high-volume roadways and, during a crash cushion repair job, special traffic control procedures are required, such as closing off the traffic lanes adjacent to the gore and using flashing arrowboards and flagmen to aid in traffic handling. Because of the complex nature of the situation, both the crew and the motoring public are exposed to greater hazard than if the repair was not necessary. For these reasons, District 12 of the Texas State Department of Highways and Public Transportation (SDHPT) enlisted the aid of the Texas Transportation Institute to develop and test accident countermeasures for freeway gore areas with crash cushions.

STUDY PURPOSE

The accident countermeasure approach that was chosen for investigation was improved delineation. The delineation treatments tested were intended to increase the conspicuity, or visibility, of the gore area and crash cushion. It was thought that improved delineation would tend to counteract the influence of other factors, such as roadway geometrics and driver inattentiveness, which contribute to crash cushion accidents.

The primary purpose of this study was to develop and implement treatments that would reduce accidents at sites with recurring accidents. The evaluation was performed to see if any of the delineation treatments could decrease accidents and, if so, what level of delineation was required to effect such a reduction.

STUDY APPROACH

Several delineation treatments were developed, implemented, and evaluated in this study. Treatment descriptions, study site locations, and the before-after evaluation method are included in the following sections.

Delineation Treatments

Four delineation treatment levels were developed. Figures 1 through 4 illustrate the four treatment levels and demonstrate how delineation elements were added to form each successive level. The elements of each treatment are listed in Table 1.

The base treatment, Level 1 (Figure 1), was made up of three elements that were common to all the treatment levels: (a) yellow and black reflectorized nose panel, (b) yellow painted barrels with reflectorized stripe, and (c) raised reflective pavement markers. The other three levels were formed by adding delineation elements to the basic configuration. As can be seen from Table 1 and Figures 1-4, Treatment Level 2 was formed by adding a yellow and black reflectorized back panel to Level 1. Level 3 consisted of the four elements of Level 2 and reflectorized chevron alignment signs. Level 4 included

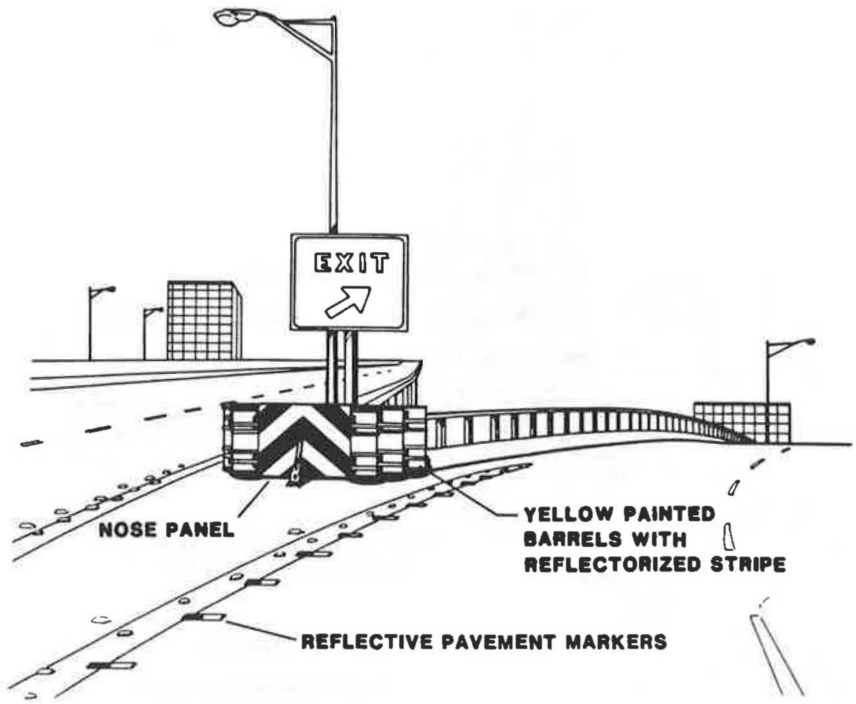

FIGURE 1 Treatment Level 1.

all the elements of Level 3 plus amber flashing lights. It is important to note the distinction between treatment Level 4 and all the other levels. Level 4 is the only level that included a dynamic delineation element, the flashing lights. All the other treatments included only static reflective elements.

Study Sites and Treatment Implementation

Ten of the most frequently repaired freeway crash cushion sites in Houston, Texas, were selected for study. Table 2 lists the average repairs per year for the 3-yr period before treatment implementation for each site. Each of the four treatments was in-

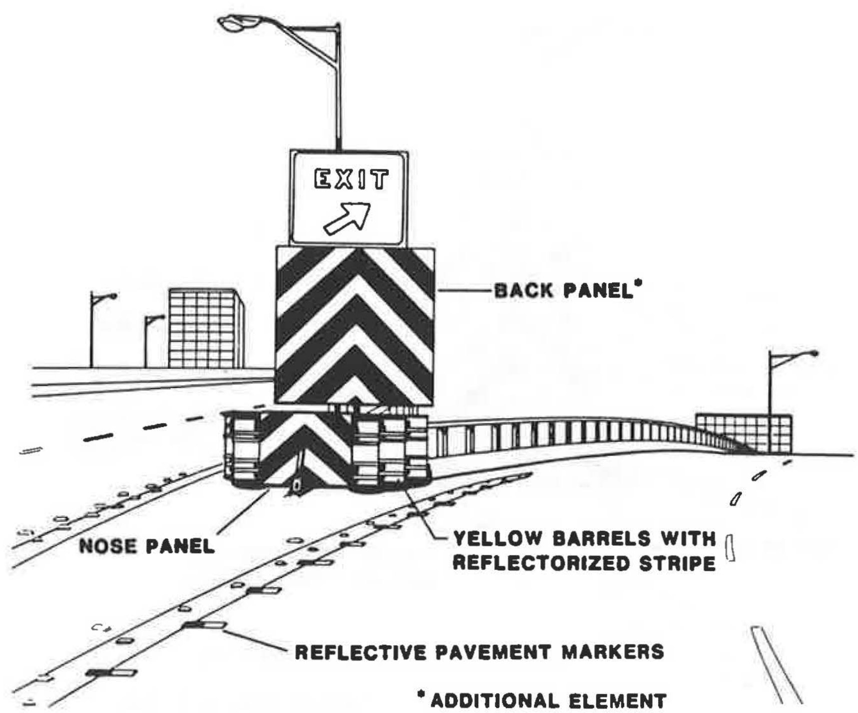

FIGURE 2 Treatment Level 2.

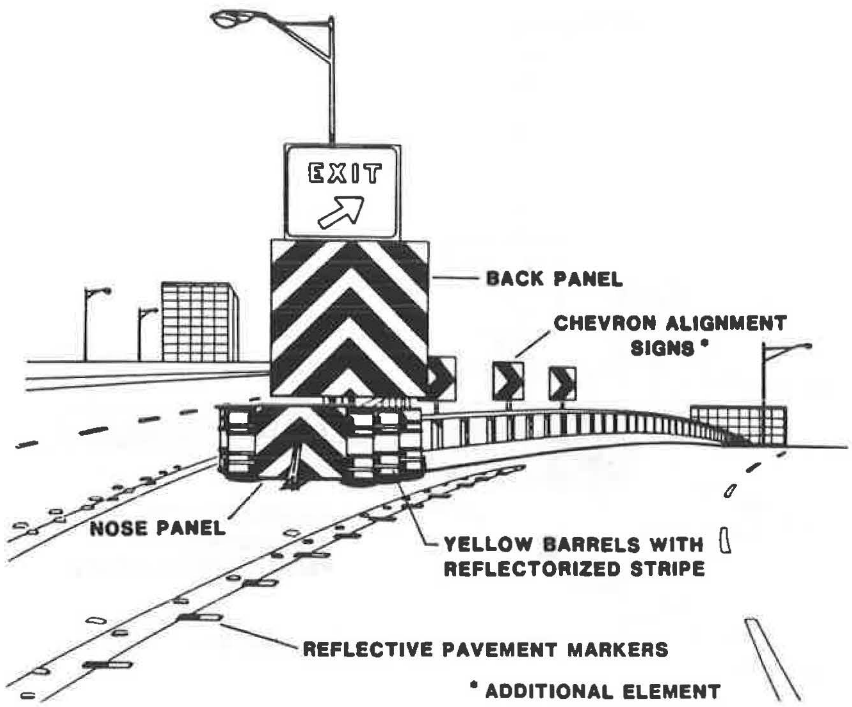

FIGURE 3 Treatment Level 3.

stalled at two sites. No changes were made at Sites 1 and 35 and these sites were designated as controls. The assignment of treatments to sites was based on 3-yr repair histories. The rationale for treatment assignment was based on matching the degree of treatment of the frequency of repairs; therefore, treatment Level 4, which included flashing lights, was installed at the two sites with high average repair rates (9 to 12 repairs per year). The remaining sites all had moderate repair rates (3 to 6 repairs per year). The lowest treatment level, 1, was installed at two sites that had rates at the low end of the moderate repair rate range. The two remaining treatments, Levels 3 and 4, were assigned to four sites with moderate rates. The two sites where treatments were not installed also had moderate rates. Thus, two distinct groups of sites (those with moderate repair rates and those with high re-

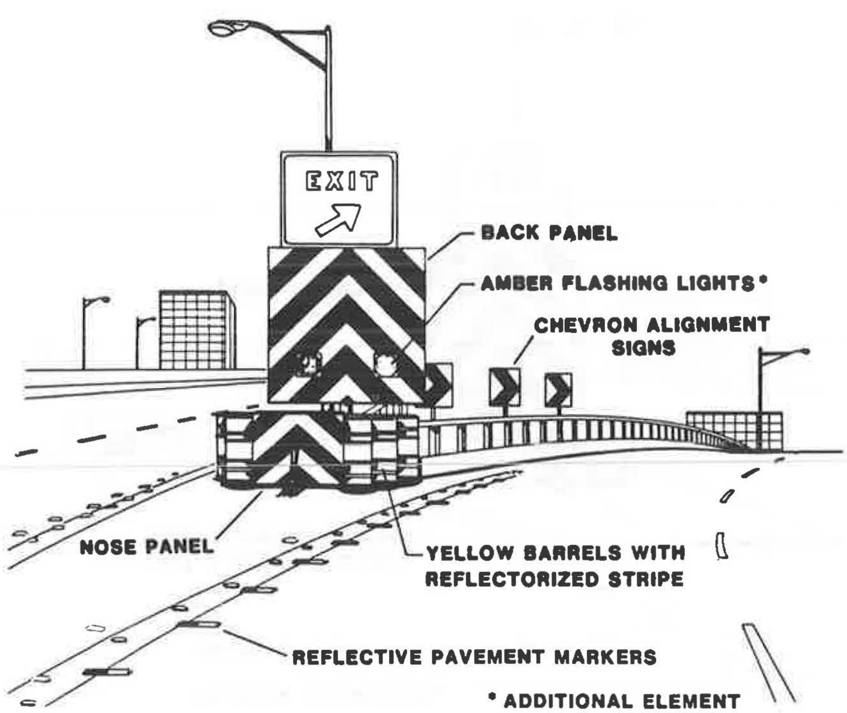

FIGURE 4 Treatment Level 4.

TABLE 1 Delineation Elements Included in Each Treatment Level

	Basic Delineation Elements			Supplemental Delineation Elements		
Treatment Level	Nose Panel[a]	Painted Barrels with Reflectorized Stripe[b]	Raised Reflective Pavement Markers	Back Panel[a]	Chevron Alignment Signs[c]	Flashing Lights[d]
1	X	X	X			
2	X	X	X	X		
3	X	X	X	X	X	
4	X	X	X	X	X	X

[a]Yellow and black alternating stripes (reflectorized).
[b]Yellow barrels and reflectorized stripe.
[c]MUTCD Sign No. W1-8 (reflectorized).
[d]Amber lenses.

TABLE 2 Study Site Summary

Site	Avg Repairs/ Year Before Treatment Installation[a]	Treatment	Site	Avg Repairs/ Year Before Treatment Installation[a]	Treatment
1	3.7	None	6	4.7	Level 2
2	3.7	None	7	4.7	Level 3
3	3.0	Level 1	8	6.0	Level 3
4	3.7	Level 1	9	12.0	Level 4
5	5.7	Level 2	10	9.0	Level 4

[a]Based on 3 yr of repair data.

pair rates) could be identified, and distinctly different delineation approaches--static reflective treatments versus static reflective with flashing light treatments--were assigned to the two groups.

Treatment Evaluation

The delineation treatments developed in this study were intended to reduce accidents. It was therefore desirable to evaluate them on the basis of their accident-reduction capability. Because most crash cushion collisions are not reflected in police accident reports and are generally not compiled specifically for gore areas, another measure of treatment effectiveness was needed. Repairs to damaged crash cushions were chosen.

In Houston, repairs to gore area crash cushions are made whenever damage is reported or discovered. Repair records will reflect hits except when another collision occurs before the original damage can be repaired. The best information available suggests that most repairs are made before another collision occurs and that damage is no more likely to go unrepaired at one site than another. Accurate crash cushion repair records are maintained by Texas SDHPT District 12 maintenance personnel. Because of the need for historical data and limitations of the existing accident data base, the consistency of repair procedures during the course of the study meant that repair rates represented the best readily available evaluation measure. The repair frequency serves as an indicator of the accident history of the sites during the same period. It was therefore assumed that changes in repair rates reflected changes in accident rates, and that treatment effectiveness could be judged on the basis of the changes in repair rates before and after treatment installation. Repair data were obtained for 3 years prior to treatment installation and for a period of 12 to 22 months after treatment implementation (depending on the initial installation date). The main limitation of a before and after study, however, is the possibility that changes in the repair rate are a result of factors other than the treatment.

Because it is not practical to control these other factors, there is a chance that changes between the periods of before and after treatment installation are not a result of the treatment. Study designs do exist that randomize the influence of factors other than the treatment. However, these designs generally call for several treatments to be installed at each site and were not practical due to time and cost considerations. In this study, each treatment was assigned to two sites to help balance any possible outside effects. Additionally, the two control sites provide an indication of what the accident experience might have been if no changes in delineation had been made.

RESULTS

The crash cushion repair data were analyzed two different ways. First, the repair rates for the group of six sites where treatments with static delineation only were installed were compared to the repair rates of the group of two sites where flashing lights were installed in addition to static elements. Second, the results for each treatment level were compared.

Comparison of Treatment Groups

The comparison of repair rates between the group of sites where only static treatments were installed and the group where flashing lights were also installed revealed two major findings:

1. Delineation treatments with static reflective signing and flashing lights appeared to reduce the number of repairs at sites with high initial repair rates (9 to 12 repairs per year).
2. Considered as a group, delineation treatments with static reflective signing only did not appear to reduce the number of repairs at sites with moderate initial repair rates (3 to 6 repairs per year).

The basis for these findings is clear after inspection of Figure 5. The "before" and "after" repair histories are shown for the two groups of sites: (a) those with moderate repair rates and static delineation treatments, and (b) those with high repair rates with flashing lights.

As can be seen in Figure 5, the average repair rate for the six sites with static delineation treatments changed only slightly over the study period. A slight decrease of about one repair per year per site occurred in the year after treatment installation, but the repair rate returned to previous levels in the second year after treatment installation.

A different pattern was evident, however, for the two sites where flashing lights were installed in conjunction with static elements. The average number of repairs per year increased during all three years before treatment installation, culminating with an average of 15 repairs per year in the year prior to implementation. The annual repairs per site decreased to about 5 in the year following installation. Importantly, repair rates continued to decline during the second year after treatment installation. The second year repair rates are based on 10 months of data for the group with high rates and an average of almost eight months for the group with moderate rates.

A decline following a sudden increase might be expected as a result of a regression to the mean ef-

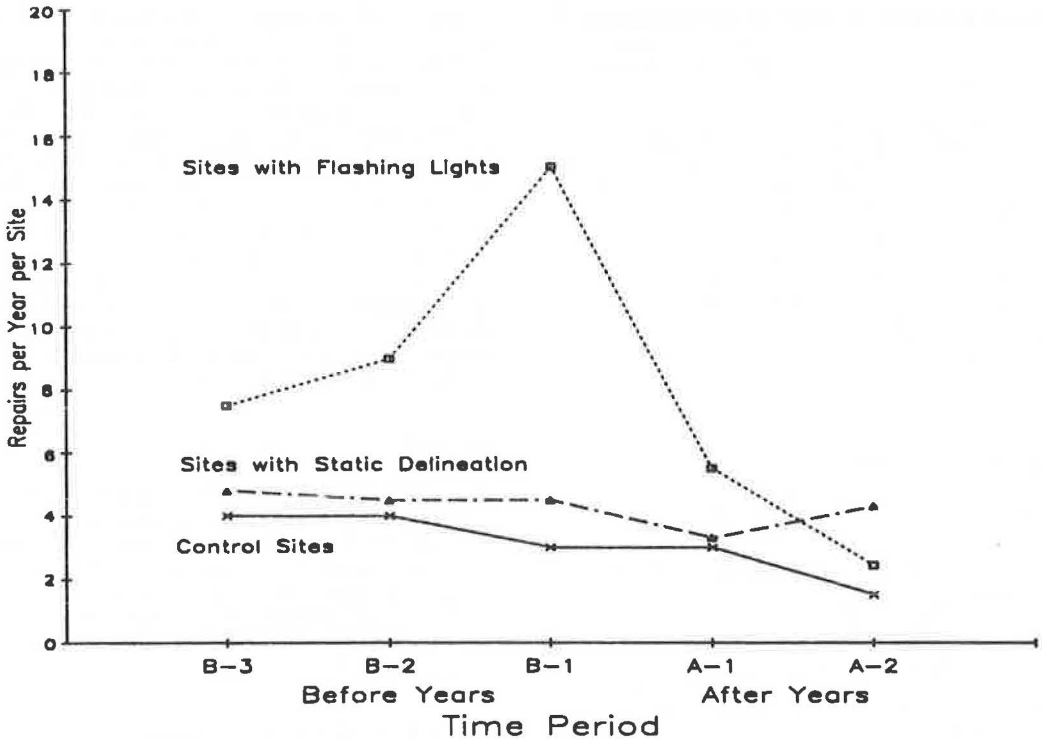

FIGURE 5 Repair rate trends.

fect. On the other hand, the increasing repair rate trend before installation was followed by a consistent reduction to a level below the original average and this occurred at both sites where Level 4 was installed. Other explanations for the changes in repair rates are possible; however, there is evidence that the treatment was responsible for at least some of the reduction, especially if the trends in repairs are examined.

The control group history could normally be used to determine what the accident experience would have been in the absence of a treatment. It is unfortunate, however, that the variation in the repair histories before treatment installation at the high repair rate sites does not match the control group. The control group experience is therefore not appropriate for comparison with the high repair rate group. In addition, the results neither indicate whether treatments involving only static treatment delineation elements would have caused a reduction in repairs at the high repair rate locations, nor do they provide information on the possible effects of flashing lights at sites with moderate repair rates.

Comparison of Individual Treatments

An inspection of the results on a treatment-by-treatment basis reveals that one of the static delineation treatments, Level 2 (base plus back panel), had a greater effect on the repair rate than other static delineation treatments. This finding is demonstrated in Table 3. A reduction in repairs of 58 percent, or 3.0 repairs per year per site, was noted for the sites with Level 2. In contrast, the Level 1 sites experienced an increase in repairs of 12 percent while a reduction of 2 percent was observed for the sites with Level 3.

It should be noted that the Level 2 and Level 3 treatments are very similar, and the Level 3 treatment had a minimal effect on the repair rate at two sites where it was implemented. Thus, it is important to consider the results from both Level 2 and Level 3 sites to assess the effectiveness of this type of treatment. The indication is that the Level 2 or Level 3 type treatment might be effective at some locations, but there is evidence that at other sites, the impact is slight. Site characteristics are likely to have a significant effect on treatment effectiveness. For example, both sites where the Level 3 treatment was installed were located on narrow gores. The sites where the Level 2 treatment was installed were at the gore of two roadways with a greater angle between the two diverging roadways.

TABLE 3 Comparison of Repair Rates Between Treatment Levels

	Repairs per Year per Site			
Treatment	Before[a]	After[b]	Change	Change (%)
None	3.7	2.4 (20)	−1.3	−35
1	3.3	3.7 (17)	+0.4	+12
2	5.2	2.2 (21)	−3.0	−58
3	5.3	5.2 (18)	−0.1	−2
4	10.5	4.2 (22)	−6.3	−60

[a]Based on data for 3 yr.
[b]Based on data for the number of months listed in parentheses.

SUMMARY

This study had provided evidence that supports the following three statements:

1. Treatments that combine flashing lights and static elements reduced repairs at sites with high initial repair rates.
2. Considered as a whole, treatments with the static delineation elements studied did not reduce repairs at sites with moderate initial repair rates.

3. At two sites where the Level 2 treatment (base plus back panel) was installed, reductions in repairs were experienced that were greater than the average found at sites where static treatments were considered as a whole.

It should be kept in mind, however, that this study did not control for changes in other factors, such as traffic volumes and advance signing, which could have influenced the frequency of accidents and, consequently, repairs. In particular, changes in traffic volumes can account for variations in accident frequency. Specific information on year-to-year traffic fluctuations at each site was not available. The best information available, however, suggests that variation in traffic volume was not the cause of the increases in repair rates at the high repair rate sites. Most freeways in Houston have experienced steady growth in traffic during the study period.

It cannot be said with absolute certainty that the changes in repair rates were solely a result of the changes in delineation. However, the results were consistent within groups of sites with similar treatments and repair histories. This investigation of delineation effectiveness at gore areas has provided insight to the problem and has indicated that accidents may be reduced through improved delineation. However, since the treatment with flashing lights was only installed at sites with high initial repair rates and static treatments were installed only at sites with moderate repair rates, the following additional questions are raised by this study:

1. Can treatments that involve only static delineation elements cause a reduction in repairs at sites with high initial repair rates?

2. Can treatments that include flashing lights and static delineation elements cause a reduction in repairs at sites with moderate repair rates?

3. Was the reduction observed at sites with high initial repair rates truly a result of treatment effectiveness?

The use of increased delineation to reduce accidents shows promise and the questions raised in this study warrant further investigation. It is suggested that additional research, using a rigorous study design that accounts for other possible influential factors (especially traffic volumes) be conducted to further explore delineation of crash cushions in gore areas.

ACKNOWLEDGMENTS

The author would like to thank Hunter Garrison of District 12, Texas State Department of Highways and Public Transportation, for providing guidance in the development and conduct of this study and for making the resources of District 12 available for treatment implementation. The efforts of Ervin Ramirez in providing the repair records used in the evaluation are also acknowledged. R. Dale Huchingson and Don L. Woods reviewed this report and contributed to the data analysis. Special acknowledgment is extended to Conrad L. Dudek for his role in conducting the study and preparing this report. The research reported herein was sponsored by District 12 of the Texas State Department of Highways and Public Transportation.

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Timing Traffic Signal Change Intervals Based on Driver Behavior

MYUNG-SOON CHANG, CARROLL J. MESSER, and ALBERTO J. SANTIAGO

ABSTRACT

Driver behavior to traffic signal change intervals (yellow plus all-red) was evaluated by using the data collected from timelapse cameras at seven sites. In particular, signal change interval timing was examined as a function of driver response characteristics involving speed, distance, and time to reach the stop line. Driver-selected yellow response time and deceleration rates were analyzed. New perception and brake reaction time of 1.2 sec with a 10.5 ft/sec^2 of deceleration rate for level grade is suggested. The potential use of a constant yellow interval of 4.5 sec is discussed. In addition, a new method is presented to determine an all-red interval.

The yellow signal indication is a warning of impending loss of right-of-way to the traffic receiving the previous green phase. On seeing the yellow onset, drivers must decide whether to stop or continue through the intersection.

INTRODUCTION

An analysis of physical laws, empirical evidence, and personal driving experience suggests that drivers' behavior in this situation appears to be affected by vehicle speed, position, and physical characteristics as well as other geometric, environmental, and possibly traffic control factors.

If the driver fails to respond safely, a major right angle collision at the intersection is possible and if the driver is startled or overreacts, a hazardous rear-end collision is possible. Because of the complexity of the driver-vehicle-environment control system involved and the potential severe consequences of system failure (a fatal accident), the design of the traffic signal change intervals (yellow time plus any following all-red interval) should be optimized, based on the best understanding of the engineering factors involved. The magnitude of the problem requires that traffic engineers do no less.

Legal Meaning of Change Interval

In 1962, the Uniform Vehicle Code (1) was modified to allow a vehicle to legally enter the intersection on the yellow and to legally clear the intersection when the red signal is displayed. This can be labeled a "permissive rule" in contrast to a "restrictive rule" that required vehicles to clear the intersection before the end of the yellow signal.

Although all states have not adopted the modified Uniform Vehicle Code meaning for the yellow signal and there is a mixture of restrictive and permissive rules across the nation, Bissel and Warren (2) contend that all states operationally allow the intersection clearance to occur during the beginning of the red. Further, a recent survey by Benioff and others (3) has indicated that the procedure used for selecting the change interval was statistically independent of the state law regarding the meaning of the yellow indication.

Signal Change Interval Design

On observing the yellow onset, drivers approaching an intersection are faced with the choice of either stopping the vehicle before entering the intersection or continuing through the intersection. Although several methods and ranges of change interval have been suggested (4), signal change interval design is more frequently based on the equation:

$$Y + AR = t + (v/2d) + [(l + w)/v] \quad (1)$$

where

- $Y + AR$ = duration of the change interval (yellow plus all-red) (sec),
- t = perception and brake reaction time of the driver (sec),
- v = approach speed (ft/sec),
- d = deceleration rate (ft/sec^2),
- w = width of intersection (ft), and
- l = length of vehicle (ft).

Note: Equation 1 was developed by Gazis and others (5) by using the following modeling formulation: A car approaching an intersection is at distance x from the intersection at the yellow onset. If the driver is to stop before entering the intersection, it can be expressed as

$$(x - vt) \geq v^2/2d \quad (2)$$

and if the driver is to clear the intersection completely without acceleration before the green cycle appears on the other street, it can be expressed as

$$(x + w + l) \leq v(Y + AR) \quad (3)$$

Assuming the equality, Equation 2 defines a stopping distance (x_s) as

$$x_s = vt + v^2/2d \quad (4)$$

and Equation 3 defines the clearing distance (x_c) as

$$x_c = v(Y + AR) - (w + l) \quad (5)$$

If $x_c > x_s$, and a driver is positioned between x_s and x_c such that $x_s < x < x_c$, then the driver can either stop or clear the intersection (called the nondilemma zone). However, if $x_c < x_s$ and a driver is positioned between x_c and x_s such that $x_c < x < x_s$, then he will be in a position where he can neither stop safely nor proceed through the intersection completely (called the dilemma zone). Therefore, the minimum change interval satisfying the safe execution of either one of the alternatives (stopping or going through the intersection without acceleration) corresponds to $x_c = x_s$. Then,

$$(Y + AR)v - (w + l) = vt + (v^2/2d) \quad (6)$$

By dividing both sides by v, however,

$$Y + AR = t + (v/2d) + [(w + l)/v] \quad (7)$$

Study Objectives

The objectives of this study are to (a) develop a comprehensive understanding of drivers' responses to the change intervals, (b) examine change interval timing as a function of driver behavior, and (c) quantify the values of the variables associated with driver reaction time and deceleration rates as applied in the Equation 1 from field studies.

DATA COLLECTION

Two timelapse cameras were used to collect data for each approach at an intersection to reduce the potential reading error in distance near the intersection when employing only one camera on an approach. The detailed description of the data collection method along with the definition of sample vehicles is found elsewhere (6,7,8).

Seven intersections (three in Virginia and four in Texas) were studied during the summer of 1983. The geometric and traffic control characteristics at the seven intersections studied are presented in Table 1. Intersections observed for this study included a variety of combinations in intersection width, controller type, grade, and change interval. Passenger cars and through vehicles approaching at speeds higher than 20 mph (composed of 1,035 clearing and 579 stopping vehicles) were collected during operating conditions that included day and night, dry and wet pavement, and peak and offpeak periods.

The list of data collected to evaluate the traffic signal change interval design and driver's response is as follows:

1. The distance from intersection and speed of approach vehicle at the onset of yellow and the driver's decision to continue or to stop,
2. The time and distance at which brakes were applied,
3. The time and distance when a vehicle stopped,
4. The time when a vehicle entered and cleared an intersection,
5. The time when a vehicle in queue started moving, and
6. The type and directional movement of a vehicle.

DATA REDUCTION

To convert film distance to roadway distance in data reduction, four roadway reference points (RRPs), three of which were not on a straight line, were established during data collection.

Establishment of Film/Roadway Relationship

Exposed film was loaded into a TIMELAPSE Model 3420 Data Analyzer Projector. Then, x and y coordinates of those field reference objects could be read with a convenient scale. The corresponding roadway coordinates of these four film reference points were already measured in the field and were available. By using the basic characteristics of photogrammetry (9), the relationship between film and roadway plane can be developed. A computer program modified from that of Bleyl (10) was developed to convert roadway coordinates to film coordinates.

Drawing of Roadway Distance Contour on Film Plane

Given four corresponding coordinates of reference points for each film and roadway, any roadway distance or points can be converted to that of the film plane. A 10-ft distance contour map was drawn from computer output. Then, the film RRPs were superimposed onto the graph RRPs. After completion of this superimposition, data reduction can be performed to read time, distance, and other characteristics of a particular vehicle on the film.

TABLE 1 Geometric and Traffic Control Characteristics of Study Sites

Location Area	Intersection Approaches	Grade (%)	Type	Speed Limit (mph)	Yellow Time (sec)	All Red Time (sec)	Intersection Width (ft)
Urban	1. Commerce, Texas	-2.5	A[a]	30	4.5	1.5	125
	2. Industrial, Texas	0.5	A	35	4.5	1.5	125
	3. US 29, Virginia	-4.5	P[b]	35	3.0	1.5	220
	4. US 50, Virginia	-5.0	P	35	3.0	1.5	200
	5. Texas Avenue, Texas	0.8	P	45	4.0	1.5	100
	6. University Drive, Texas	-0.8	P	40	4.5	1.5	100
Suburban	7. South Lamar, Texas	-3.5	P	45	4.0	0.0	60
	8. Old Keene 1, Virginia	-3.5	A	45	5.0	0.0	60
	9. Old Keene 2, Virginia	-6.0	A	45	5.0	0.0	60
Rural	10. US 1, northbound, Virginia	1.0	A	50	5.0	1.0	80
	11. US 1, southbound, Virginia	-6.5	A	50	5.0	1.0	80
	12. SH 1, Texas	0.0	A	55	4.0	1.0	80
	13. SH 2, Texas	0.0	A	50	3.0	1.0	80

[a]A = actuated traffic controller.
[b]P = pretimed traffic controller.

Derivation of Yellow Response Time and Deceleration Rate

Yellow response time is measured in this study as the time elapsed from the onset of yellow until the brake light is observed to come on.

Deceleration rates can be derived from either one of the following equations:

$$d_t = 2L/T^2 \tag{8}$$

or

$$d_v = v^2/2L \tag{9}$$

L = braking distance (ft),
T = time elapsed from brake actuation to a complete stop,
v = speed at the time of brake actuation,
d_t = deceleration rate derived from using L and T, and
d_v = deceleration rate derived from using L and v.

It should be noted that between two deceleration rates in Equations 8 and 9, the expression vT = 2L must hold. If any measurement error is involved in one of v, T, and L, the two deceleration rates will not be identical. For this study, the (vT - 2L) has an error of ±5 ft and the average of d_t and d_v is used as a deceleration rate (d) for this study.

STUDY FINDINGS

The observed relative frequency of stop and go characteristics introduce the findings. Basic descriptive statistics on yellow response time and deceleration rates observed from field studies are presented. The potential perception and brake reaction time is deduced from the yellow response time with consideration of speed influence. Subsequently, other relevant information, such as the time taken for the clearing vehicle to reach the stop line, follows. Further, other factors that may influence driver-selected characteristics of yellow response time, deceleration rate, and the decision to stop at or go through a yellow light were analyzed.

Driver Response Characteristics to Change Interval

The data in Table 2 indicate the observed relative frequency of driver response characteristics with respect to signal change intervals. It shows that the overall relative ratio of stopping or going is one to two. Fifty-seven percent of the total vehicles entered intersections during the yellow cycle. Among these vehicles, two-thirds cleared the intersection during the yellow cycle but the other one-third cleared during the red cycle. The high number and percentage of "yellow entering" and "red clearing" at the site of US 29 and US 50 is attributed to the extreme width of the intersection.

The Table 2 data also indicate that 7 percent of the total number of vehicles entered the intersection during the red cycle. A substantial portion of those vehicles were observed at the sites of US 29 and US 50 in Virginia and on state highways in Texas. These two sites were operated at long cycle lengths and were observed to experience frequent long queues. The traffic operation appeared to contribute the impetus for drivers to take a high risk by entering during the red cycle.

From observed site geometric and traffic operational conditions, two suggestions can be made to reduce the frequency of vehicles that enter and clear during the red cycle: (a) that a sufficient all-red interval is to be used for those wide intersections, and (b) that traffic operations particularly as a result of long cycle length should be improved at those sites that experience a high proportion of vehicles that enter during the red cycle.

Yellow Response Time

Table 3 summarizes the values observed for yellow response time at each intersection approach and the total vehicles observed. It shows that the mean yellow response time of all drivers in the subject population was 1.3 sec and the median was 1.1 sec. Eighty-five percent of stopping vehicles applied their brakes within 1.9 sec while 95 percent did it within 2.5 sec. The cumulative distribution of yellow response time for 579 stopping vehicles is shown in Figure 1.

Yellow response time usually also includes some lag time because most situations do not require an immediate braking reaction. To derive a perception and brake reaction time from yellow response time, speed influence is introduced. The hypothesis is that drivers' yellow response time at high speed (for example, 50-55 mph) may be closely equivalent to their perception and brake reaction time because their high speeds require immediate reactions to avoid excessive deceleration or even collision with other vehicles.

Figure 2 presents the yellow response time classified by speeds. The observed speeds were classified into seven categories from 25 to 55 mph. The speed shown is the middle point of 10-mph intervals. It is shown that the median yellow response time is stabilized at 0.9 sec at speeds over 45 mph. The current value of 1 sec suggested by the ITE handbook (11) corresponds to 70 percent of the total vehicles observed in the 55 mph speed categories in this study.

TABLE 2 Observed Relative Frequency of Driver Response to Signal Change Interval

	Intersection Approach No.														
Vehicle Action	1	2	3	4	5	6	7	8	9	10	11	12	13	Total (actual)	Total (%)
S[a]	20	27	64	126	37	43	59	38	14	81	37	16	17	579	36
YEC[b]	10	6	0	0	24	34	46	73	40	253	131	2	1	620	38
YERC[c]	2	22	75	105	10	3	20	9	4	25	4	6	14	299	19
RE[d]	1	1	18	64	8	0	1	2	0	2	2	9	8	116	7
Total	33	56	157	295	79	80	126	122	58	361	174	33	40	1,614	

[a]S = stopping.
[b]YEC = enter and clear during yellow
[c]YERC = enter during yellow and clear during red.
[d]RE = enter during red.

TABLE 3 Summary Characteristics of Stopping Vehicles

	Intersection Approach No.													
Variables	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
YRT[a]														
Mean	1.2	1.5	1.4	1.4	1.3	1.3	1.4	1.1	0.7	1.0	1.2	1.0	1.0	1.3
Median	1.0	1.4	1.2	1.3	1.1	1.0	1.3	1.0	0.7	0.9	1.1	1.0	1.0	1.1
85 percent	1.7	2.1	1.9	2.2	1.9	1.8	2.0	1.6	1.0	1.5	1.6	1.3	1.1	1.9
95 percent	2.0	2.4	3.2	2.9	3.4	2.7	2.8	1.9	1.2	1.9	1.9	1.5	1.4	2.5
DR[b]														
Mean	9.3	8.6	8.6	7.8	10.6	9.8	7.8	10.7	13.4	11.5	10.8	8.3	8.9	9.5
Median	7.7	8.6	8.1	7.6	10.9	9.4	7.9	10.2	12.9	10.8	11.3	8.1	9.1	9.2
85 percent	4.2	5.9	5.3	5.0	6.5	6.9	4.8	5.6	9.7	8.4	6.4	6.2	5.4	5.6
95 percent	3.1	3.6	3.8	4.2	5.5	5.5	3.7	4.6	7.4	6.8	4.2	4.0	4.9	4.3

[a]YRT = yellow response time (sec).
[b]DR = deceleration rate (ft/sec^2).

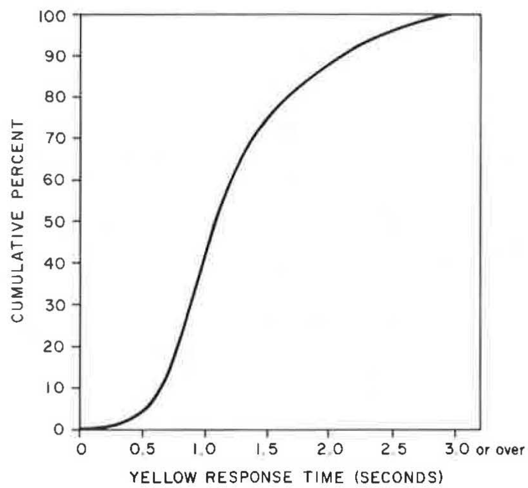

FIGURE 1 Cumulative distribution of yellow response time.

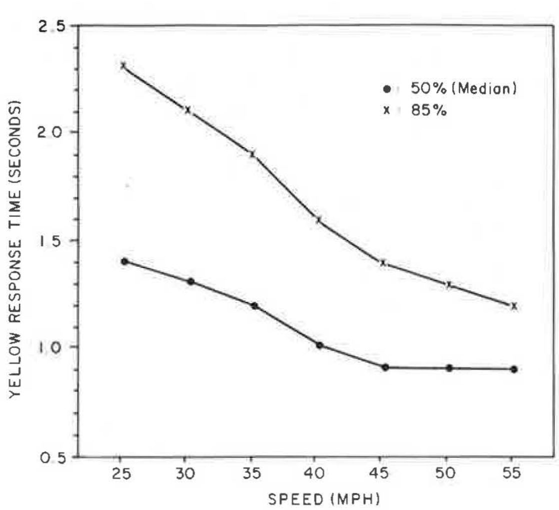

FIGURE 2 Yellow response time by speed categories.

To further validate the conceptual appropriateness of the derivation of perception and brake reaction time from yellow response time at high speed categories, another similar case requiring immediate reaction is considered in terms of distance for vehicles traveling over 40 mph. The reader is reminded that as vehicles move closer to the intersection, drivers tend to react immediately, whereas, when they are further away, the yellow response time will involve a substantial amount of response lag time. Figure 3 presents the yellow response time by distance for vehicles traveling over 40 mph. The distances shown are the middle points of 100-ft intervals. It is shown that when vehicles are relatively closer to the intersection, their 85 percent yellow response time was 1.1 sec at 200 ft and 1.3 sec at 250 ft. It is also noted that the median yellow response time for vehicles approaching over 40 mph is 0.9 sec. Combined results from Figures 2 and 3 indicate that the median perception and brake reaction time of drivers is 0.9 sec. The response lag time for the median drivers is not expected to be significant and the probable value may be around 0.1 sec.

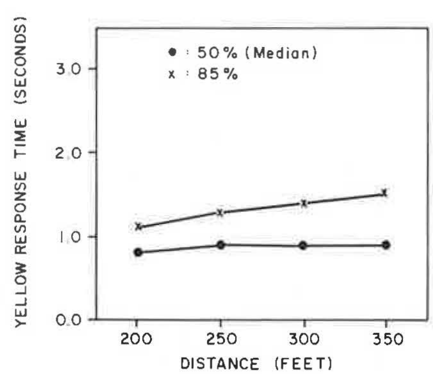

FIGURE 3 Yellow response time by distance for speed over 40 mph.

If the practice of setting the speed limit as 85 percent of the approach speed is adopted, the combined results from Figures 2 and 3 indicate that 1.2 sec of yellow response time observed from both higher speed categories and the closer distances to the intersection appears to be a good estimator of perception and brake reaction time. It is also noted that the 1.2 sec will also include an unidentified amount of response lag time. Therefore, the 85 percentile value taken from yellow response time may be close to a 90 or 95 percent value in the perception and brake reaction time distribution.

Factors That Affect Yellow Response Time

The general characteristics of yellow response time affected by driver approach speeds, distance to the intersection, and interaction of these two are reported in a previous pilot study (8). This expanded analysis revealed similar characteristics. The effect of speed on yellow response time previously shown in Figure 2 illustrated that the yellow response time decreases as speed increases, and increases as speed decreases. This relationship is apparently attributed to driver response lag time, which usually occurs between perception and brake reaction.

To examine the effect of distance on yellow response time, the observed distances were classified into six categories ranging from 100 to 350 ft. Figure 4 presents the effect of distance on yellow response time over all speed categories. The distance shown is the middle point of 100-ft intervals. It shows, in general, that yellow response time increases as the distance to the intersection increases, and decreases as the distance to the intersections decreases. The driver response lag time is also applicable to this phenomenon.

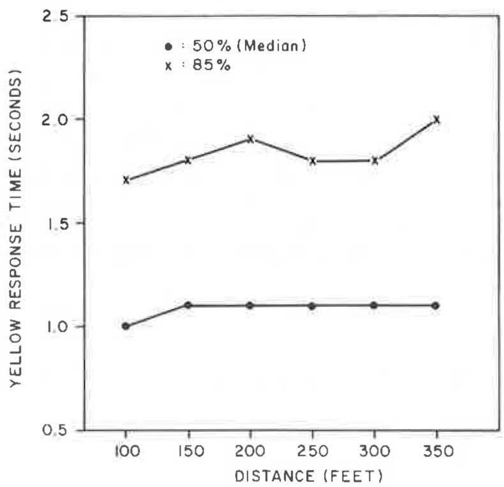

FIGURE 4 Yellow response time by distance categories.

To understand the combined effects of speed and distance, and its interaction on yellow response time, stepwise multiple regression was used and the best model obtained at $\alpha = .03$ is as follows:

$$YRT = 0.507 - [0.712\ (DONSETY/100)] + [0.423\ (DONSETY/ASPEED)] + \{0.091\ [(DONSETY/100)^2]\} \quad R^2 = 0.39$$

where

YRT = yellow response time (sec),
DONSETY = distance to intersection at yellow onset (ft), and
ASPEED = approach speed of vehicle (ft/sec).

Graphic presentation of the model in Figure 5 shows that driver yellow response time decreases as approach speed increases, and decreases as distance to the intersection at yellow onset decreases. Further, the model revealed that driver's yellow response time decreases as time available to reach the stop line decreases.

Effect of Signal Controller Type on Yellow Response Time

A test was performed to see if there is any effect of signal controller type on yellow response time. The test revealed that there is a statistically significant effect, at $\alpha = .05$, as a result of a different controller type on yellow response time. It was specifically noted that drivers tended to react more quickly to the actuated controller than to a fixed time controller; however, the magnitude of difference was found to be less than 0.1 sec. Thus, the effect of controller type on yellow response time is practically negligible.

Effects of Light and Weather on Yellow Response Time

Three intersections that have sufficient samples encompassing day and night were tested to see if there is any difference in yellow response time as a result of light conditions. The test results indicated that there is no difference (at $\alpha = .05$). This suggests that drivers tend to react consistently during day and night.

One intersection that had sufficient samples covering dry and wet pavement conditions was tested to see if there is any difference in yellow response time due to weather. The test results indicated that there is no difference in yellow response time (at $\alpha = .05$). This suggests that drivers do not appear to adjust their response particularly as a result of wet pavement conditions.

Discussions of the Various Effects on Yellow Response Time

The effects of various traffic control and environment conditions on yellow response time were tested. (Note that these effects were not tested on perception and brake reaction time.) The test results are to be interpreted such that the majority of drivers do not appear to react differently because of different conditions. It may be expected that the same driver may react more quickly during night and/or wet pavement conditions. However, the test based on identical drivers could not be performed for this study. It should also be noted that even if there is a difference in perception and brake reaction time for a different condition by a driver, the practical difference may be small because of the limitations on the extent of mental and physical reactions.

Deceleration Rate

The data in Table 3 also indicate the deceleration performance observed from field studies: They indicate that the mean and the median deceleration performance for the total vehicles was 9.5 and 9.2 ft/sec^2, respectively. Eighty-five percent of vehicles selected a deceleration rate of 5.6 ft/sec^2 or more and 95 percent used 4.3 ft/sec^2 or more. The cumulative distribution of deceleration rates observed for 579 stopping vehicles is shown in Figure 6. It should be noted that the deceleration rate observed from field studies is primarily the result of the driver's selection of comfort. It is not an indication of whether they can perform certain deceleration rates.

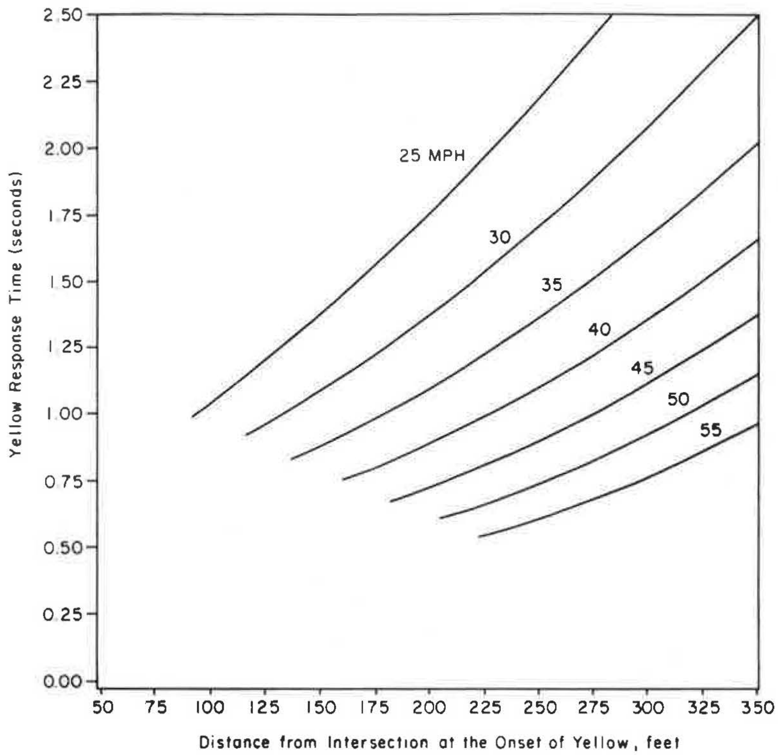

FIGURE 5 Yellow response time as a function of distance, speed, and time.

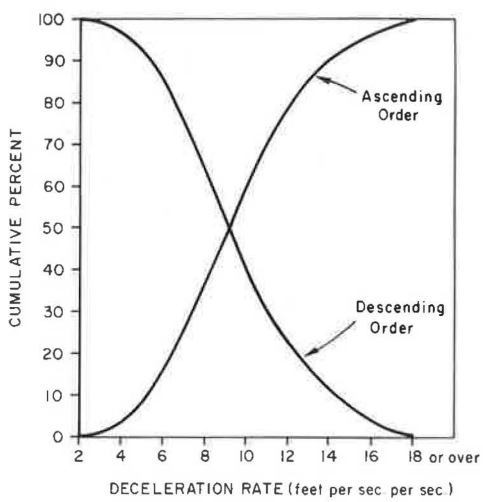

FIGURE 6 Cumulative distribution of deceleration rate.

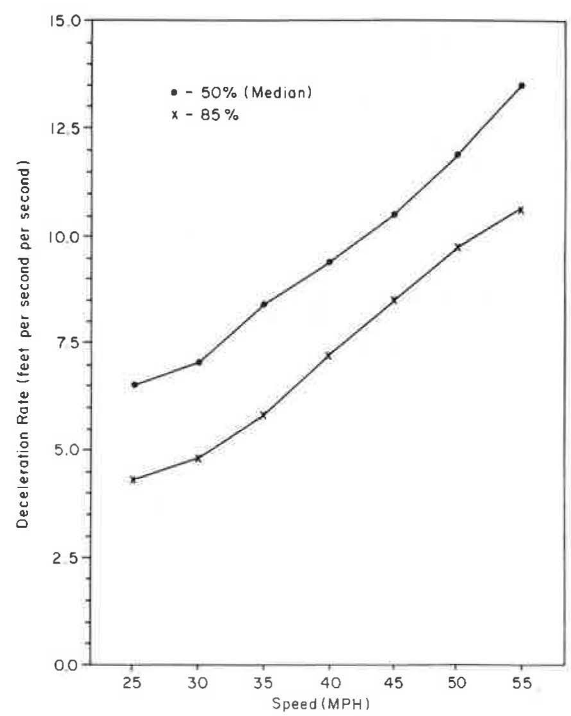

FIGURE 7 Deceleration rates by speed categories.

To derive a deceleration rate that drivers can perform, the speed influence approach used in deriving perception and brake reaction time is adopted. Figure 7 shows the deceleration performance categorized by speed. It shows that 85 percent of vehicles at 55 mph speed categories can perform deceleration rates of 10.6 ft/sec^2 or more. The deceleration rate of 10 ft/sec^2 assumed by the ITE Handbook (11) corresponds to 90 percent performance in this speed category. It is further noted that this 10 ft/sec^2 deceleration rate can also be performed by more than 85 percent of trucks (12). A deceleration rate of 10.5 ft/sec^2 is suggested for level grade.

Factors That Affect Deceleration Rate

The model to evaluate the factors that affect deceleration rate must be guided by the laws of motion from physics. These indicate that the deceleration rate is affected by speed, distance, time, and grade. Because the braking distance available depends on the distance traveled during yellow response time, the additional interaction variable between speed and yellow response time is introduced. The deceleration rate (DR) model obtained from stepwise regression at α = .01 is as follows:

$$DR = 13.365 + \{0.176\ [(ASPEED/10)^2]\} - [2.933\ (DONSETY/100)] + 0.085\ GRADE - [1.110\ (DONSETY/ASPEED)] + [0.044\ (ASPEED \times YRT)] \qquad R^2 = 0.86$$

The model shows that deceleration rate increases as speed, grade, and distance traveled during yellow response time increases. It increases as distance to the intersection at yellow onset and available time to reach the stop line decrease. The square distance term ($DONSETY^2$) is added for graphic presentation for the case of grade = 0 and YRT = 1 sec shown in Figure 8 to increase its predictability.

Grade Effect on Deceleration Rate

The adjustment of grade effect on deceleration performance has been advocated by some researchers (13). Consequently, the effect of grade on deceleration rate was tested. Several multiple linear regression models were evaluated by using the general linear test method. Throughout the models tested, the coefficients of grade were significant at α = .05 and remained relatively stable at around .065 to .085 at an average of .075.

When the exact adjustment of grade effect on deceleration rate is desired, the following equation may be used:

$$d = 10.5 \pm 0.075g \qquad (10)$$

where g is the absolute percent of grade (use positive for upgrade and negative for downgrade). For safety and practical purposes, use of the following deceleration rate is recommended:

For level and upgrade	$d = 10.5\ ft/sec^2$
For downgrade	$d = 10.0\ ft/sec^2$

Effect of Other Factors on Deceleration Rate

A test was performed for three intersections that had sufficient samples covering day and night samples to see if there is any difference on deceleration rate as a result of light condition. The test results indicated that there was no difference in deceleration rate (at α = .05). This suggests that drivers do not appear to select significantly different deceleration rates during night as opposed to day. Further, a test was performed for an intersection that had sufficient samples covering dry and wet pavement conditions to see if there was any difference on deceleration rates as a result of weather. The test results indicated that there was no difference in deceleration rate (at α = .05).

It should be noted that the limitations previously described in the "Deceleration Rate" section also hold for this test.

Time Effects on Drivers' Decision to Stop or Go

It is hypothesized that drivers' perceived times to reach the stop line may influence their decision to stop or go. The time to reach the stop line for stopping vehicles is obtained by assuming constant approach speed. The time to reach the stop line for going vehicles is the actual time elapsed from the yellow onset to reach the stop line.

Figure 9 presents the time effect illustrated by speed categories. It shows that

1. Practically no vehicles stopped when they were 2 sec or less away from the intersection,
2. Eighty-five percent of stopped vehicles did stop because they were about 3 sec or more away from the intersection,
3. Eighty-five percent of going vehicles continued through the intersection because they could actually enter it within approximately 3.7 sec or less travel time,
4. Ninety-five percent of going vehicles took less than 4.5 sec to enter the intersection, and
5. The time effect was relatively stable across the speed categories. (Note: It is important to remember that these data pertain to stopping and going vehicles.)

Figure 9 also shows the dilemma of continued use of current change interval design formula. The current formula provides increased time for higher speeds while practice provides minimum yellow interval for lower speeds. Figure 9 shows that the real

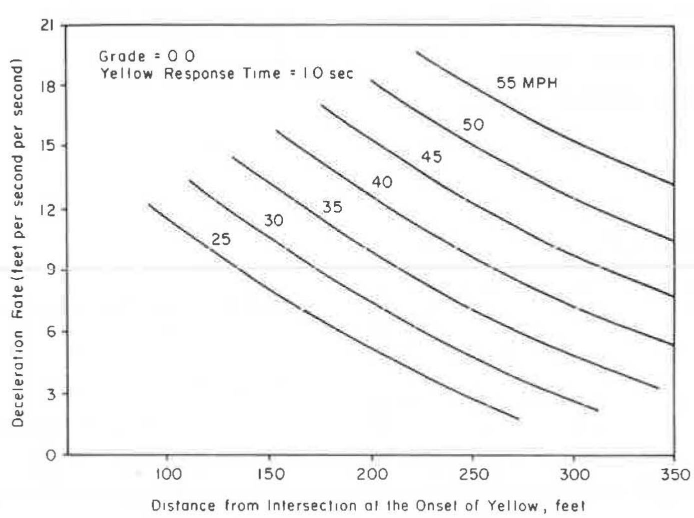

FIGURE 8 Deceleration rate as a function of distance, speed, and time.

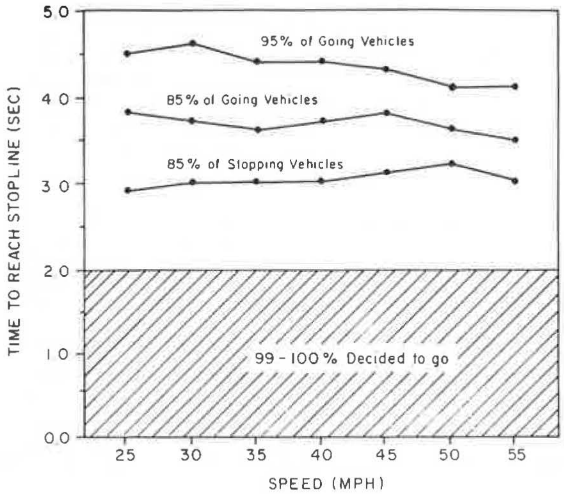

FIGURE 9 Driver's decision to stop or go by time.

danger may lie in lower speed categories below 40 mph.

Figure 9 also provides a good opportunity to consider the use of a constant yellow interval across the speed ranges. Olson and Rothery (14) suggested a constant yellow interval of 5.5 sec proclaiming that such yellow duration will provide all or nearly all drivers with time to clear an intersection. While their justification does not appear to be sound because of different dimensions of intersection width, Figure 9 appears to show a warranting condition for a constant yellow interval of 4.5 sec from the fact that 95 percent of going vehicles did go through when they took less than about 4.5 sec regardless of their speeds. The determination of yellow interval based on going vehicles is warranted because the fundamental problem of the yellow interval lies in the clearing vehicle rather than the stopping vehicle. The basic reason is that the first car stopped has no vehicles with which to collide. The following vehicles may collide with the first vehicle stopped. However, the rear end collision in this case is a result of driver expectancy violation along with following too closely, rather than a consequence of the yellow interval.

Probability Modeling of Driver Decision to Stop or Go

Past studies reported that driver decisions to stop or go were affected by approach speed, distance from the intersection at the yellow onset, and the time to reach the stop line (15,16,17). These three different decision-affecting factors are illustrated in Figure 10. Figure 10a shows the case of speed dominance decision in which the slope of the same time is downward to the lower probability of stopping as distance is increased. Figure 10b shows the distance dominance decision in which the slope of the same time is upward to the higher probability of stopping as distance is increased. Figure 10c shows the time dominance decision in which the slope of the same time has approximately the same probability of stopping. The model by Williams (15) has a time characteristic of Figure 10b while the model by Sheffi and Mahmassani (16) has a time characteristic of Figure 10c.

Logistic regression (or logit model) was used to derive the probability of stopping or going as a function of speed, distance, and time. For the stopping vehicle, time is derived by assuming constant speed as mentioned previously. The stepwise logistic regression model revealed that the first important variable entered was time, the second was distance, and the third was speed in sequence. However, when the distance and speed were entered, time became insignificant at the chi-square value of 0. Thus, the model obtained at α = .05 is as follows:

Probability of stopping = 1/{1 + exp [2.083 - 2.755 (DONSETY/100)] + [0.071 x ASPEED]}

The model revealed that the probability of stopping decreases as distance decreases and it decreases as approach speed increases. The graphic presentation shown in Figure 11 illustrates the characteristics. Figure 11 also revealed that the driver decision is a distance dominance pattern previously illustrated in Figure 10b. The predicted performance of the probability model compared to the observed frequencies of stopping and going is shown in Table 4. The probability model predicted with 80 percent accuracy the responses of stopping and going.

Effect of Grade on Driver Decision to Stop or Go

It is postulated that grade may have an effect on drivers' decision to stop or go. It is expected that more drivers may decide to go through rather than to stop, given the same approach speed and distance to the intersection, at downgrades than upgrades. The effect of grade on drivers' decision to stop or go

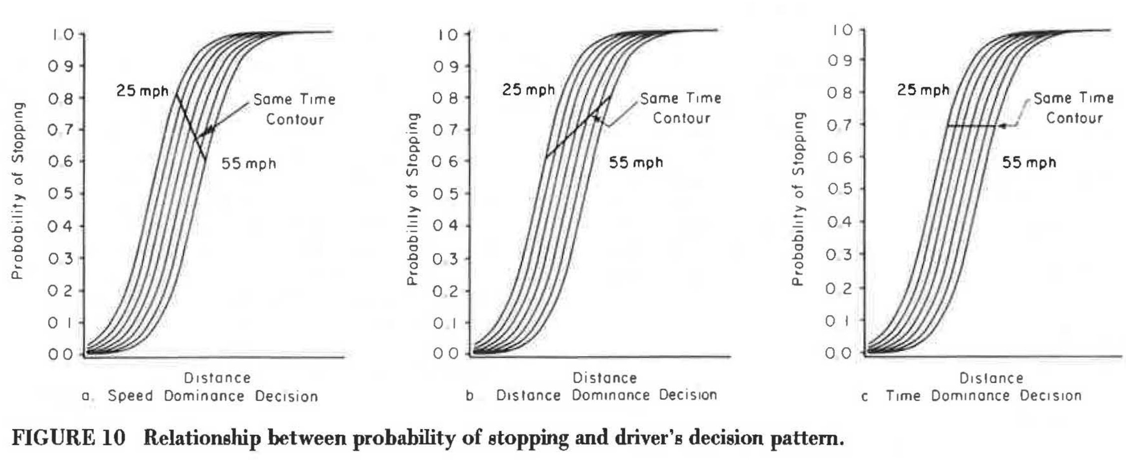

FIGURE 10 Relationship between probability of stopping and driver's decision pattern.

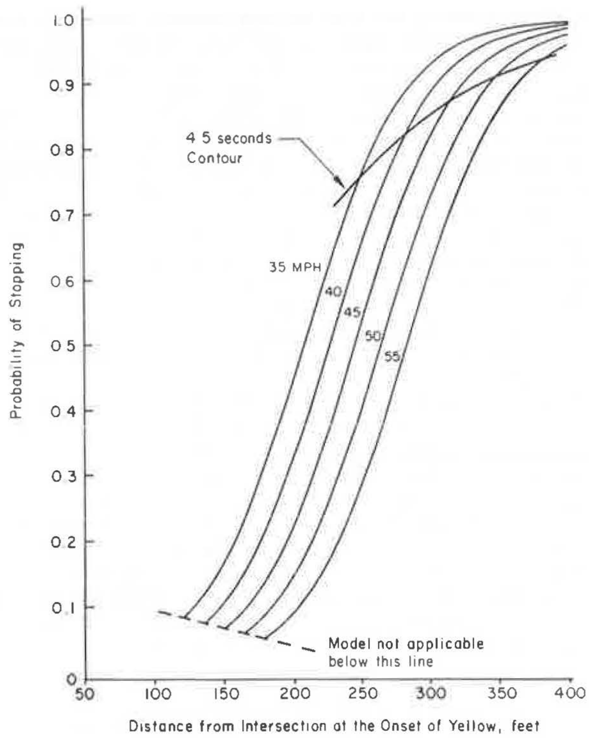

FIGURE 11 Probability of stopping as a function of distance and speed.

TABLE 4 Predicted Performance of Probability Model

	Predicted		
Observed	Going	Stopping	Total
Going	887	148	1,035
Stopping	182	397	579
Total	1,069	545	1,614

Note: The correct rate = (887 + 397)/(1,614 = 79.6 percent; the false stopping rate = 148/545 = 27.2 percent; and the false going rate = 182/1,069 = 17.0 percent.

was tested using the logistic model. The model obtained at α = .01 is as follows:

Probability of stopping = 1/{1 + exp [1.870 − 2.790 (DONSETY/100)] + 0.069 x ASPEED − 0.115 x GRADE}

where grade is the percent of grade (use positive for upgrade and negative for downgrade).

The model revealed that the probability of stopping increases as grade increases and it decreases as grade decreases. In other words, more drivers tend to stop on upgrades but tend to go through on downgrades. The provision of an all-red interval on downgrades would be helpful to counterbalance the potential accidents as a result of the greater tendency toward drivers going through on downgrade approaches.

Effect of Intersection Width on Driver Decision to Stop or Go

It is also postulated that intersection width may also have an effect on the driver's decision to stop or go. It is expected that more drivers may decide to go through rather than stop, given the same approach speed, distance, and grade to the intersection, at narrower intersections than at wider intersections. The effect of intersection width on the driver's decision to stop or go was tested by using the logistic model. The model obtained at α = .01 is as follows:

Probability of stopping = 1/{1 + exp [5.038 − 3.013 (DONSETY/100)] + 0.044 x ASPEED − 0.198 x GRADE − 0.014 x INTW}

where INTW is the intersection width in feet. The model revealed that the probability of stopping increases as intersection width increases and it decreases as intersection width decreases. In other words, more drivers tend to stop at wider intersections but tend to go through at narrower intersections.

All-Red Interval

All-red time is often provided at some intersections to let vehicles clear the intersection during the protected time. All-red time is particularly useful when the intersection is wide and when many vehicles tend to enter the intersections during the latter part of the yellow interval.

Speed Influence on All-Red Time

The current change interval design provides clearance time in the form of (1 + w)/v. It is noted that a constant approach speed is used to derive clearance time. Observations revealed that the majority of drivers accelerate when they need to clear the intersection. It also appears to be a duty for those drivers who entered the latter part of the yellow cycle to clear the intersection as soon as possible within their vehicles' capabilities. The speed difference between the approach speed before the yellow cycle (ASPEED) and the final average speed after the yellow cycle until the vehicle clears the intersection (FSPEED) was analyzed and the relationship was obtained at α = .01 as shown in the following equation:

$$\text{FSPEED} = 1.08\ \text{ASPEED} \qquad R^2 = 0.99 \qquad (11)$$

This suggests that use of constant speed may provide unnecessarily long all-red time particularly when the intersection is wide.

Starting Delay Influence on All-Red Time

There is a delay from the time the driver first sees the green onset until the driver starts moving the vehicle from a stopped position. The starting delay obtained from this study was analyzed and the starting characteristics of a total of 3,527 vehicles (being the first ones positioned in queue) were observed at the onset of the green cycle. Twenty-seven vehicles started before green onset (called "light jumping"), which was 0.8 percent of the total samples. (It should be noted, however, that light jumping is an illegal violation of traffic signal display.) The mean starting delay was 1.8 sec and the median was 1.7 sec. Eighty-five percent of vehicles took longer than 1 sec to start. Ninety-five percent of vehicles took more than 0.8 sec to start. Since the stop line was set back from the path of cross traffic, the use of a 1-sec starting delay may be applicable to 95 percent of vehicles.

Application of Results--Example to Determine All-Red Interval

Consider the following example of determining the duration of the all-red interval. The following intersection characteristics will be assumed:

- Speed limit or 85 percentile speed = 40 mph (58.7 ft/sec),
- Intersection width = 100 ft,
- Passenger car length = 20 ft, and
- Yellow time = 4 sec.

Step 1. Calculate the distance traveled during the yellow interval: Distance = 58.7 x 4 = 235 ft;
Step 2. Add the intersection width and the passenger car length: Distance = 235 + 100 + 20 = 355 ft;
Step 3. Divide the Step 2 distance by FSPEED: Time = 355/(58.7 x 1.08) = 5.6 sec;
Step 4. Subtract the starting delay from the Step 3 time: Time = 5.6 - 1.0 = 4.6 sec;
Step 5. Subtract the yellow time from Step 4: All-red interval = 4.6 - 4.0 = 0.6 sec.

If the value obtained in Step 5 is negative, no all-red time is necessary. Therefore, this intersection would need 0.6 sec of all-red time.

It is warned, however, that the starting delay of 1 sec should be applied with extreme caution and a value of 0 should be used under the following conditions: (a) the driver's view to the intersection is obstructed as a result of either intersection geometrics or adjacent large vehicles such as trucks, and (b) the crossing signal is visible or progression is provided such that approaching vehicles either do not tend to stop completely or do not take significant time to start. Further, legal implications based on local laws and ordinances should be investigated before the engineer decides to apply the cross-flow reduction value.

Change Interval and Signal Lost Time

Signal lost time is a parameter used for the calculation of signal timing and is consequently applied to effective green time and level of service at signalized intersections. The signal lost time is defined as

$$T_l = (Y + AR) - U_y + T_s \qquad (12)$$

where

T_l = signal lost time (sec),
U_y = utilized yellow time (sec), and
T_s = starting delay (sec).

The current study revealed that the mean travel time for clearing vehicles was 2.6 sec and the mean starting delay was 1.8 sec. Therefore, mean signal lost time will be

$$T_l = (Y + AR) - 2.6 + 1.8 = Y + AR - 0.8 \qquad (13)$$

Taking a conservative value, the signal lost time corresponds to (Y + AR) - 1 sec.

CONCLUSIONS

The following conclusions were drawn from the data collected and field observations made within this study. They apply within the seven intersections studied and the observed operational environments.

1. The observed mean yellow response time selected by drivers was 1.3 sec and the median was 1.1 sec. Eighty-five percent of stopping drivers applied their brakes within 1.9 sec after the yellow onset while 95 percent of drivers did it within 2.5 sec.
2. The derived 85 percent perception and brake reaction time excluding driver's response lag time from yellow response time was 1.2 sec.
3. Driver's yellow response time is affected by distance to the intersection at the yellow onset, approach speed, and the time available to reach the stop line after the yellow onset.
4. The observed mean deceleration rate selected by drivers was 9.5 ft/sec^2, and the median was 9.2 ft/sec^2.
5. Grade affects deceleration rate approximately 0.075 ft/sec^2 for each percentage of grade. For safety and practical purposes, a deceleration rate of 10.5 ft/sec^2 is suggested for level and upgrades, and 10.0 ft/sec^2 is suggested for downgrade.
6. Driver-selected deceleration rate was affected by approach speed, the distance to the intersection at the yellow onset, the time available to reach the stop line after the yellow onset, and the distance traveled during the yellow response time.
7. Eighty-five percent of stopping vehicles stopped when they were more than 3 sec away from the intersection.
8. Eighty-five percent of going vehicles went through the intersection when they were less than 3.7 sec away. Ninety-five percent of going vehicles continued when their travel times to the intersection was less than 4.5 sec.
9. The safety implication of going vehicles and the stability of going vehicles with respect to time suggest the potential use of a constant yellow interval of 4.5 sec across all speeds.
10. Driver probability of stopping or going was affected by approach speed and the distance to the intersection at the yellow onset.
11. The higher risk of accidents as a result of the yellow interval appears to exist for the lower speed categories below 40 mph.
12. The average speed for going vehicles from the yellow onset to clearing the intersection is 8 percent higher on the average than their approach speeds before the yellow onset.
13. The mean starting delay to the green onset was 1.8 sec and the median was 1.7 sec. Eighty-five percent of the vehicles took more than 1 sec to start. This starting delay may be applicable to determine all-red time.

ACKNOWLEDGMENTS

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Abridgment

Route Designators to the Centers of Large Urban Areas and Suburbs Within Urban Areas

ROGER W. McNEES

ABSTRACT

Presented in this paper are the results of a laboratory study to determine proper freeway guide sign descriptors for use in directing motorists to the central business district (CBD) of a large urban (metropolitan) area and to the central business district of a suburb situated in a large metropolitan area. Locations within the metropolitan area where the various descriptors are appropriately used are also discussed. Both one-word and two-word descriptors are suggested. The drivers approaching the urban area (15 to 20 mi away) indicated that they preferred the NAME OF THE CITY as the one-word descriptor and DOWNTOWN--NAME OF THE CITY as the two-word descriptor. As they approach the loop area (5 to 10 mi from the CBD) the preferred single-word message was DOWNTOWN or BUSINESS, and the two-word message was again DOWNTOWN--NAME OF THE CITY. As they were approaching the interchange with another freeway near the CBD area (1 to 5 mi away) the preferred one-word messages were either DOWNTOWN or NAME OF A MAJOR ARTERIAL in the area. The same two-word message was preferred or a similar DOWNTOWN--NAME OF THE CITY combination was preferred. In the central city area, the NAME OF MAJOR ARTERIALS are preferred for one-word messages and DOWNTOWN--NAME OF ARTERIAL was the preferred two-word message. In the suburbs either the NAME OF THE SUBURB or the MAJOR ARTERIAL to the city center is used. The term DOWNTOWN is never used in combination with the larger urban area when reference is being made to the downtown of the suburb as it would confuse the motorist.

As motorists approach the central business district (CBD) of a large urban area, the appropriate designators used to guide these motorists may change depending on the motorist's location. As motorists get closer to the downtown area, their preference in terminology may shift from more general terms, such as DOWNTOWN or the CITY NAME, to more specific terminology, such as the NAME OF A MAJOR ARTERIAL leading into the downtown area. The exact location of these changes and the preferred language has not been determined. Another related problem exists when motorists approach suburbs that are surrounded completely by the larger urban area. Motorists usually are not aware when they enter a suburb unless the city limit sign appears on the overhead sign structure. To the unfamiliar motorist, it is very difficult if not impossible to distinguish between when they are in the metropolitan area or a suburb without getting off the freeway and asking. The unfamiliar motorist has no way of knowing whether a particular street will take them to the central business district of a suburb. The findings of a study that addresses these important guidance problems are reported in this paper. The major objectives of the study were to determine (a) the most appropriate terminology for guiding motorists to the downtown or CBD area of large metropolitan areas, and (b) the most appropriate terminology for guiding motorists to the center or downtown area of a suburb.

RESEARCH METHODOLOGY

A scenario laboratory technique was used to conduct this research. Slides were used to present the various messages to 100 test subjects, who were selected on the basis of age, sex, educational background, and whether or not they held a driver's license. The subjects were told they were traveling along prescribed routes to specific destinations. To determine the most appropriate descriptions to use for the downtown area, six one-word messages and six two-word combination messages were presented at each location under investigation. The six one-word messages were: DOWNTOWN, CBD, DENVER CBD, BUSINESS, DENVER, and LAMAR STREET. The six two-word messages were: DOWNTOWN--DENVER, BUSINESS--DENVER, DENVER CBD--LAMAR STREET, BUSINESS--DENVER CBD, DOWNTOWN--LAMAR STREET, and BUSINESS--LAMAR STREET. Each test message was presented on a miniature sign complete with route shield and cardinal direction. A number appeared below each test sign. The test subjects were to indicate the number of the sign they would (a) expect to see at this location, and (b) prefer to see at this location. The locations at which each of the one- and two-word messages were presented are

1. Near the entering city limits,
2. Approaching a major loop around the urban area,
3. Approaching an intersecting freeway near the center of the urban area, and
4. Near the subjects' destination.

Each test sign was projected for 6 sec followed by a 20-sec pause to allow the subjects to respond. The subjects were required to find the test panel and respond by pressing a button that corresponded to the number under the sign panel of their choice. The time required to locate the test sign and respond, as well as the subjects' preferences and expectancies were also recorded.

To investigate the appropriate suburb city descriptors to use, an identical laboratory technique was employed. In two of the four trips through Denver, the subjects' destination was either in a suburb or such that they must travel through a suburb to reach their destination. The subjects were presented a slide showing their destination, the suburb city limit sign, and a sign bridge with four sign panels. These three slides were presented in sequence. Each of the four trials were designed such that the subjects were evaluating four ways of presenting suburb information on the same sign. The types of information directing the subjects to their destinations were (a) suburb city arterial street information (Marion Avenue, Linsay Street) and (b) destination city (Limon, Kansas City) which was on another freeway that passes through a suburb. The presentation of both control cities on the same sign can be confusing to motorists. The test subjects would respond by pushing the button corresponding to the sign they would use to reach their destination.

Another related problem addressed in this study was the presentation of destination information relating to an intermediate destination in the downtown area of the city after the motorists had passed the downtown area when their primary destination is another city. Therefore, information that directs motorists back to the downtown area of the city they had just passed is not expected by the motorist. In this portion of the study, the use of the term DOWNTOWN, the NAME OF THE DESTINATION CITY, and the NAME OF THE URBAN AREA just passed were evaluated.

RESULTS

The results indicated that 69.7 percent of the subjects expected to see the message DENVER and/or DOWNTOWN displayed as they approached the city limits and 61.4 percent of the subjects indicated they preferred the same two-word message at this location. Seventy percent of the subjects were able to choose the correct lane in an average of 5.7-sec response time (Table 1). When the term DOWNTOWN was used, 63 percent selected the correct lane in an average time of 5.7 sec. The use of the terms DOWNTOWN and/or DENVER at this location is strengthened when considering that almost one-half of the subjects (44.6 percent) indicated they expected to see DOWNTOWN--DENVER as a two-word message. Thirty-three percent of the subjects indicated they preferred DOWNTOWN--DENVER as an alternative two-word message at this location. The next closest two-word message was DOWNTOWN--LAMAR STREET, which 18.7 percent of the subjects selected.

As the subjects approached the loop area, 59.8 percent indicated they would expect DOWNTOWN (33.3 percent) and/or BUSINESS (26.5 percent) as the message. Twenty-one percent of the subjects indicated they would also expect to see DENVER at the same location, which means that 81.2 percent of the subjects expected to see either DOWNTOWN, BUSINESS, or DENVER. Sixty-five percent of the subjects indicated they preferred to see DOWNTOWN (35.5 percent), BUSINESS (14.9 percent), and/or DENVER (14.9 percent). Sixty-four percent of the subjects selected the correct lane in an average time of 5.4 sec when the term DOWNTOWN was used, and 59 percent chose the correct lane in 5.8 sec when DENVER was used. Almost three-fourths of the subjects (72 percent) said they would expect the two-word messages when DENVER, DOWNTOWN, and BUSINESS were used in combination, and 43.1 percent said they would prefer the two-word messages in which these three terms were used.

The wide disparity between the messages the motorists expect and those they prefer indicates a shift between driver expectancy and driver preference. Driver expectancy is based on past driving experiences. A portion of drivers' previous driving experience relates to the signing presented, which, in turn, becomes an integral part of each driver's data base and driving set (expectancy). What the drivers learn to expect and what they would prefer to see may be completely different. For this reason, the terms "the drivers expect to see" and "what they prefer to see" are not the same. The results obtained from this study tend to support this initial premise.

As the subjects approached an intersecting freeway leading into the CBD, 70.2 percent indicated they would expect (a) DOWNTOWN (34.6 percent), (b) DENVER (17.8 percent), and (c) LAMAR STREET (17.8 percent). Seventy-four percent indicated they preferred to see (a) DOWNTOWN (22.8 percent), (b) DENVER (13.9 percent), and (c) LAMAR STREET (32.4 percent). Again, this fact is borne out when considering that almost one-half (45.6 percent) of the subjects selected two-word messages that contained the three terms described previously, those they would expect to see, and more than one-half (57.8 percent) of the subjects indicated they would prefer to see these messages at this location. The two, two-word messages were (a) DOWNTOWN--DENVER and (b) DOWNTOWN--LAMAR STREET. When the term LAMAR STREET was used, 53 percent of the subjects selected the proper lane in an average time of 6.8 sec. When DOWNTOWN was used, 47 percent selected the correct lane in 6.4 sec, and when DENVER was used, 37 percent chose the correct lane in 5.9 sec.

At location 4 (the LAMAR STREET exit), 64.2 percent of the subjects indicated they would expect to see LAMAR STREET used and 72.8 percent indicated they preferred to see LAMAR STREET used. At this location, 75 percent of the subjects selected the cor-

TABLE 1 Percentage of Motorists Selecting the Correct Lane and the Average Decision Time Required to Select by Message and Sign Location

	Near City Limits		Near Loop Around City		At Intersecting Freeway near CBD		Near Exit to CBD	
Test Messages	Lane Choice (%)	Decision Time ($\bar{x}$)	Lane Choice (%)	Decision Time ($\bar{x}$)	Lane Choice (%)	Decision Time ($\bar{x}$)	Lane Choice (%)	Decision Time ($\bar{x}$)
Downtown	63	5.7	64	5.4	47	6.4	66	7.2
CBD	62	5.5	45	6.3	41	6.4	39	8.3
Denver CBD	75	5.7	57	6.3	38	8.0	56	7.0
Business	72	7.2	–	–	48	5.9	59	6.2
Denver	70	5.7	59	5.8	32	5.9	38	7.7
Lamar Street	–	–	40	5.4	53	6.8	75	5.7

Note: Dashes indicate lane choice responses and decision times were not obtained because of experimental error.

rect lane in 5.7 sec. When the two-word messages were used, more than one-half of the subjects (70.8 percent) selected one of two messages [DOWNTOWN--LAMAR STREET, (41.6 percent) and the second was BUSINESS--LAMAR STREET (29.2 percent)] as those they would expect to see at this location. These same two messages were selected by 76.1 percent of the subjects as the messages they would prefer to see at this location.

The results of the suburb city descriptors are presented in Tables 2 and 3. These results indicate that there was no significant difference between the CONTROL CITY and the MAJOR ARTERIAL STREET messages. This would indicate that motorists can relate to either type of message when traveling to a specific destination in a suburb city. The subjects were told that there was a street to the downtown section of Sherwood and the name of the street was either Marion Avenue or Linsay Street. With regard to ARTERIAL STREET messages, the message LINSAY STREET 1/4 MILE had a significantly higher number of responses (80) than the message MARION AVENUE EXIT (47). The message providing advanced warning information had a significantly higher response frequency

TABLE 2 Subjects' Preference with Regard to Information Presented in Suburb Within a Metropolitan Area by Chi-Square Significance—Trip 1

Trip #	Category Tested	Messages	Frequency	Chi-Square Significance
1	Individual Messages	I-50 East, Limon, 3/4 ↓ Mile. I-50, Kansas City ↗. Marion Ave ↗. Linsay St., 1/4 Mile, ↗.	63 47 47 80	$Ho{:}f_1=f_2=f_3=f_4$ $\chi^2 = 13.04$ $\alpha = 0.005$
	Control City Messages	I-50 East, Limon, 3/4 ↓ Mile. I-50, Downtown-Kansas City, ↗.	63 47	$Ho{:}f_1=f_2$ $\chi^2 = 2.65$ n.s.
	Arterial Street Messages	Marion Ave., ↗. Linsay St., 1/4 Mile, ↗.	47 80	$Ho{:}f_1=f_2$ $\chi^2 = 8.57$ $\alpha = 0.005$
	Control City Versus Arterial Street Messages	I-50 East, Limon, 3/4 ↓ Mile; & I-50 - Kansas City, ↗. Marion Ave., ↗; and Linsay St., 1/4 Mile, ↗.	110 127	$Ho{:}f_1=f_2$ $\chi^2 = 1.37$ n.s.
	Advanced Warning Versus Immediate Exit Messages	I-50 East, Limon, 3/4 ↓ Mile, & Linsay St., 1/4 Mile, ↗. I-50, - Kansas City, ↗; and Marion Ave., ↗.	143 94	$Ho{:}f_1=f_2$ $\chi^2 = 10.59$ α 0.005

TABLE 3 Subjects' Preference with Regard to Information Presented in Suburb Within a Metropolitan Area by Chi-Square Significance—Trip 2

Trip #	Category	Messages	Frequency	Chi-Square Significance
2	Individual Messages	I-50, Downtown-Kansas City, ↗. I-50, Denver-Kansas City, ↗. I-50 West, Downtown, 1/2 Mile, . I-50 West, Denver, 1/2 ↓ Mile.	40 58 81 78	$Ho{:}f_1=f_2=f_3=f_4$ $\chi^2 = 17.07$ $\alpha = 0.005$
	Downtown Versus Denver Messages	I-50, Downtown-Kansas City, ↗; & I-50 West, Downtown, 1/2 Mile, ↓. I-50, Denver-Kansas City ↗; & I-50 West, Denver, 1/2 ↓ Mile.	121 136	$Ho{:}f_1=f_2$ $\chi^2 = 0.88$ n.s.
	Immediate Exit Message	I-50, Downtown-Kansas City, ↗. I-50, Denver-Kansas City, ↗.	40 58	$Ho{:}f_1=f_2$ $\chi^2 = 3.31$ n.s.
	Advanced Warning Messages	I-50 West, Downtown, 1/2 Mile, ↓. I-50 West, Denver, 1/2 ↓ Mile.	81 78	$Ho{:}f_1=f_2$ $\chi^2 = 0.06$ n.s.
	Advanced Warning Versus Immediate Exit Messages	I-50 West, Downtown, 1/2 Mile ↓; & I-50 West, Denver, 1/2 ↓ Mile. I-50, Downtown-Kansas City, ↗, & I-50, Denver-Kansas City ↗.	159 98	$Ho{:}f_1=f_2$ $\chi^2=14.48$ α =0.005

(143) than the exit direction or gore messages (94). The location of the test sign in relation to the destination to which the subjects were traveling may have biased the subjects in responding more to advanced warning signs than to exit direction signs. The location of the test sign in the slide indicates that the subjects could have continued a little further down the loop before exiting.

The message I-50, DOWNTOWN--KANSAS CITY,↗, in trip number 2 had the worst response rate and the longest response time than that of the other three messages. This indicates that when the term DOWNTOWN is used with a familiar city name, the subjects were confusing the term downtown to mean downtown Kansas City and not downtown Denver. The term DOWNTOWN in all other cases performed well. This means that the term DOWNTOWN should be used alone or with the name of the urban center the motorists are presently in. It should not be used with a familiar city name several miles away.

CONCLUSIONS

In the metropolitan/downtown study, it was determined that at the entering city limits, the subjects both expected and preferred the CITY NAME as the one-word message. The two-word message both preferred and expected was DOWNTOWN--DENVER. As the subjects approached the loop, they expected to see DOWNTOWN or BUSINESS. The two-word message that the subjects both expected and preferred was again DOWNTOWN--DENVER. As they approached the intersecting freeway leading to the downtown area, the subjects indicated they would expect either DOWNTOWN or LAMAR STREET. The subjects responded that at this location, they would expect to see the two-word message DOWNTOWN--DENVER. And as the subjects were approaching their exit on LAMAR STREET, they responded that they would expect and prefer DOWNTOWN--LAMAR STREET as the two-word message. The analysis of variance indicated that the location and the message at each location had a significant effect on the subjects' response times, whereas the messages themselves did not have a significant effect.

In the study investigating descriptors for central areas of suburbs, the subjects' responses for the CONTROL CITY messages were not significantly different than the MAJOR ARTERIAL message for determining any meaningful relationship. The responses for the advanced warning messages were significantly different than those for the exit direction messages. The responses also indicated that there was no significant difference in response rates between the DOWNTOWN messages and the CITY NAME messages. The average response time for the DOWNTOWN messages was 8.7 sec. and 8.66 sec for the CITY NAME messages. The only message in which there were very few correct responses and longer response times was DOWNTOWN--KANSAS CITY. This indicated that the subjects were interpreting their messages to mean downtown Kansas City literally and not downtown Denver. In all other situations the term DOWNTOWN was competitive with the other messages. Thus, DOWNTOWN should be used on a sign panel either alone to refer only to the downtown area of the central city, or in combination with the name of the central city of the metropolitan area.

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Abridgment

Highway Sign Meaning as an Indicator of Perceptual Response

KENNETH A. BREWER, ALICE A. THIEMAN, WILLIAM F. WOODMAN, and LLOYD L. AVANT

ABSTRACT

Semantic differential scaling has been used as a method of evaluating and assessing driver understanding and comprehension of traffic signs in the past. Litigation and other operational pressures on traffic engineering agencies have created an interest in finding a laboratory method for quick and easy estimation of driver performance in processing communication via signs. This paper contains data on research attempts to correlate the meanings assigned to road signs through semantic differential scales. These scales are correlated with drivers' abilities to detect, recognize, and react to road signs. Significant correlations were most often found between meanings attributed to signs in semantic differential scales and the performance of drivers in recognizing signs. No semantic differential scales were found for any sign tested for which a significant correlation existed in detection, in recognition, and in decision-reaction tests. It was concluded that semantic differential scaling has little or no relationship to perceptual response to highway signs by drivers.

During the past decade, tort litigation has made those agencies that are responsible for signing and traffic control of streets and highways very sensitive to the problem of traffic sign effectiveness and driver communication. Although substantial discussion about this heightened sensitivity of state agencies has taken place, the authors' experience has been that local agencies are as much or more affected than state agencies. As engineering organizations have become more interested in examining the fundamental effectiveness of existing and proposed signs, or new applications of existing signs, a concern has arisen as to how testing and evaluation of signs should be carried out.

The typical engineering approach has been to create a prototype and make a pilot plant installation. The design of a sign and test installation on a limited portion of the street and highway system that is suggested by this philosophy has become quite risky as a result of the threat of tort litigation over accidents during testing. Thus, concerns over potential safety hazards inherent in full scale sign testing as well as the potential financial loss during subsequent litigation has increased interest in the laboratory testing of signs.

The Manual on Uniform Traffic Control Devices (1) identifies the generally accepted five basic requirements of an effective traffic control device. Engineering studies can determine whether the need for traffic control devices exists. Traffic enforcement and the judicial process are the primary mechanisms by which road users develop respect for traffic control devices, and likewise, the authors are not concerned with a laboratory method to test respect for traffic control devices. However, it would seem that if laboratory experiments can be conducted that measure differences among signs related to commanding attention, conveying a clear and simple meaning, and giving adequate time for proper response, then much can be learned about the effectiveness of a sign without the necessity of using prototype field testing.

A technique suggested as providing a simple, inexpensive method for evaluating traffic signs is that of the semantic differential (2). The semantic differential technique developed by Osgood et al. assumes that an underlying structure exists for the meanings (semantic context) assigned to elements in a perceived environment (3). Osgood et al. wrote that these underlying or subconscious structures of meanings may be studied by means of a scaling technique similar to a questionnaire. Although Osgood et al. used exploratory factor analysis to find four dimensions of meaning among the set of scales by which the respondents rated a test item, Nunnally has defined analysis validity for each scale (4). Because factor analysis of semantic scales is only a qualitative or arguable assessment of the interaction of scale responses, the authors have chosen for this analysis a portion of their research data set to follow Nunnally and examine each scale separately.

If semantic differential scales of perceived meaning of signs are to be useful in addressing, via laboratory tests, the three basic sign requirements of interest identified previously, then it should be possible to demonstrate some relationship between semantic scales and quantitative tests designed to measure responses to these sign requirements. This paper reports one of a number of analyses performed in the course of a research project funded by the Iowa Department of Transportation Highway Division and demonstrates that caution must be exercised in attempting to extrapolate perceived highway sign meaning into driver response.

EXPERIMENTATION

Three laboratory experiments were designed to test driver responses to a set of 16 signs. The fundamental focus of the research was to examine differences between word legend and symbol legend Stop Ahead warning signs. However, to test the significance and sensitivity of any experimentally determined differences between these signs, it was necessary to incorporate a larger sign set into the design. The total sign set consisted of the 16 signs shown in Figure 1.

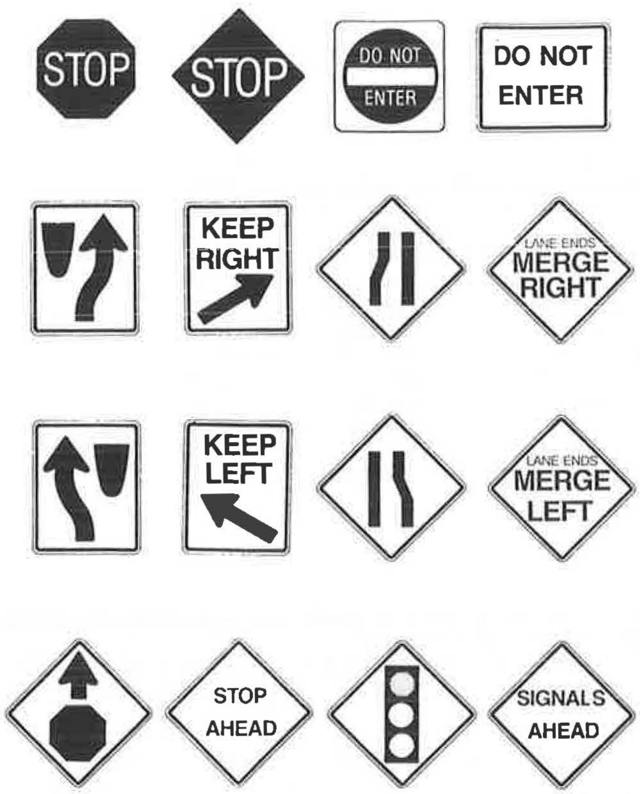

FIGURE 1 Matrix of signs for detection, recognition, and reaction experiments.

Respondents who participated in the experiments described in the following sections were volunteers from undergraduate courses as well as faculty and administrative staff at Iowa State University. Faculty and staff members (16 of 108 persons) ranged from late 30s to early 60s in age. All participants had to possess a valid driver's license. Because the design of the experiments and the testing equipment made potential differences in visual acuity among subjects an irrelevant consideration, no measurement of visual acuity was conducted. Age was not asked of the respondents because a measure of driving experience was obtained (found not to be a significant influence on performance in any of the authors' analyses).

Experiment 1: Detection

A detection experiment was conducted first. Each of 30 persons was presented a series of pre- and post-masked tachistoscopic inputs and asked, after each trial, whether the input was a road sign or a blank flash. Subjects began each trial viewing a mask slide consisting of randomly assembled pieces of various road signs, and the test input for each trial was essentially a brief interruption in the viewing of the mask slide. Each series of trials included presentations of the 16 signs listed previously and 16 blank presentations in a random order. For each subject, the first series of trials began with 110-msec presentations that were clearly visible to the subject. On succeeding series of trials, exposure durations were reduced until the subject performed at no better than chance level in deciding whether each presentation was a blank or a road sign. The criterion of acceptable consistency for a given subject was performance at or below chance level on three consecutive sequences of 32 presentations. Once this criterion was met, three additional series of 32 presentations each were administered to the subject and recorded along with the results of the previous three series.

For each sign, the measure submitted to statistical evaluation was the number of times the sign was correctly detected over the six series at chance-level exposure duration. For the analysis reported here, the probability of correct detection was correlated with semantic differential scale results. The mean chance-level exposure duration for all 30 subjects was 24 msec.

Experiment 2: Recognition

The same sample of 16 signs was used in a second experiment designed to test for differences in recognizability among signs. The experiment was designed to determine whether, after a sign's presence is detected, differences exist in the perceptual operations involved in the recognition process that make the driver aware of the sign. A total of 36 subjects participated in the experiment.

The general procedure was to present the subject a road sign tachistoscopically and then have the subject decide which of the two signs (the just-presented sign and another sign randomly selected from the set) shown outside the tachistoscope in clear vision was the sign that had just been presented. Each trial began with the subject viewing the previously described mask slide; as in the preceding detection experiment, the stimulus presentation was essentially an interruption of the subject's viewing of the mask. The experiment required 240 trials for each subject. This permitted 15 test trials for each sign, that is, 15 trials on which a given sign was presented tachistoscopically and then paired with each of the other signs for the subject's forced choice identification of which sign has been presented tachistoscopically on that trial. The performance measure was the number of errors, of a possible 15, that each subject made. For the analysis reported here, the probability of correct recognition was correlated with the semantic differential scale results.

The 36 subjects were assigned to three groups of 12 subjects each. This made it possible to evaluate the effect of viewing time on sign recognition. Exposure durations were based on the results of Experiment 1 (Detection). Recognition experiment exposure times for Groups 1, 2, and 3 were 32, 41, and 49 msec, respectively. These exposure durations were, respectively, 1, 2, and 3 standard deviations about the mean exposure duration for chance-level presence-absence detection in Experiment 1 (24 msec).

Experiment 3: Decision Reaction Times

This experiment was designed to measure the speed with which subjects could decide on appropriate driver actions for various road signs once the signs were recognized. Forty-eight subjects participated in the experiment. Each subject was provided a response box that housed four response buttons. Respondents were seated in front of a screen onto which road sign slides were projected. At the beginning of the experiment, they were told that road signs would be projected onto the screen and that, for each sign, one of four action decisions would be appropriate. The response decisions would be to stop, to go right, to go left, or to slow down. The subjects were asked to indicate, by pressing the appropriate response button as rapidly as possible, what driver action they would take in response to each of the projected signs.

Proper experimental control required that the assignment of the four response buttons to the four decision actions be varied across subjects. The 48 subjects were accordingly assigned to four groups of 12 subjects each, and assignment of decision actions was counterbalanced across the four groups. As positioned from left to right, the response buttons indicated the following action decisions for the four groups of subjects:

Group	Action Decision
1	Stop, left, right, slow
2	Slow, stop, left, right
3	Right, slow, stop, left
4	Left, right, slow, stop

The performance measure was each subject's mean response reaction time for each sign over 10 randomly ordered presentations of each of the 16 signs. As might be expected, the reversal of decision associated with button position for "go left" and "go right" for Group 3 produced such aberrant values that the results from Group 3 were deleted for this reported analysis.

Semantic Differential Tests

Each subject in the detection, recognition, and decision-reaction experiments was instructed to go to another laboratory to complete a second test. There they were administered the semantic differential scale. Not all subjects did so and the exclusion of subjects in Experiment 3 with reversed left-right response buttons (Group 3) provided 27 subjects from Experiment 1, 35 subjects from Experiment 2, and 23 subjects from Experiment 3 who completed the semantic differential and whose performance could be correlated across the experiments.

To limit the time required in the semantic differential test and minimize subject resistance, the authors decided to utilize only a portion of the complete set of 16 signs. Because the contract focus of the research revolved around the differences between the word and the symbol Stop Ahead signs, both of those were included. Driver behavior using the Stop sign as a "slow" rather than a "stop" driver action was also an issue in the research question, so it was determined that the set of signs to be tested would be the four "slow down" driver action signs and the four "stop" driver action signs.

Twelve 7-point scales were created for each subject to mark in response to each of the eight signs. The extreme ends of each scale were identified with the following pairs of descriptors: good to bad; familiar to unfamiliar; active to passive; predictable to unpredictable; beautiful to ugly; meaningful to meaningless; fast to slow; strong to weak; valuable to worthless; important to unimportant; sharp to dull; simple to complex. These descriptors were selected after consulting original work by Osgood et al. (3) and considering the application previously made by Dewar and Ells (2).

A random number generator was used to select two different sequences of the eight signs to produce slide set A and slide set B to be displayed to respondents. Trial measurements indicated that no more than one person would be expected to be waiting while a subject was participating in the semantic scale test. A random number generator was used to select the order in which the scales were placed on the answer sheet with the same answer sheet being used for all signs viewed and all subjects. Each subject was seated in a room with subdued lighting and shown slides of the previously described eight signs. Each subject was allowed to study each sign as long as he or she wished, but the instructions given at the beginning of the test informed each subject that each scale was to be marked with the first impression about the sign. A randomized order to the scales also included a randomization of the "positive" or the "negative" descriptor as the left end of the scale. The positive end of the scale was given a weight of 7 and the negative end was given a weight of 1 in the data reduction.

RESULTS

It should be pointed out that there were extremely few statistically significant correlations where 192 calculations per table were carried out. In Table 1 there were 18 statistically significant correlations (9.37 percent), whereas in Table 2, only 4 of the correlations were significant (2.08 percent). In Table 3, 10 of 192 possible correlations were significant (5.20 percent), and in Table 4, there were again 10 statistically significant correlations (5.20 percent). Thus, an average 5.46 percent of the possible correlations were statistically significant.

At the same time, the only meaningful patterns of significant correlations were found in relation to the signs bearing the following legends:

- Stop Ahead (symbol)
- Signal Ahead (symbol)
- Stop Ahead (word)
- Do Not Enter (word)

Given that the purpose of the authors' research was to examine formats of the Stop Ahead warning to motorists, the authors found this pattern of findings interesting but puzzling. One possible interpretation of these results might be that all four signs are not seen with great frequency and are likely not thought about when seen. Unlike standard Stop signs that have been so frequently seen that they may have become functionally invisible, these signs may still bear sufficient freshness that they engender responses and meaning attribution. At the same time, the semantic differential scales that generate substantial patterns of correlations (three or more significant correlations) included only active to passive and predictable to unpredictable.

Why these two meaning dimensions would produce these patterns of correlations is also unclear. Given the preceding comments regarding the frequency of sign usage, it may well be that these less frequently seen signs generated both respondent certainty and uncertainty as well as the vitality or robustness of the message contained.

CONCLUSIONS

The basic hypothesis of this research was that tests of perceptual detection, recognition, and action decision latency would correlate with measures of perceived meaning of signs (i.e., that the ability to

TABLE 1 Semantic Differential Scale Correlations with Detection Experiment Results by Sign Shown

	Signal Ahead (Sym)	Signal Ahead (Word)	Stop Ahead (Sym)	Stop Ahead (Word)	Do Not Enter (Sym)	Do Not Enter (Word)	Stop (Oct)	Stop (Diam)
Good - Bad								
Perf Same	--	--	--	+0.39	--	--	--	--
Perf Opp	--	--	--	+0.52	--	--	--	--
Familiar - UNF								
Perf Same	--	--	--	--	--	-0.40	--	--
Perf Opp	--	--	--	--	--	--	--	--
Active - Passive								
Perf Same	+0.40	--	--	+0.37	--	-0.52	--	--
Perf Opp	--	--	--	+0.54	--	-0.52	--	--
Pred - Unpred								
Perf Same	+0.41	--	--	+0.42	--	-0.37	--	--
Perf Opp	+0.50	--	--	--	--	--	--	--
Beautiful - Ugly								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Mean'ful - Mean'less								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Fast - Slow								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	+0.44	--	--	--	--
Strong - Weak								
Perf Same	--	--	--	--	--	-0.48	--	--
Perf Opp	--	--	--	--	--	--	--	--
Val - Worthless								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	+0.40	--	--	--	--
Imp - Unimp								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Sharp - Dull								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	+0.55	--	--	--	--	--	--	--
Simple - Complex								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	+0.43	+0.46	--	--	--	--	--	--

"--" = Not significant at 0.05 or better level.

Perf Same = detection, recognition or decision-reaction performance on sign with same lexical status to legend as the one scaled.

Perf Opp = detection, recognition or decision-reaction performance on sign with opposite lexical status in legend as the one scaled.

32ms and 49ms = milliseconds exposure duration in tachiostoscopic presentation during recognition experiment, etc.

TABLE 2 Semantic Differential Scale Correlations with 32 msec Recognition Experiment Results by Sign Shown

	Signal Ahead (Sym)	Signal Ahead (Word)	Stop Ahead (Sym)	Stop Ahead (Word)	Do Not Enter (Sym)	Do Not Enter (Word)	Stop (Oct)	Stop (Diam)
Good - Bad								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Familiar - UNF								
Perf Same	--	--	--	--	--	--	-0.57	--
Perf Opp	--	--	--	--	--	--	--	--
Active - Passive								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Pred - Unpred								
Perf Same	--	--	--	-0.67	--	--	--	--
Perf Opp	--	--	--	-0.57	--	--	--	--
Beautiful - Ugly								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Mean'ful - Mean'less								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Fast - Slow								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Strong - Weak								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Val - Worthless								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Imp - Unimp								
Perf Same	+0.69	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Sharp - Dull								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Simple - Complex								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--

"--" = Not significant at 0.05 or better level.

Perf Same = detection, recognition or decision-reaction performance on sign with same lexical status to legend as the one scaled.

Perf Opp = detection, recognition or decision-reaction performance on sign with opposite lexical status in legend as the one scaled.

32ms and 49ms = milliseconds exposure duration in tachiostoscopic presentation during recognition experiment, etc.

TABLE 3 Semantic Differential Scale Correlations with 49 msec Recognition Experiment Results by Sign Shown

	Signal Ahead (Sym)	Signal Ahead (Word)	Stop Ahead (Sym)	Stop Ahead (Word)	Do Not Enter (Sym)	Do Not Enter (Word)	Stop (Oct)	Stop (Diam)
Good - Bad								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Familiar - UNF								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Active - Passive								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Pred - Unpred								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Beautiful - Ugly								
Perf Same	--	--	+0.61	--	--	--	--	--
Perf Opp	--	--	--	--	+0.70	--	--	--
Mean'ful - Mean'less								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Fast - Slow								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	-0.73	--
Strong - Weak								
Perf Same	--	--	+0.69	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Val - Worthless								
Perf Same	--	--	+0.63	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Imp - Unimp								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	+0.68	--	--	--
Sharp - Dull								
Perf Same	--	--	+0.61	--	--	--	--	--
Perf Opp	--	-0.87	--	--	--	--	--	--
Simple - Complex								
Perf Same	--	--	+0.66	--	--	--	--	--
Perf Opp	--	--	+0.60	--	--	--	--	--

"--" = Not significant at 0.05 or better level.

Perf Same = detection, recognition or decision-reaction performance on sign with same lexical status to legend as the one scaled.

Perf Opp = detection, recognition or decision-reaction performance on sign with opposite lexical status in legend as the one scaled.

32ms and 49ms = milliseconds exposure duration in tachiostoscopic presentation during recognition experiment, etc.

see and recognize signs in very short time durations was somehow related to semantic differential measures of stored meaning). Data that the authors will report elsewhere clearly show that sign detection, recognition, and action decision latency are all clearly related to sign meaning. However, for this report, the authors computed a total of 1,152 correlations between laboratory tests of perception and 12 semantic differential meaning scales and so few were found to be significant that it is clear that semantic differential measures of attributed meanings of a sign are not systematically related to laboratory tests of the ability to detect, recognize, and decide on driver actions. The clear suggestion of these findings is that the semantic differential, as an adjunct and verification device for laboratory detection/recognition research is of questionable reliability and validity.

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Drivers' Unconscious Errors in the Processing of Traffic Signs

LESLIE A. WHITAKER

ABSTRACT

Human information processing is divided into two processing modes. One is a conscious, attention-demanding method that is flexible and can be readily controlled. The second is an unconscious, essentially uncontrolled processing that is triggered by well-practiced stimulus-response associations. This paper contains a description of two types of errors to which unconscious processing is prone: illusory combinations of display elements and interference from conflicting irrelevant display elements. Traffic guide signs that may be susceptible to unconscious (automatic) processing errors are also presented as well as research results that are consistent with the hypothesized errors.

Unconscious behaviors during driving are a common experience for some motorists. A person may drive a familiar route and arrive at the destination without being aware of the frequent turns and stops along the way. Drivers are most often aware of this unconscious behavior when they have intended to alter a familiar route by stopping, for example, at the golf course or the gas station. Then, they arrive at the office after having missed the intended stop completely.

Well-learned tasks move from attention-demanding ones to the effortless nature of automatic unconscious processing. This processing is susceptible to a different set of problems than is conscious processing. Driving is a task that may be particularly susceptible to the tricks of automatic processing because it is so well-practiced.

Study of the unconscious was an active area of investigation early in this century (1). It fell subsequently into disfavor and it has been only re-

TABLE 4 Semantic Differential Scale Correlations with Decision Reaction Results by Sign Shown

	Signal Ahead (Sym)	Signal Ahead (Word)	Stop Ahead (Sym)	Stop Ahead (Word)	Do Not Enter (Sym)	Do Not Enter (Word)	Stop (Oct)	Stop (Diam)
Good - Bad								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Familiar - UNF								
Perf Same	--	--	--	--	--	-0.55	--	--
Perf Opp	--	--	--	--	--	-0.55	--	--
Active - Passive								
Perf Same	--	--	--	--	--	-0.61	--	--
Perf Opp	--	--	+0.36	+0.44	--	-0.49	--	-0.43
Pred - Unpred								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	+0.42	--	--	--	--	--	--	--
Beautiful - Ugly								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Mean'ful - Mean'less								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Fast - Slow								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Strong - Weak								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Val - Worthless								
Perf Same	--	--	--	--	--	-0.58	--	--
Perf Opp	--	--	--	--	--	-0.45	--	--
Imp - Unimp								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Sharp - Dull								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--
Simple - Complex								
Perf Same	--	--	--	--	--	--	--	--
Perf Opp	--	--	--	--	--	--	--	--

"--" = Not significant at 0.05 or better level.

Perf Same = detection, recognition or decision-reaction performance on sign with same lexical status to legend as the one scaled.

Perf Opp = detection, recognition or decision-reaction performance on sign with opposite lexical status in legend as the one scaled.

32ms and 49ms = milliseconds exposure duration in tachiostoscopic presentation during recognition experiment, etc.

cently that cognitive psychologists have again turned their hand to study the effects of unconscious processing in automatic behavior (2-5). This research has inadvertently produced a significant body of data dealing with automated or nonconscious processing. For example, the development of skill with practice results in a decrease in time, effort, and attention needed to complete a task (6). The consequence is that the skilled operator can apply more cognitive resources to the execution of concurrent subsidiary tasks while well-practiced tasks are run by automated processes (7). The development of time-sharing skills for the dual tasks of copying text while memorizing unrelated material was reported by Spelke et al. (8). After 17 weeks of practice, two subjects learned to execute these two tasks concurrently without interference.

Drivers develop similar time-sharing skills with experience. The new driver concentrates effort on the guidance task, and is later able to add some navigation and route-finding tasks, but only after some weeks of practice can this novice converse comfortably with a passenger while driving through relatively uncongested streets. Even after years of practice, an experienced driver may ask passengers to stop conversing when particularly difficult winter driving conditions exist. The driving task under these adverse conditions again demands full attention. It is no longer sufficient to allocate even the guidance tasks to well-practiced, possibly automated processing.

These anecdotal experiences are supported by recent reports from cognitive psychology. Posner has described automatic processing and emphasized its resistance to change (3). He described conscious processing as requiring attention and as being quite flexible and amenable to change. In contrast, automatic processing places little or no burden on attention but is essentially hardwired (through physiology or practice) and very resistant to modification. For example, Shiffrin and Schneider trained a group of subjects to respond yes to various shapes (6). After 2,100 trials, the subjects could respond to sets containing one or two different items with speed equal to that of larger sets of items. Processing speed was independent of the amount of information to be processed. Subjects reported that the positive stimuli seemed to "jump out of the display." They were not conscious of having to search the display for positive items. These are characteristics that describe automated processing developed by practice. These same subjects subsequently were retrained to respond no to the formerly positive set. Almost 1,000 trials were necessary to remove the previously learned positive response.

After these 1,000 trials, the subjects' response times had returned to baseline. The response times for the new positive set did not yet show the independence of information load (set size) characteristic of automatic processing. Automatic processing is very resistant to change.

Posner and Snyder have defined the criteria for automatic processing: Automatic processing occurs without conscious awareness and without interfering with other concurrent processing activities (9). The results of automated processing may actually interfere with the appropriate response to a concurrent task.

Dewar has tested this problem with traffic signs (10). Prohibited-turn signs are a combination of a directional arrow plus a prohibited symbol (see Figure 1). Dewar argued that the subject's prepotent response to the directional arrow is to respond in the direction of the arrow. The negation (prohibition) of this action is a time-consuming and error-prone process. The prepotent, overlearned response

FIGURE 1 Prohibited left-turn sign and components.

of going with the direction of the arrow interferes with the designated correct response. For permissive signs, arrows indicate the permitted direction(s) and a redundant green circle indicates permission (see Figure 2).

Treisman and her associates have proposed a mechanism that predicts errors when combinations of sign

FIGURE 2 Permitted straight-ahead and right-turn sign and components.

elements (e.g., arrow plus contradiction qualifier) are necessary to interpret the sign's meaning (2,11, 12). These errors are predicted for conditions in which automatic, instead of conscious, processing occurs. In a series of laboratory conditions, Treisman has established that it is the combination of visual features that requires attentional resources for successful processing. When these resources are withdrawn from processing the display, errors result in illusory combinations of separate features. For example, a display containing an array of ┐'s and ╱'s will be seen as an array containing ↗┐'s. In addition, ↗┐ may be decomposed into ┐ + ╱. These errors occur when the subject's attentional resources are withdrawn to a second concurrent task. The automatic processing of the first task's feature components can still be accomplished. However, the correct combination of those features into the displayed objects is impaired without conscious attention-demanding) processing.

An experienced operator seems to be doing much of the driving as an automatic process by the Posner and Snyder definition (9). Therefore, observation and processing of standard highway guide signs may be impaired in ways predicted for the automatic mode. The combination of component features will be vulnerable to error. Some traffic signs are more prone to combinatoric problems by the very nature of their content. For example, the prohibited-turn sign requires the accurate combination of components: arrow plus red circle and a slash. The combination gives the message. Losing either component or combining them incorrectly can lead to driving errors.

At especially difficult intersections, a redundant system is sometimes employed. On the same stanchion, one sign shows the prohibited left turn, whereas the sign immediately below displays a one-way right arrow (see Figure 3). Erroneous combinatorics would be especially disastrous here. It is relevant that this double signing is used to control intersections prone to a left-turn error; intersections where the driver's preconception, preoccupation with other aspects of driving, or road condi-

FIGURE 3 Prohibited turn onto one-way street.

tions make automatic processing the most likely mode to be applied to this sign's perception.

A benefit of automatic processing is that limited attentional resources are made available for processing subsidiary tasks; however, a correlate of this benefit is that automatic processing is not controlled by attention, effort, or direction from the operator. Posner states that automatic processing runs its course from stimulus to response without attentional control (9,13). The input stimulus triggers the automatic processing. If the output of this processing is advantageous for the conscious task, then performance may be augmented. However, if the automatic output is contrary to the goal of the conscious task, then a decrement occurs. A classic example of such interference was originally reported by Stroop (14). Observers were asked to say the color in which a word was printed. For example, the word "great" was printed in red ink so the correct response was "red." In one condition, noncolor words were printed. In the other condition, the names of colors were printed. Color names were never written in the same ink color as their name (e.g., "green" was printed in red ink). Observers responded much more slowly when the words printed were the names of colors than when they were noncolor words. The experimenter had manipulated the task to produce conflict between the automatic process of reading the printed word and the conscious task of naming the color in which the words were printed. When the printed word was in the same category (color) as the response word (ink color), the output of the automatic processing conflicted with the required response of the conscious processing and response time increased.

This "Stroop effect" has been studied for a variety of tasks and is an experimental paradigm used to measure the conflict between unintended automatic processing and the conscious task's intended processing. The Stroop phenomenon for processing traffic guide signs is illustrated in two recent studies by Whitaker and Sommers, and Whitaker, respectively (15,16). In these studies, airport guide signs were used. Each sign consisted of a pictograph (a swept wing jet aircraft) indicating an international airport plus an arrow tab indicating the airport's direction. The direction of the plane symbol and the direction of the arrow could either agree, be orthogonal, or disagree (see Figure 4). Subjects were fastest and produced the fewest errors when the plane and the arrow agreed. Orthogonal pairing produced intermediate performance, and disagreement between plane and arrow produced the worst performance.

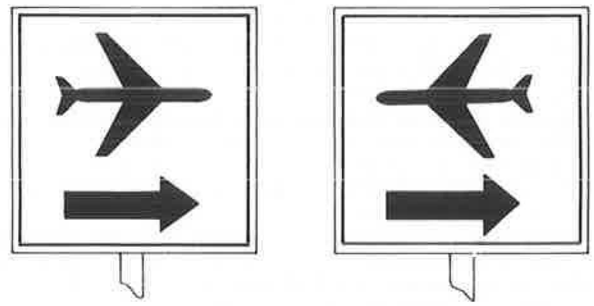

FIGURE 4 Airport guidance signs with airplane symbol plus guidance arrow.

These results were interpreted in the following way. Automatic processing of both components of the sign took place. The strong directional information from the plane symbol augmented the arrow information for agreement signs. Responses to these signs, consequently, were faster than under the baseline (orthogonal) condition. Disagreement between plane and arrow on the bipolar dimension left-right meant that both responses were triggered. The subject had to suppress the incorrect (plane) direction and output the correct (arrow) direction. This conflict produced the poor performance for disagreement signs.

One recent study of traffic accidents concluded that human error was implicated as the definitive cause of 70 percent of the accidents. Half of these errors were information processing failures of perception or comprehension [Treat (17)].

This paper concentrates on possible sources of error specific to the automatic processing mode. This mode (in combination with attentional information processing) is a mode frequently used for well-learned, highly practiced tasks such as driving. Research has provided helpful guidelines for signing standards and current guidelines are based on knowledge of conscious information processing. In establishing traffic signing standards, it will also be beneficial to be aware of the unique pitfalls produced by our unconscious (automatic) cognitive behaviors.

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Abridgment

Recognition Errors Among Highway Signs

LLOYD L. AVANT, KENNETH A. BREWER, ALICE A. THIEMAN, and WILLIAM F. WOODMAN

ABSTRACT

Forced choice recognition errors were examined for tachistoscopic presentations of four sign messages (Stop, Go Right, Go Left, Slow Down) displayed in word versus symbol format. Sign exposure durations were 1, 2, and 3 standard deviations (32, 41, and 49 msec) above the mean exposure duration for chance-level presence or absence detection of a traffic sign in the visual field (24 msec). As exposure duration increased, recognition errors decreased more rapidly for Stop message signs than for other messages. Word versus symbol format differentially influenced reductions in recognition errors for Right, Left, and Slow messages but had little influence on errors on Stop message signs. Several pairs of signs were shown to be reciprocally confused with each other, and Merge Right signs were frequently confused with signs presenting three different action messages. For the signs tested, those that are likely to produce recognition errors resulting in accidents were identified as well as those for which recognition errors are unlikely to produce accidents.

The present research was prompted by two major concerns. One concern was the pragmatic concern of civil engineers interested in effective traffic signing to safely guide traffic flow. The second was the theoretical need to discriminate between (a) the purely perceptual operations performed by the brain in extracting sign information and (b) the mental operations involved in driver actions that occur after the recognition process is completed.

The research was initiated by a focus on the failure of drivers to recognize and properly respond to the symbol legend Stop Ahead standard sign W3-1a

(1). The specific circumstance indicating the urgency to examine these issues involved the intersection of two paved county trunk highways in Buena Vista County, Iowa. The highways cross at right angles in rolling terrain. The north-south route is Stop sign-controlled, and east-west traffic is through traffic. Signing of the intersection is clearly visible to drivers approaching from all four directions. North- and westbound traffic encounter a visual obstruction in the southeast quadrant of the intersection, making it imperative that drivers approaching from the south obey the Stop sign on that leg of the intersection. Soon after new symbol legend Stop Ahead signs were erected to precede the Stop signs, a number of accidents occurred that involved failures of drivers to respect the Stop signs. This unexpected increase in accident frequency prompted the County Highway Engineer to request research to more clearly differentiate the factors that cause such accidents. Reported in this paper is a portion of the data from that research.

EXPERIMENTATION

Introduction

The pragmatic concern that initiated the research focused on potential differences in the effectiveness of the word and symbol versions of the Stop Ahead advance warning sign. However, considerations of proper experimental designs dictated that a larger sample of signs be studied, and the set of 16 signs shown in Figure 1 were selected.

Three laboratory experiments were conducted. Experiment 1 tested the effects of these signs on drivers' detection of sign presence or absence in the visual field when tachistoscopic exposures of the signs reduced overall detection performance to chance level. Experiment 2 increased exposure durations above detection level and investigated sign recognition errors as time for the recognition process increased. Experiment 3 measured the time required for deciding what driver action was appropriate for each sign. Reported in this paper is a portion of the data from the second experiment and an interpretation of the recognition error patterns for traffic engineering purposes.

Procedure

The intent of the experiment was to determine whether or not the 16 test signs produced differences in the perceptual operations that extract sign information and generate conscious recognition of the signs. Respondents who participated in the experiment were 36 volunteers from undergraduate courses, faculty, or administrative staff at Iowa State University. All respondents were licensed drivers. Tests of visual acuity were not conducted because (a) the authors' concern was to obtain a representative sample of Iowa drivers rather than a sample of drivers with 20/20 visual acuity, and (b) the experimental design and testing equipment made differences in visual acuity an irrelevant consideration. Age of respondents was not asked because a measure of driving experience was obtained (and found not to be a significant influence on performance in any of our analyses).

The general procedure was to tachistoscopically present two road signs to the subject and then have the subject decide which of the two (the just-presented sign and another sign) shown outside the tachistoscope in clear vision was the sign presented on that trial. Each trial began with the subject viewing a mask and slide consisting of randomly assembled pieces of traffic signs, and the sign presentation was essentially an interruption of the subject's viewing of the mask. The experiment required 240 trials for each subject. This permitted 15 test trials for each sign. The performance measure was the number of error choices, of a possible 15, that each subject made for each sign.

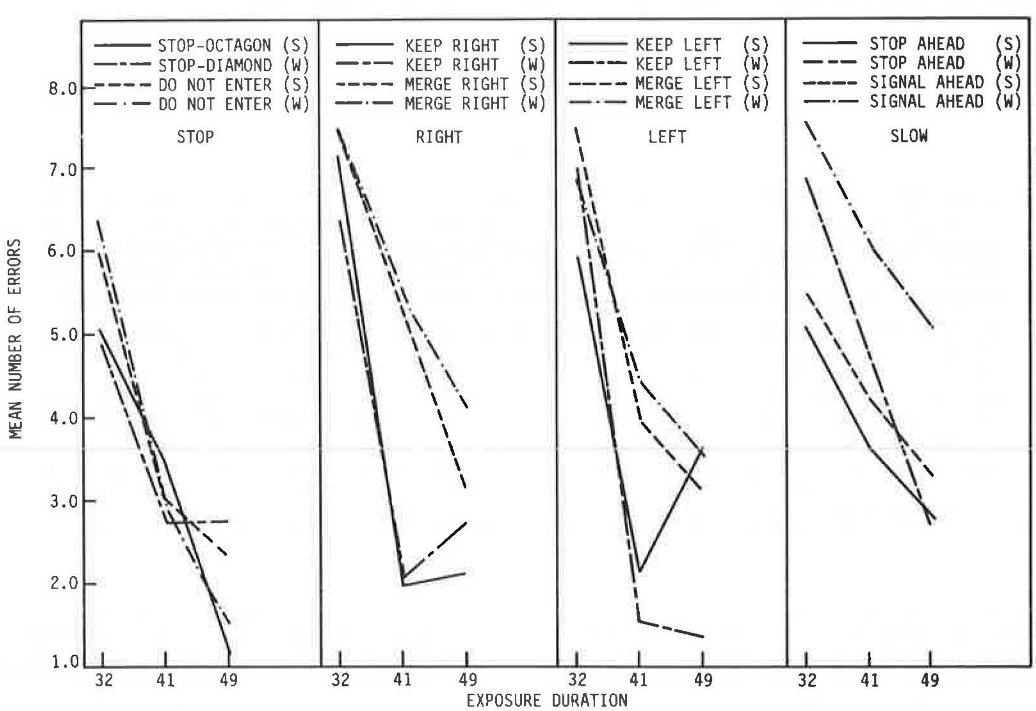

FIGURE 1 Sign recognition errors.

The 36 subjects were assigned to three groups of 12 subjects each, and exposure durations differed for the three groups. Exposure durations were based on the results of the detection experiment (Experiment 1). For Groups 1, 2, and 3, exposure durations were 32, 41, and 49 msec, respectively. These durations were, respectively, 1, 2, and 3 standard deviations above the mean exposure duration for chance-level presence or absence detection in Experiment 1 (24 msec). This manipulation permitted evaluation of the influence of sign message (Stop, Go Left, Go Right, Slow Down) and format (word versus symbol) on reducing recognition errors as time for completion of the recognition process increased.

RESULTS AND DISCUSSION

Most simply stated, the results of this experiment showed that the perceptual operations performed in recognizing highway signs differ considerably among signs. The message presented by the sign, the symbol versus word format of the sign, and exposure duration all interacted in determining number of recognition errors. This complex interaction is summarized graphically in Figure 2. However, findings of pragmatic concern were clear in the data.

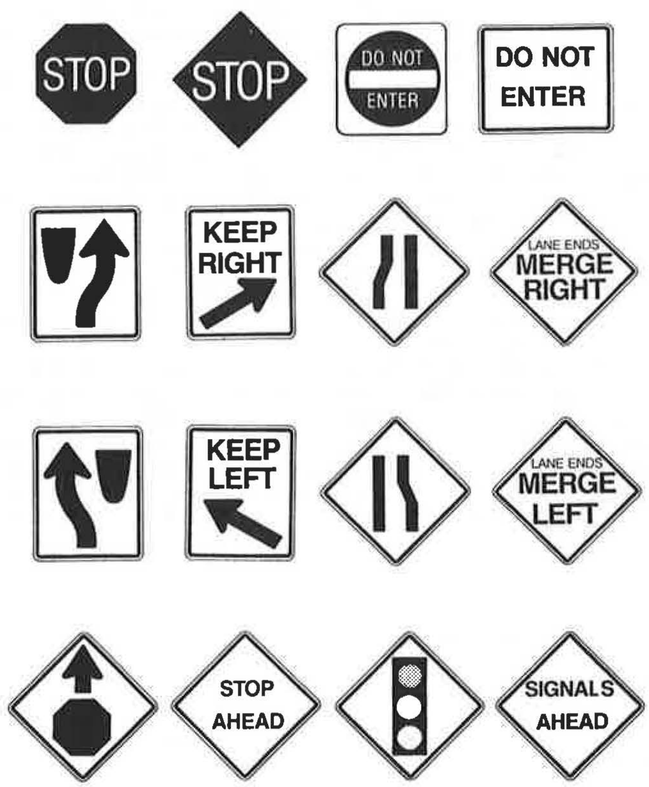

FIGURE 2 Matrix of signs for detection, recognition, and reaction experiments.

As expected, the number of recognition errors decreased as exposure duration increased, and most of the reduction in errors occurred as exposure duration increased from 32 to 41 msec; further reduction in errors when exposure duration increased from 41 to 49 msec was not significant. The important implication here is that the perceptual operations of sign recognition are completed rapidly, and the action decision triggered by those perceptual operations occurs in a time period that is likely to be less than 50 msec. A second finding of practical interest was that fewer recognition errors were made for signs that instruct a driver to stop than for signs that instruct a driver to go right, go left, or slow down. This result conformed to the result from Experiment 1, reported elsewhere (2), showing that even when overall presence or absence detection performance was at chance level, Stop message signs were detected more accurately than were signs instructing a driver to go right, go left, or slow down.

These findings are, in general, evident in the data presented in Figure 2. Inspection of Figure 2 also reveals informative differences in the patterns of error reductions for Stop, Go Right, Go Left, and Slow Down sign messages. For Stop action message signs, errors declined in about the same fashion for Stop and Do Not Enter signs whether they were symbol or word format signs. For Go Right action and Go Left action signs, similar patterns of error reduction were evident. As exposure duration increased, the number of recognition errors decreased more rapidly for Keep Right (or Left) signs than for Merge Right (or Left) signs, and there was little difference between word and symbol signs. For Stop Ahead signs, fewer errors were made for symbol signs than for word signs when the exposure duration was 32 msec but, when exposure duration was increased to 49 msec, the number of errors for both word and symbol signs had reduced to about the same level. The implication is that the symbol version of the Stop Ahead sign can be more readily recognized if viewing time is extremely limited, but if sufficient viewing time is available, both word and symbol Stop Ahead signs can be recognized equally well. For Signal Ahead signs, fewer recognition errors were made for symbol signs at all three exposure durations.

The authors examined these data more closely to determine the types of confusions among signs that occur during perceptual analysis of the various signs. For the three groups of 12 subjects who were tested with 32-, 41-, and 49-msec presentations, the authors calculated the mean number of subjects who incorrectly chose, for each presented sign, each of the other 15 signs in recognition errors. A 99 percent confidence interval about each of those means was then calculated. Signs for which the number of subjects making recognition errors exceeded that confidence interval were identified as signs producing either significantly larger or significantly smaller than average numbers of errors.

Table 1 summarizes the evidence for significantly high numbers of errors. The extreme left column identifies the 16 signs presented for identification. The column headings of the table identify, for the 32-, 41-, and 49-msec test exposures, the message of the sign that was given in the error response. The numbers presented in the body of the table identify the specific sign that was given in an incorrect response.

At least three kinds of important information can be extracted from Table 1. First, one can identify the signs for which confusions were reciprocal--that is, signs that were confused with each other irrespective of which sign was the presented test sign and which sign was the error choice. For 32-msec test presentations, the following signs were reciprocally confused:

Stop Ahead (word)--Do Not Enter (word)
Stop Ahead (word)--Keep Left (word + symbol)
Merge Right (word)--Do Not Enter (word + symbol)
Merge Right (word)--Merge Right (symbol)

TABLE 1 Sign Pairs Producing High Error Rates

	Error Choice Message											
	32 msec				41 msec				49 msec			
Sign No.	Stop	Right	Left	Slow	Stop	Right	Left	Slow	Stop	Right	Left	Slow
1	2				2				2			15
2		9	8,11			5	12					
3	1			13		6						
4	2	9							2			
5	1,2		12	16						6		
6	2	10	7	14,16							12	
9	1,3,4	6	11	13	1,3	5,6	7,11	13,16	1	6	11	13
10	2	10	8	14,16	4		8,12	14		10		14
7	2	10		13								
8										5,6,9		13
11	4	10	8	13	3,4	9		13		9	7,8	13,16
12	3	19	7	13,15,16		10	7,8		2			13,14
13	3,4	5,6	7,8	14		6,9	7,8	15,16	1	6,9	11	
14							7		4		7,8,11	
15	2,3,4		11	13,14,16	2,3	6,9	7,8	13,14,16	3,4	6,9	8	13,14,16
16	4					10	8,12					13,14

When test exposures were 41 msec, the following signs were reciprocally confused.

Merge Right (word)--Merge Left (word)
Merge Right (symbol)--Merge Left (symbol)
Stop Ahead (word)--Merge Right (word)
Stop Ahead (word)--Signal Ahead (word)

When test exposures were 49 msec, the following signs were reciprocally confused.

Merge Right (word)--Merge Left (word)
Stop Ahead (word)--Merge Right (word)
Stop Ahead (word)--Merge Left (word)

The next important problems that these findings address are determination of (a) which recognition errors are likely to produce incorrect driver actions and (b) which ones are not likely to be dangerous. This is determined in part by the reciprocal confusions between pairs of signs noted earlier. The Left-Right message signs provide a particularly useful example. For all three test exposures, signs that instruct a driver to either Merge or Keep Right or Left were reciprocally confused with each other, and the confusions occurred with both the word and symbol legend signs. In fact, the reciprocal confusions appear to identify Merge Right signs as particularly troublesome. Drivers appear to have particular difficulty in recognizing these signs; Merge Right signs were involved in 7 of the 11 reciprocally confusing sign pairs noted above, and they were reciprocally confused with 5 different signs among which three different messages were presented. It is also important to notice that confusions involving Left-Right messages were not much affected by viewing time.

Some of the other signs were also frequently given in error responses, but these error choices are unlikely to produce dangerous driver actions. For example, the standard octagonal Stop sign (MUTCD R1-1) was given in a number of error responses, but those responses were to other signs that instruct a driver to stop. These errors may indicate that even when the driver is uncertain about which of several possible signs was shown, enough sign information has been extracted to communicate the Stop message, and the driver chooses the sign that presents that message most clearly.

The format of Table 2 duplicates that of Table 1 but summarizes the evidence on signs that prompted significantly lower than average numbers of error choices. These data indicate that Stop message signs were least frequently confused with signs presenting other action messages; the next-least-frequently confused signs were those that instruct a driver to

TABLE 2 Sign Pairs Producing Low Error Rates

	Error Choice Message											
	32 msec				41 msec				49 msec			
Sign No.	Stop	Right	Left	Slow	Stop	Right	Left	Slow	Stop	Right	Left	Slow
1	4	5,6,9	8,11,12	14,16	3,4	5	7,11					
2	1	6,10	7	14,15,16		9	7,11			5,10	8,12	13,15,16
3		10		15	1,4	9,10		13,15,16	4	5,10		16
4	1		7,12		3		8,10	11	3			
5	4	9	11	14	1,3,4	9,10	11	15		10		15
6	3	5			1,2,3,4	10		14,15,16			11	16
9							12					
10						5		13		9		
7				16	1,2	6,8	9,10	13,14,15,16			11,12	13
8	3,4		12	15	2,4	5,9	11	13,15,16				
11				16		5						
12	2		11		2,3							
13			12	15	1,2					10		15
14	1	5,9	8,12	15	3,4	5						
15	1	6										
16			7,11,12	13								

slow down and be cautious. The least frequently given error choices were the Signal Ahead symbol sign (MUTCD W3-3), the Signal Ahead word sign (MUTCD W3-3a), and the Merge Left word sign (MUTCD W9-2).

CONCLUSIONS

Based on these data, the authors have concluded that

1. Driver errors in recognizing signs once a sign is detected in the visual field are lower for signs that require a stop action by the driver than those that require a driver to either slow down or move laterally. This finding implies that failures to respond to Stop message signs are likely due to factors other than perceptual operations.
2. Errors in recognizing signs decrease sharply with very small increases above threshold presence or absence detection exposure durations. Errors in perceptual recognition operations are likely to occur within the first 50 msec of viewing time after which recognition errors tend to level off.
3. The formats of some signs tend to produce many recognition errors with other sign messages (Merge Right) whereas other signs infrequently occur in recognition errors (Signal Ahead).

ACKNOWLEDGMENT

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The opinions, findings, and conclusions expressed in this paper are solely those of the authors.

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Abridgment

Restraint Usage at Child Care Centers

DANIEL S. TURNER, MICHAEL C. ROBERTS, and WILLIAM J. WHALEN, Jr.

ABSTRACT

A pilot project was conducted by the University of Alabama to study child restraint usage at two child care centers. Rewards were used to encourage parents to transport their children in approved safety devices, and the usage characteristics were examined. A traditional ABA study (baseline--intervention--return-to-baseline) indicated that usage increased from 48 to 72 percent at one center, and from 11 to 54 percent at the other center. These results soundly demonstrated that psychological learning theory was extremely effective in increasing safety seat usage. A major thrust of the project was the study of pertinent characteristics of parents and children. Age, sex, arrival time, vehicle type, and place in vehicle were found to influence restraint use. Overall, the pilot study provided a sound beginning for an intensive program to increase child restraint usage.

This paper outlines an innovative study conducted at the University of Alabama. A lottery reward system was investigated as a potential way to enhance restraint usage. Parents at child care centers were the target group for the study. The goals for the project were to (a) test a hypothesis, based on psychological learning theory, that positive reinforcement increases restraint use and removal of reinforcement decreases use; (b) develop data collection procedures during a pilot study; and (c) examine characteristics that affect restraint use.

RESEARCH TECHNIQUE

Study Sites

Daycare centers were designated as the study location because of the abundance of children in the targeted age group and the direct contact with parents. Two private centers were identified after an analysis of 20 competing locations. The two served different socioeconomic groups. Center 1 catered to professional parents, whereas Center 2 was heavily subscribed by blue collar families.

Methodology

A traditional ABA (baseline, reward intervention, and return-to-baseline) procedure was designated for the study. The research steps were implemented sequentially. While the B phase was underway at Center 1, the A phase was underway at Center 2 to serve as a control group.

During the intervention period, the parents of properly restrained children received rewards from a lottery pool. Adequate data were gathered to track the increase in usage by individual parents. During the return to baseline, rewards were suspended and the decrease in usage was observed. After the study was completed, several follow-up observations were conducted to monitor any long-term effects.

Data Collection

Parents and children were observed during the morning arrival period to determine whether the children were properly restrained. Observers were volunteers recruited from undergraduate psychology and engineering classes. They were trained to record 19 data items, including each vehicle's license number and the sex of the driver (to permit tracking of individual drivers over time), time of day, estimated age of the child, type of restraint (if any), and other pertinent data.

There were approximately 3,500 observations made continuously across the three phases at the two centers during a 9-week period. Because of the difficulty in visually ascertaining the foregoing criteria, the observers were necessarily noticeable to the parents. Observers sometimes found it difficult to determine whether a restraint was used correctly without peering into the vehicle.

Data Reliability

The data were collected by volunteer undergraduate students under the guidance of the principal investigators. Because the observers rotated shifts, and portions of the data were difficult to gather, there was some concern that the data might not be consistent. On 11 randomly selected days, second observers gathered duplicate sets of data to compare with the findings of the primary observers. Reliability was defined as the number of times the observers agreed divided by the total observations. Of the 11 comparisons, all were strong except for a single day. This one day's data were vastly weaker than the other data, and were dismissed from the data set.

A second reliability check was made by comparing known ages of children (obtained from the centers) with ages estimated by students. Ten cross-checks were made, with nine exhibiting strong correlation. Again, one day's data were much weaker than the others and were dismissed from the data set.

Reward (Incentive) Procedure

The rewards were coupons and gift certificates from various businesses in the community. A lottery system was used to dispense these gifts. Parents with properly restrained children (compliant with state law) were allowed to draw a token on their arrival at the center. Parents removed a Happy Face sticker from the token to determine whether they had won. Gift certificates were then given to those with winning tokens. The observers at the centers had no prior knowledge of the win or no-win status of the tokens. After the reward phase, the return to baseline was made without any tokens being given to parents, although observation of compliance continued. No prior announcement was made concerning cessation of the tokens.

RESULTS

The prominent concern was the ratio of compliant to noncompliant parents during all research phases for each daycare center. Figures 1 and 2 show the percentage of compliance for each weekday during the study. The average compliance percentage for baseline at Center 1 was 48.7 percent, with the percentage increasing during the reward period to 72.7 percent. Compliance decreased to an average of 69.8 percent during the return phase. Follow-up showed 59.6 percent compliance 2 to 3 weeks later, and 60.0 percent 3 months after the return to baseline.

For Center 2, the mean percentage of compliance for the baseline was 11.3 percent, increasing to 54.0 percent over the reward period. Compliance declined to 44.8 percent during the return-to-baseline phase. Follow-up observations conducted 2 weeks later disclosed that compliance had dropped to 17.9 percent. Additional data taken 3 months after the return to baseline indicated a compliance rate of 18.8 percent.

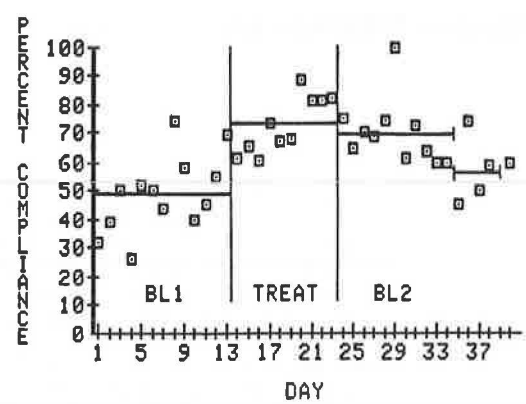

FIGURE 1 Percentage of children in child restraints at day care Center 1.

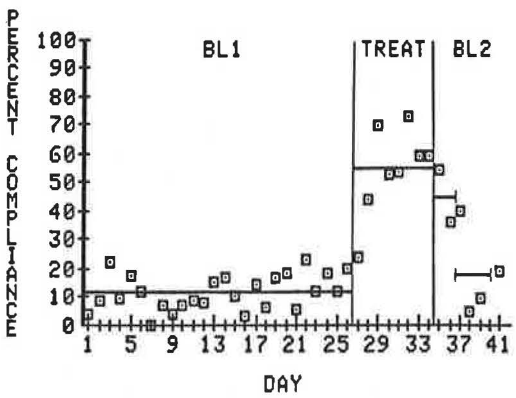

FIGURE 2 Percentage of children in child restraints at day care Center 2.

Rewards were extremely helpful in encouraging parents to "buckle up" their children. During the reward phase and immediately afterwards, the compliance rates were higher at both centers than those typically achieved by other intervention programs. With reinforcement, the desired behavior increased in frequency. After removal of reinforcement, the rate of compliance began to decay. (The rate of decay and the ultimate decay have not yet been determined.)

The findings of this study are significant for at least two reasons. First, the dramatic increases in compliance demonstrate the effectiveness of rewards for increasing safety behavior. Second, there are important health implications for reducing injuries and deaths through increased restraint usage for children. No other studies have utilized rewards for child restraint usage, and few studies can demonstrate such immediate increases in seat-belt usage regardless of the intervention.

CHARACTERISTICS OF STUDY POPULATION

The primary objectives of the project were met when the research proved that child restraint usage could be increased through a reward program. To define why these changes occurred, and to identify the characteristics of the parents and children who were influenced by the rewards, a more detailed analysis was conducted.

Ages of Children

Based on 502 observations at Center 1, the mean age was 2.21 years, with a standard deviation of 1.28 years. At Center 2, the mean age was 3.10 years with a standard deviation of 1.27 years, based on 655 observations. The ages approximated normal distributions at both locations.

Age Versus Compliance

The data clearly showed that younger children are more apt to be restrained than older children. This may be noted in Figure 3. Regression equations were fitted to the baseline data, with the curves displaying negative slopes. This confirms the negative age-compliance relationship. Data taken during the intervention period indicated a similar tendency. These equations were weighted to reflect the number of observations of each age group:

Center 1, baseline:

$$C = 84.7 - 25.1(A) + 3.0(A^2) \qquad R^2 = 0.91 \qquad (1)$$

Center 1, reward:

$$C = 75.3 + 5.3(A) - 2.5(A^2) \qquad R^2 = 0.74 \qquad (2)$$

Center 2, baseline:

$$C = 26.0 - 7.5(A) + 0.8(A^2) \qquad R^2 = 0.94 \qquad (3)$$

Center 2, reward:

$$C = 70.2 - 17.9(A) + 3.3(A^2) \qquad R^2 = 0.73 \qquad (4)$$

where

C = compliance (%),
A = age (years), and
R^2 = the coefficient of multiple determination.

Regardless of the location, rate of compliance, or project phase, the youngest children had the highest rates of compliance.

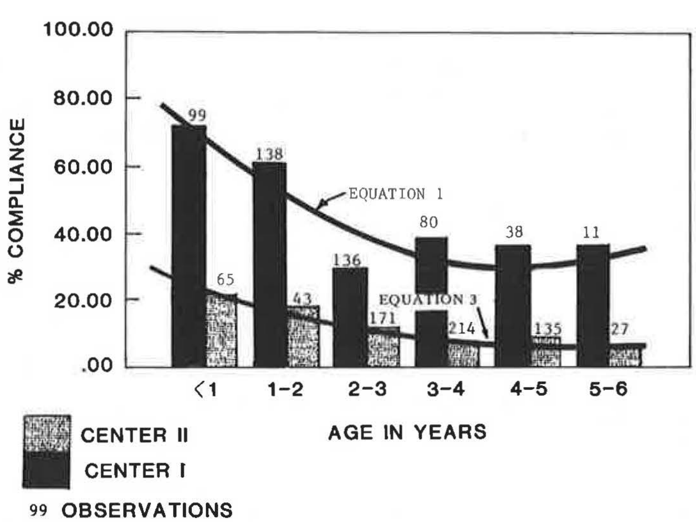

FIGURE 3 Age distribution of compliant children.

Time of Day

During their debriefing, the student observers indicated that compliance was better during some portions of the day than others. In particular, parents who seemed in a hurry or late to work usually did not have their children restrained. Early in the day (7:00 a.m.), extremely high compliance was noted, with the rate dropping as work time approached. Parents who arrived during the 7:30-7:45 a.m. period had the lowest rate of restraint usage, and seemed to be in the biggest hurry.

During the reward phase, there was some improvement from 7:00 through 7:30 a.m., but the major change occurred from 7:40 a.m. until the end of the observation period. This indicates that parents are more responsive if they have a more leisurely trip to the child care center. These findings suggest that intervention methods may not need to include fear appeals, educational material, or rewards, but may simply require the parents to work toward better time management.

Sex of Parent

Compliance by sex of parent and child is shown in Table 1. Female drivers are more likely to "buckle up" their children than are male drivers. In 1,015 occurrences, females had properly restrained their children 42 percent of the time. Male drivers had restrained their children in 35 percent of the 519 occurrences.

TABLE 1 Compliance Versus Sex

	Child's Sex				
	Male		Female		
Parent's Sex	Percent	No. of Observations	Percent	No. of Observations	Center and Study Phase
Male	35	63	41	66	Baseline, 1
Female	52	143	40	112	
Male	8	131	10	63	Baseline, 2
Female	9	137	13	217	
Male	59	80	80	51	Reward, 1
Female	70	145	66	121	
Male	44	41	50	24	Reward, 2
Female	67	61	56	79	
Male	31	315	40	204	All Observations
Female	47	486	37	529	

The second conclusion that may be drawn from the table is that parents appear partial to children of the opposite sex. For example, mothers took better care of sons than daughters (47 to 37 percent). Fathers favored daughters over sons (40 to 31 percent). These conclusions were based on all observations during the baseline and reward periods, and the sex preference was found to be significant by the chi-square test at a 95-percentile level.

Place in Vehicle

The location of children within the vehicle was examined for any consistent patterns. During the baseline phase, 69 percent of the children were in the front seat, 30 percent were in the rear seat, and slightly over 1 percent were in other locations. There were almost no changes in these ratios during the reward period (65 percent, 35 percent, less than 1 percent).

Two-thirds of the baseline children were in the front seat, but those in the back seat were twice as likely to be restrained. During the reward phase, front-seat compliance increased but back-seat children were still restrained at a higher rate. The ages of children did not seem to affect compliance by location. The various groups were found to be almost equally allocated among the seating positions.

Vehicle Type

Restraint usage was related to the type of vehicle. Passenger cars had the best rates, followed by station wagons, pickups, and other vehicles including vans, respectively. Some vehicles may be more conducive to using safety seats than others. For example, the open area in the back of a van or station wagon is a natural play area for a child; consequently, children may be more reluctant to get into safety seats.

It is interesting to note that compliance shifted during the reward phase and became almost uniform regardless of vehicle type. Automobiles were at 65 percent, station wagons were at 69 percent, and pickups were at 63 percent. Apparently, incentives can overcome the inconvenience associated with certain types of vehicles.

FINDINGS

A pilot study was conducted to determine whether or not rewards would increase child restraint usage at two child care centers. The findings were as follows:

1. A data collection procedure utilizing volunteer university students as observers was shown to be very effective.
2. The reward procedures were extremely effective in increasing restraint usage. Restraint usage jumped from 48.7 to 72.7 percent at one location, and from 11.3 to 54.0 percent at a second location. This behavioral change conforms to psychological reward theory.
3. Younger children were more likely to be restrained than older children. This was clear at both centers, in both the baseline and reward periods.
4. Restraint usage was found to be related to time of day. Parents who arrived before or after the rush period exhibited higher levels of compliance.
5. Parents were more apt to restrain children of the opposite sex. Fathers favored daughters, and mothers favored sons.
6. Most of the children arrived seated in the front; however, those in the back seat were twice as likely to be restrained.
7. The type of vehicle influenced restraint use. Automobiles had the highest rates of use followed by station wagons and pickups, respectively. The reward mechanism overcame this bias.
8. Implementation of a general program of this nature could be inexpensive and could provide a substantial health benefit.

IMPLEMENTATION OF FINDINGS

The pilot program's initial success may imply future application in naturalistic settings (e.g., drive-up bank windows, gas stations, etc.). If the rate of behavior change was completely understood, it would be possible to design a reward program to achieve large, long-lasting changes in restraint usage. Refresher rewards could be issued periodically to boost decaying usage rates back to higher levels.

Before widespread implementation of reward programs, further research must be performed. Tests must be conducted to define specific rates of change, when to use refreshers, the effects of socioeconomic status, reward ratios, maintenance levels, and other parameters.

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Optimal and Minimal Luminance Characteristics for Retroreflective Highway Signs

MICHAEL SIVAK and PAUL L. OLSON

ABSTRACT

Presented in this paper are optimal and minimal sign luminance recommendations based on a review of available applied research. Optimal recommendations are based largely on peak luminance-legibility relationships. In the absence of other criteria, minimal recommendations are based on performance levels of 6 m/cm (20/23) for younger persons and 4.8 m/cm (20/29) for older persons. By using a computer sign legibility model, calculations were then made to determine the photometric characteristics of signing material required to obtain the values indicated.

One of the more significant questions facing any traffic agency is the optimum and replacement level for retroreflective signs. Caught up in this question are issues of safety, efficient movement of traffic, and costs. Because these are such important issues, a great number of investigations have been conducted to determine guidelines. The purpose of this paper is to review a selected portion of this research, and summarize the recommendations.

The review included experimental investigations pertaining to the legibility of a message on a sign constructed of retroreflective materials. Studies concerned with the relative merits of illuminated and retroreflective signs, as well as those dealing with nonlegibility issues (e.g., detection, color recognition, conspicuity, and comprehension), address a different set of problems and thus are beyond the scope of this review. Only applied research--whether on the road or in the laboratory--is covered. Purely basic research is not included.

As a first step in this work, a review of the literature was carried out. A total of 18 experimental studies were finally included. [See the original report for the detailed reviews of these studies (1).] Tabular reviews of each paper were prepared to facilitate a comparison of methods, findings, and recommendations. A synthesis of these data was prepared and will be presented in the next section.

A SYNTHESIS OF EXPERIMENTAL FINDINGS

A synthesis of the findings of the past research in terms of optimal and replacement (minimal) luminance values is provided in this section. The two most common sign types will be considered--a sign with a nonreflective black legend on a reflective light background, and a fully reflectorized sign with a white legend. In arriving at luminance recommendations, we will use geometric means to minimize the effects of extreme values.

The retroreflectance values required to achieve the desired luminance levels will be derived in the next section. The computations of the recommended luminance (and retroreflectance) values will be based on data collected under generally ideal conditions, such as signs placed in dark environments, sober observers, and clean signs. Therefore, in a later section, several variables that contribute to the argument for higher luminance values will be listed, along with some correction factors.

Optimal Luminance

Legibility is generally an inverted U-shaped function of luminance (2-4). Thus, determining optimal luminance for legibility purposes should be relatively easy: The optimum is at the crest of the function. Unfortunately, the issue is not that straightforward. The problem is that there exists a variety of inverted U-shaped functions, one each for all combinations of legend-background contrast, complexity of competing visual environment, age of the observer, and so forth. The state of the art is not advanced enough to deduce the parameters of all of these relevant inverted U functions. As a consequence, in reaching a synthesis regarding the optimum, we were forced to average over all relevant parameters. Furthermore, in a study by Allen and Straub, the crest of the function for black legend on light background was apparently not reached even with the highest tested luminance level (5). Nevertheless, the highest tested level in that study was used in averaging with optimal values from other studies that did find an asymptote or a decrease in legibility with the highest levels tested (4,6).

Black Legend on Light Background

The following studies have relevant luminance recommendations or findings for the situation where only the background luminance is appreciably greater than 0.

Luminance Value (cd/m^2)	Study Characteristics
343.0	Allen and Straub laboratory study (5)--an asymptote was apparently not reached even with the highest level tested
34.3	Allen et al. field study (7)--dark rural (used both 100% and 75% legend/background luminance contrast)
60.0	Dahlstedt field study (8)
206.0	Hind et al. laboratory study (6)--the data appear to asymptote at 206 cd/m^2)
55.0	Olson et al. laboratory study (4)--(recommended luminance: 10-100 cd/m^2)
24.0	Smyth laboratory study (9)

The geometric mean of these values is equal to approximately 75 cd/m^2. As a result, the recommended optimal luminance of a white, orange, or yellow background with a black legend is 75 cd/m^2.

Fully Reflectorized Signs

For fully reflectorized signs, as Olson et al. (4) pointed out, the optimal luminance of one component varies with the luminance level of the other component. As a consequence, for fully reflectorized signs, analogous computations were performed on the contrast findings.

Contrast Value	Study Characteristics
9.5:1	Forbes et al. laboratory study (10)--recommended range of 6-13:1
3.0:1	Forbes et al. field study (10)--light legends
7.5:1	Hills and Freeman laboratory study (11)--recommended minima of 8-10:1 for red, 7:1 for green, and 6-7:1 for blue
45.0:1	Olson et al. laboratory study (4)--recommended 30-60:1 for signs in the currently typical luminance range
12.9:1	Sivak et al. field study (12)--best performance at 10-15.8:1
21.0:1	Sivak and Olson field study (13)--best performance at 9-33:1

The geometric mean of these values is equal to approximately 12:1. As a result, the recommended optimal legend-background contrast for fully reflectorized signs is 12:1. For example, if the background luminance is 1 cd/m^2, the optimal luminance of the legend should be 12 cd/m^2.

Replacement Luminance

"Replacement (minimal) luminance" implies the point at which the sign is failing to fulfill its nighttime function. From many points of view, replacement luminance is more important than optimal luminance. There is, unfortunately, no consensus concerning what the minimum function of a sign is. If, for example, there was some agreement concerning minimum legibility distance, then determining the luminance levels required to achieve it would be relatively simple. Lacking such guidelines, luminance-legibility relationships might be expected to be found that would suggest a cut-off point. If, for example, there was a luminance-contrast level below which legibility dropped off very rapidly, this might then serve as an obvious minimum level. However, there does not appear to be such a discontinuity in the available data.

No criteria now exist for establishing minimal sign luminance levels that are likely to meet with wide acceptance. In the absence of such criteria, the replacement level recommendations presented here are based on the following legibility levels: 6 m/cm (50 ft/in.) of letter height for studies that use exclusively younger observers, younger and older observers, or if the observers' age was not reported; and 4.8 m/cm (40 ft/in.) of letter height for studies that use exclusively older observers. The rationale for the selection of these criteria is as follows:

1. 6 m/cm corresponds to visual acuity of approximately 20/23 (14). This value is close to the usually found average visual acuity for younger and middle-aged persons. [In one of the most comprehensive studies on this topic, Burg (15) found the average visual acuity of 16,137 persons between 16 and 64 years of age to be 20/20.] Furthermore, 6 m/cm is frequently used as a legibility criterion.
2. 4.8 m/cm corresponds to visual acuity of approximately 20/29. This value is close to 20/26, the average visual acuity obtained by Burg (15) for a sample of 1,301 persons between 65 and 92 years of age. [By combining the 16,137 persons between 16 and 64 years of age with the 1,301 persons between 65 and 92 years of age, Burg found the average visual acuity to be approximately the same as the average visual acuity for the subsample of persons between 16 and 64 years of age. (This is a consequence of the relatively few older persons who enter the averaging process for the combined sample.) Therefore, the same criterion was used for studies using

either exclusively younger subjects, or younger and older subjects.]

The following are findings relevant to the issue of replacement luminance (values shown are for the lighter component, whether legend or background):

Replacement Value (cd/m^2)	Study Characteristics
3.00	Allen and Straub (5); white and black backgrounds; estimated from their Figure 7 for Series C letters; criterion: mean at 6 m/cm (young observers)
2.00	Allen (16); black background; estimated from his Figure 8; criterion: mean at 6 m/cm (younger and older subjects)
6.90	Allen et al. (7); white and black backgrounds; estimated from their Figure 11; criterion: mean at 4.8 m/cm (older observers)
2.00	Hills and Freeman (11); green, blue, and red backgrounds (of up to about .3 cd/m^2); estimated and averaged from their Figures 6 through 8; criterion: mean at 6 m/cm (observer age unspecified)
1.30	Olson et al. (4); green and red backgrounds (of up to about .4 cd/m^2), as well as white, yellow, and orange backgrounds; estimated and averaged from their Figures 1-29 through 1-33 and 1-35; criterion: 50% correct at 4.8 m/cm (older observers)
0.93	Richardson (17); various backgrounds; criterion: mean at 6 m/cm (young observers)
4.60	Smyth (9); white and black backgrounds; criterion: mean at 6 m/cm (observer age unspecified)

The geometric mean of these values is equal to approximately 2.4 cd/m^2. As a result, the recommended replacement luminance of the lighter component is 2.4 cd/m^2. This recommendation applies to light backgrounds (white, yellow, and orange) with black legends, and to white legends with dark (green, blue, red, or brown) backgrounds having background luminance of up to 0.4 cd/m^2. [As the luminance of the background increases above 0.4 cd/m^2, the replacement luminance of the legend is dependent on the particular level of the background luminance (4)].

RETROREFLECTANCE CONSIDERATIONS

The technical term for retroreflectance is "coefficient of retroreflection," symbolized by R' (ASTM E 808, Standard Practice for Describing Retroreflection). In metric form, R' is defined as $cd/lux/m^2$. Knowing the luminance levels required to achieve a given objective, it is desirable to determine the retroreflectance required. This could be done in several ways. One is to use existing data, such as that of Woltman and Youngblood (17), and extrapolate from their measurements. In this case, calculations of the retroreflectance values were made by using a computerized nighttime sign legibility model. This program was developed by the University of Michigan Transportation Research Institute (UMTRI) for the 3M Company (4). The model accepts a great number of input parameters (e.g., sign location, retroreflective materials, color, headlamps, road geometry, background characteristics, and viewing distance) and predicts legibility distance. For this task, the input values were optimal and replacement luminance values derived earlier; candela values derived from U.S.- and European-type low-beam headlighting systems; an assumed legibility distance of 183 m (600 ft); and four sign locations designated as follows: right shoulder [2.4 m (8 ft) up, 2.4 m (8 ft) right], left shoulder [2.4 m (8 ft) up, 6 m (20 ft) left], shoulder guide [2.4 m (8 ft) up, 10.7 m (35 ft) right], and overhead [6 m (20 ft) up, 0 m (0 ft) right]. (The right shoulder and shoulder guide locations were measured from the right edge of the lane, the left shoulder from the left edge of the lane, and the overhead from the center of the lane.)

The calculated optimal and replacement retroreflectance values for signs placed in dark surrounds are given in Tables 1 and 2. The optimal values in Tables 1 and 2 apply to signs having light (white, yellow, and orange) backgrounds with black legends. For fully reflectorized signs, the optimal retroreflectance of one component (legend or background) depends on the given retroreflectance of the other component. For these signs, the optimal contrast value of 12:1 derived earlier can be used to obtain an approximation to the optimal retroreflectance of one component from the known retroreflectance of the other component. For example, if the background retroreflectance is set at 2 $cd/lux/m^2$, the corresponding optimal retroreflectance of the legend is 24 $cd/lux/m^2$.

The replacement values in Tables 1 and 2 apply to signs placed in dark surrounds. These values apply to light backgrounds (white, yellow, and orange) with black legends and to legends of fully reflectorized signs having backgrounds of up to 0.4 cd/m^2. (As the luminance of the background increases above 0.4 cd/m^2, the replacement luminance of the legend is dependent on the particular level of the background luminance (4).

The replacement luminance values derived earlier were based on mean data, which are likely to be in the neighborhood of the 50th percentile, and in one instance, on the 50 percentile (4). However, 75th and 85th percentile estimates would also be desirable. Consequently, Tables 1 and 2 list the corresponding sign luminance and retroreflectance values for the 50th percentile performance, as well as for 75th and 85th percentiles, which were obtained from the 50th percentile values by using factors of 3 and 7. These factors were estimated and averaged from Olson et al. (4) by using their data for signs with green and red backgrounds of up to

TABLE 1 Optimal and Replacement Coefficients of Retroreflection ($cd/lux/m^2$) when Using U.S.-Type Low-Beam Headlighting Systems[a]

		Sign Location			
Level	Sign Luminance (cd/m^2)	Left Shoulder	Overhead	Right Shoulder	Shoulder Guide
Optimal	75	2,806	3,547	736	856
Replacement percentile					
85th	16.8	630	798	168	189
75th	7.2	270	342	72	81
50th	2.4	90	114	24	27

[a]The optimal values apply to white, yellow, and orange backgrounds of signs with black legends. (For fully reflectorized signs, the optimal legend-to-background contrast is 12:1.) The replacement values apply to white, yellow, and orange backgrounds of signs with black legends, and to legends of fully reflectorized signs with backgrounds of up to 0.4 cd/m2. The listed optimal and replacement values apply to generally ideal conditions; for possible correction factors, see Table 3.

TABLE 2 Optimal and Replacement Coefficients of Retroreflection (cd/lux/m^2) when Using European-Type Low-Beam Headlighting Systems[a]

Level	Sign Luminance (cd/m^2)	Sign Location			
		Left Shoulder	Overhead	Right Shoulder	Shoulder Guide
Optimal	75	4,644	7,252	2,436	1,113
Replacement percentile					
85th	16.8	1,043	1,624	546	252
75th	7.2	447	696	234	108
50th	2.4	149	232	78	36

[a]The optimal values apply to white, yellow, and orange backgrounds of signs with black legends. (For fully reflectorized signs the optimal legend to background contrast is 12:1.) The replacement values apply to white, yellow, and orange backgrounds of signs with black legends, and to legends of fully reflectorized signs with backgrounds of up to 0.4 cd/m^2. The listed optimal and replacement values apply to generally ideal conditions; for possible correction factors see Table 3.

0.4 cd/m^2 and for signs with white, yellow, and orange backgrounds.

CONTRIBUTING VARIABLES

The derivations of the optimal and replacement luminances presented earlier were based on data collected with sober subjects under low-luminance surround conditions. As a result, the derived values are probably conservative. Table 3 gives several variables that contribute to the argument for higher luminance values, along with some corresponding correction factors. (The listed factors were derived from the cited references.)

TABLE 3 Contributing Variables and Correction Factors

Contributing Variable	Correction Factors	
	Optimal Value	Replacement Value
High-luminance surround and environmental glare *(7)*	20x	20x
Driver age *(12)*	–[a]	–[b]
Truck drivers: observation angle *(19)*	2-5x	2-5x
Alcohol intoxication *(20)*	–[a]	–[a]
Dirty signs *(21)*	1.2-20x	1.2-20x
Dirty headlamps *(22)*	<1-10x	<1-10x
Misaligned headlamps *(23)*	–[a]	–[a]

[a]Data unknown.
[b]The effect of driver age on replacement values can be considerable; however, it is highly specific to the set of conditions used. In addition, older drivers have shorter legibility distances and therefore have less time in which to act on the information in the sign message.

CONCLUSIONS

In this study applied research on sign legibility was reviewed to obtain information on optimal and replacement luminances of retroreflective traffic signs.

The legibility data reviewed suggest that for signs that have light (white, yellow, and orange) backgrounds with black legends placed in low luminance surrounds, the optimal luminance of the background is approximately 75 cd/m^2. For fully reflectorized signs, the optimal luminance of one component depends on the given luminance of the other component. The data suggest that for these signs the optimal legend to background contrast is about 12:1.

By assuming legibility criteria of 6 m/cm of letter height for younger subjects and 4.8 m/cm for older subjects, the reviewed legibility data suggest that the replacement luminance value is 2.4 cd/m^2. This applies to light legends with dark (green, blue, red, and brown) backgrounds of up to 0.4 cd/m^2, and to light (white, yellow, and orange) backgrounds with black legends. By using these optimal and replacement luminance values, optimal and replacement retroreflectance values for commonly used colors of retroreflective materials were derived in Tables 1 and 2 for signs in four different locations, illuminated by U.S. or European low-beam headlighting systems. The present recommendations were derived by averaging a set of values from studies run under generally favorable conditions. As a result, several variables that contribute to the argument for higher luminance values were listed in Table 3, along with some correction factors.

This review dealt only with legibility issues. However, luminance contributes to other functional properties of traffic signs, including conspicuity and ease of color recognition. Thus, the compromises that led to the legibility-based recommendations must be supplemented with compromises based on other criteria applicable to traffic signs. An issue of consequence is how the minimal recommendations can be used by traffic agencies in their replacement programs. This is a difficult question because although equipment for measuring retroreflectance in the field exists, it is not practical for regular measurements on large numbers of signs. Until a more convenient means can be developed, the simplest way is probably to rely on time-related performance data, either from manufacturers or from the agency's own experience.

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Freeway Lighting and Traffic Safety—A Long-Term Investigation

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ABSTRACT

The objective of this study was to assess the effectiveness of freeway lighting. To achieve this, a case study on traffic accident characteristics was conducted that utilized a suburban freeway area west of Frankfurt, Federal Republic of Germany, between 1972 and 1981. The study revealed that (a) the effects of lighting on suburban freeway accident rates was positive--there was a reduction in accidents, and (b) these positive results of continuous freeway lighting were lost in the case of partial lighting, especially after switching off lights at night between 10:00 p.m. and 5:30 a.m. for the purpose of saving energy.

Sufficient reliable documentation is available to support the assumption that nighttime traffic accident rates are considerably higher than daytime rates. For example, the National Safety Council has determined that in both urban and rural areas, the mileage death rates at night are at least 4 times higher than death rates during the day (1). Also, in the Fatal Accident Reporting System (2), it was reported that

> There is a distinct pattern of fatalities by time of day and day of week. A greater number of fatalities occurred from 4:00 to 8:00 p.m. on weekdays and from midnight to 4:00 a.m. on weekends. The weekends, however, had the highest concentration of fatalities [during nighttime].

There are many reasons for the unbalanced night accident rate. The following are but a few of the factors that cause driving at night to be hazardous (3).

1. The average person is poorly equipped to see adequately at night. This problem becomes more serious as a person grows older. For example, the glare resistance of the over-65-year-old driver is one-third that of the 25-year-old driver (4). Persons who are 60 years old require 8 times as much light as those 20 years old. Therefore, many of the driving assignments that involve such factors as speed and roadway conditions become more difficult and hazardous to most drivers when confronted with darkness (5).
2. The physical condition of the average driver must be recognized. Fatigue, drowsiness, influence of alcohol, and psychological aspects all have a definite influence on an individual's driving ability, especially at night (3).
3. There is a lack of understanding by many motorists and pedestrians regarding the hazards of night driving.

Many experts generally agree that the use of lighting is justified as a safety improvement because it reduces the frequency as well as the severity of accidents (6). In addition, it provides peace of mind to the traveling public, affords protection to pedestrians, reduces crime, and enhances street appearance (3).

A literature review revealed positive effects of lighting on reducing the frequency, as well as the severity of accidents on urban streets, regular highways, and at intersections. With regard to the effects of freeway lighting on traffic safety, however, mixed opinions were observed. Although several studies concluded that continuous freeway lighting reduces nighttime accident rates and that lighted freeways have significantly lower accident potential than unlighted ones, other studies more or less indicated that continuous freeway lighting did not reduce nighttime accidents.

CASE STUDY

To obtain a better understanding of the effect of lighting, an analysis was conducted from 1972 to 1981 of traffic accident characteristics on a suburban freeway (see Figure 1) area west of Frankfurt,

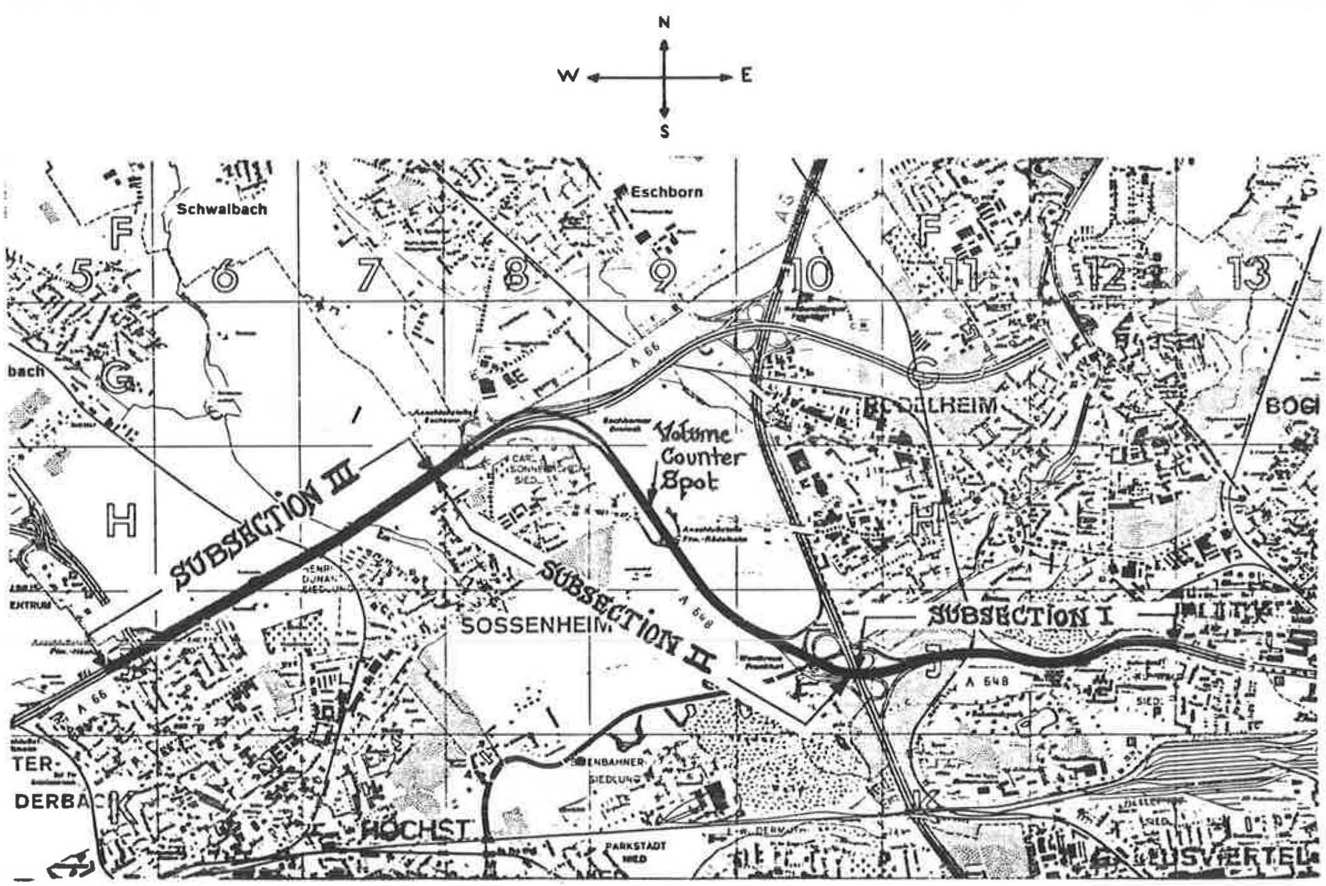

FIGURE 1 Plan of the freeway subsections under study, west of Frankfurt, Federal Republic of Germany.

Federal Republic of Germany (7). Although the freeway section (German Autobahn A648/A66) is heavily developed along the right-of-way, it has a full control of access. Between the interchanges, the divided freeway consists of two lanes in each direction (lane width 3.75 m) with an additional emergency lane (lane width 2.5 m) and unpaved shoulders (width 1.5 m) on each side. The median is approximately 3 m wide and contains a double blocked-out metal beam barrier and a fence to minimize headlight glare. On part of the freeway section under study, lighting devices were installed in 1973. Typical suburban traffic was dominant on this section. (Note: No major construction zones or lane closures were present at the time the study was conducted.)

It is stated in the German Standard (DIN 5044, Part I) that:

> Freeways (Autobahnen) with a traffic volume of 900 vehicles per hour or more and a speed limit of $V \leq 110$ km/h (as is the case of the present study), should be provided with fixed highway lighting with an illuminance level (Nennleuchtdichte) of 1 candela per square meter and a uniformity ratio (Laengsgleichmaessigkeit) of 0.7.

The uniformity ratio is defined as the minimum illuminance level divided by the maximum illuminance level. In a telephone conversation with a representative at the Ministry of Economy and Technique of the state of Hessen, Federal Republic of Germany, in early 1985, it was confirmed that the previously stated lighting values were maintained during the entire study period.

THE INVESTIGATED FREEWAY SECTION

The freeway section studied was divided into three subsections: two were lighted and the third was unlighted for a parallel study. The three freeway subsections under study are shown in Figure 1. Subsection 1 (a) is 1.9 km long; (b) is equipped with cable-suspended luminaires (high-pressure sodium, 250 W) at heights of 12 m, 20 to 21 m apart, on poles installed in the middle of the median and protected by longitudinal barriers; and (c) consists of relatively flat curves with radii $\geq$ 1000 m that correspond to a design speed of 120 km/h (75 mph) (8). Subsection 2 (a) is 3.7 km long; (b) is equipped with cable-suspended luminaires, as described under subsection 1, until approximately the volume counter spot (see Figure 1) after which high mast lighting with luminaires (400 W) are mounted on poles at heights between 25 to 31 m; and (c) consists of flat curves until the volume counter spot and then of curves of minimum radii of 600 m that correspond to a design speed of 100 km/h (62.5 mph) (8).

Because radii $\geq$ 600 m do not substantially affect changes in operating speeds, the effect of the horizontal alignment on the accident situation may be excluded (9). Following subsections 1 and 2 is an unlighted straight subsection 3 that is 2.3 km long.

ASSUMPTIONS AND DEFINITIONS

The period of investigation from 1972 to 1981 was divided into the categories shown in the following table:

Investigation Period	Duration
Before	December 5, 1972 to December 4, 1973
After 1	December 5, 1973 to November 14, 1978
After 2	November 15, 1978 to November 14, 1981

During the before period (B), all subsections were unlighted. During the first after period (A1), subsections 1 and 2 were lighted from dusk to dawn, and subsection 3 was unlighted. During the second after period (A2), subsection 1 was lighted from dusk to dawn and subsection 2 was lighted from dusk until 10:00 p.m. Between 10:00 p.m. and 5:30 a.m., subsection 2 was unlighted. From 5:30 a.m. until dawn, subsection 2 was lighted only if necessary. In summary, the lighting conditions on the investigated subsections are given in Table 1.

TABLE 1 Lighting Conditions on the Three Subsections Investigated

Subsection	B	A1	A2
1	●	O	O
2	●	O	Δ
3	●	●	●

Note: ● = unlighted, ○ = lighted, and Δ = partially lighted.

It should be noted that in the following text the term "day" (dawn to dusk) means the period between the morning after sunrise until the evening before sunset. The term "night" (dusk to dawn) means the period between the evening after sunset until the morning before sunrise.

The basis for the investigation was accident reports filed by the police over a 9-year period. Overall, 1,899 accident reports were surveyed. Approximately 52 percent of the accidents occurred in the east-west direction, and 48.3 percent occurred in the west-east direction. No vehicle-pedestrian accidents were observed on the study section. Accident types that will not be analyzed in this study because of the limited data base can be broken down as follows:

Type of Accident	Total Occurrences (%) Day	Night
Run off the road	18.5	37.1
Rear-end collisions	31.3	19.4
Passing collisions	5.1	6.0
Changing lanes	17.5	13.6
Merge, diverge collisions (only related to the through-traffic lanes, not the ramps)	15.3	10.4
Others	12.4	13.4

Another factor considered is the vehicle kilometers of travel (VKT) produced on the freeway subsections investigated. The average daily traffic (ADT) values were calculated from the yearly data collection of traffic volumes conducted by the federal government (see the following table):

	Subsection 1	Subsection 2	Subsection 3
Before (B)	53,400	51,500	64,700
After 1 (A1)	59,600	54,200	69,000
After 2 (A2)	53,500	48,500	77,100

For a clear comparison of day versus night accident developments (that do not necessarily conform to the definition given in the police records), the exact times of sunrise and sunset were determined for each day of the year. Thus, for each month of the year, the percentages of the VKT could be calculated for day and night. For example, the data in

TABLE 2 Determination of Day and Night Vehicle Kilometers of Travel for a Typical Year in the Investigated Period at the Volume Counter in Subsection 2

HOUR	JAN %	FEB %	MARCH %	APR %	MAY %	JUNE %	JULY %	AUG %	SEP %	OCT %	NOV %	DEC %
11 p.m.	1.61	1.51	1.41	1.30	1.41	1.45	1.49	1.54	1.63	1.72	1.82	1.72
10 p.m.	1.79	1.70	1.60	1.51	1.99	1.97	1.95	1.93	1.94	1.96	1.97	1.88
9 p.m.	2.21	2.20	2.20	2.19	2.25	2.19	2.13	2.07	2.12	2.17	2.23	2.22
8 p.m.	3.39	3.24	3.09	2.94	3.26	3.28	3.30	3.32	3.44	3.57	3.69	3.54
7 p.m.	5.17	4.91	4.65	4.39	4.62	4.68	4.74	4.80	5.09	5.38	5.68	5.43
6 p.m.	6.64	6.54	6.43	6.33	6.47	6.46	6.45	6.43	6.57	6.71	6.85	6.75
5 p.m.	7.81	7.79	7.77	7.74	7.80	7.91	8.03	8.15	8.05	7.95	7.85	7.83
4 p.m.	8.40	8.47	8.54	8.61	8.05	8.23	8.37	8.60	8.49	8.38	8.27	8.34
3 p.m.	6.64	6.66	6.68	6.69	6.59	6.57	6.55	6.53	6.55	6.57	6.60	6.62
2 p.m.	5.87	5.87	5.86	5.85	5.52	5.58	5.64	5.70	5.76	5.82	5.88	5.88
1 p.m.	5.48	5.46	5.44	5.43	5.31	5.34	5.37	5.40	5.43	5.46	5.50	5.49
Noon	4.98	5.04	5.10	5.16	5.12	5.10	5.08	5.06	4.99	4.92	4.85	4.91
11 a.m.	4.76	4.87	4.98	5.09	5.02	5.02	5.02	5.02	4.86	4.70	4.54	4.65
10 a.m.	5.25	5.35	5.45	5.55	5.17	5.18	5.19	5.20	5.15	5.10	5.04	5.14
9 a.m.	5.71	5.82	5.93	6.03	5.79	5.69	5.59	5.49	5.49	5.50	5.50	5.60
8 a.m.	7.13	7.02	6.91	6.80	6.84	6.90	6.96	7.01	7.12	7.23	7.35	7.24
7 a.m.	8.16	8.16	8.15	8.15	8.42	8.36	8.30	8.24	8.22	8.20	8.17	8.17
6 a.m.	5.94	6.22	6.50	6.78	6.65	6.34	6.02	5.70	5.59	5.48	5.38	5.66
5 a.m.	1.27	1.36	1.45	1.53	1.63	1.56	1.48	1.40	1.29	1.19	1.09	1.18
4 a.m.	0.32	0.32	0.33	0.33	0.44	0.41	0.38	0.35	0.34	0.33	0.31	0.31
3 a.m.	0.19	0.19	0.19	0.19	0.22	0.21	0.20	0.19	0.19	0.19	0.19	0.19
2 a.m.	0.27	0.28	0.28	0.28	0.27	0.28	0.30	0.31	0.30	0.28	0.27	0.27
1 a.m.	0.44	0.45	0.46	0.46	0.43	0.48	0.53	0.58	0.53	0.48	0.42	0.43
Midnight	0.72	0.70	0.69	0.67	0.72	0.81	0.90	0.98	0.91	0.84	0.76	0.74
Night-	38.12	23.08	11.70	6.93	5.04	5.20	5.37	7.95	12.69	23.59	30.66	38.49%
Day-	61.88	76.92	88.30	93.07	94.96	94.80	94.63	92.05	87.31	76.41	69.34	61.51%

Legend: ——— change between dawn and dusk, respectively between dusk and dawn.

Table 2 indicate that for a typical year, 38.5 percent of the VKT occurred at night during the month of December, whereas only 5.2 percent of the VKT occurred at night during the month of June. On the average, for the 9-yr period investigated, 81.1 percent of the VKT occurred during the day compared with 19.9 percent at night (7).

DEVELOPMENT OF THE ACCIDENT SITUATION

The analysis of the accident situation on the freeway section investigated is based on the total number of accidents as follows:

	Total Accidents					
	Subsection 1		Subsection 2		Subsection 3	
Period	Day	Night	Day	Night	Day	Night
B	61	30	102	46	62	19
A1	148	77	316	121	266	114
A2	55	39	141	85	144	73

The corresponding accident rates (accidents per 10^6 VKT) are given in Table 3, and the accident cost rates for all accidents (German Marks per 100 VKT) are given in Table 4.

TABLE 3 Accident Rates (per 10^6 VKT) During the Different Time Periods for Day and Night Conditions on the Three Subsections Investigated

	Subsection 1			Subsection 2			Subsection 3		
	Day	Night	N/D Ratio	Day	Night	N/D Ratio	Day	Night	N/D Ratio
B	2.03	4.28	(2.1)	1.83	3.33	(1.8)	1.41	1.85	(1.3)
	x	x		x	x		0	0	
A1	0.88	1.97	(2.2)	1.08	1.66	(1.5)	1.13	2.08	(1.8)
	x	0		0	x		x	0	
A2	0.62	1.76	(2.8)	0.90	2.18	(2.4)	0.93	1.89	(2.0)

Note: x = significant at the 95 percent level of confidence, 0 = nonsignificant at the 95 percent level of confidence, and N/D = night/day ratio of accident rates.

Before discussing the analysis, it is important to note that the rate for personal injury accidents has decreased steadily in the Federal Republic of Germany since 1972. Factors that may have contributed to this decrease include the energy crisis of 1973-1974, the introduction of a general speed limit of 100 km/h on 2-lane rural roads in 1973, the introduction of a strict anti-drunk driving law, which set a BAC at 0.08 percent as the intoxication level in 1973, the mandatory use of seat belts in front seats, and the mandatory safety-helmet law for motorcyclists that was put into effect in 1976 (10).

TABLE 4 Accident Cost Rates (German marks per 100 VKT) During the Different Time Periods for Day and Night Conditions on the Investigated Three Subsections

	Subsection 1			Subsection 2			Subsection 3		
	Day	Night	N/D Ratio	Day	Night	N/D Ratio	Day	Night	N/D Ratio
B	4.90	15.99	(3.3)	4.07	8.40	(2.1)	2.66	6.25	(2.3)
A1	1.98	5.85	(2.9)	2.55	4.42	(1.7)	3.17	5.30	(1.7)
A2	1.32	3.44	(2.6)	1.58	4.33	(2.7)	1.69	3.18	(1.9)

Note: N/D ratio is the night/day ratio of accident cost rates, and \$1.00 (U.S.) corresponds to about 2.7 German Marks (GM). For the calculation of the accident cost rate, the following assessments of personal injuries were applied: fatally injured—500,000 GM, heavily injured—55,000 GM, and lightly injured—10,000 GM. The costs of property damage accidents were compiled from police records.

A decrease in the accident rate on the investigated freeway section should therefore be expected, aside from the installation of lighting devices. For example, the following table, which shows the development of the rate for personal injury accidents on the investigated three subsections and on the whole interstate (freeway) network system in the Federal Republic of Germany, clearly supports the assumption.

	B	A1	A2
Interstate network	0.36	0.23	0.20
Subsection 1	0.86	0.29	0.16
Subsection 2	0.56	0.32	0.22
Subsection 3	0.31	0.29	0.19

Also, it is interesting to note that on the one hand, the accident rates on the unlighted subsection 3 for the years 1973, 1976, and 1980 nearly correspond to that on the whole interstate (freeway) network system in the Federal Republic of Germany, while on the other hand, the continuously lighted subsection 1 showed the strongest decrease from 0.86 to 0.16, followed by the partially lighted subsection 2 from 0.56 to 0.22.

SUBSECTION 1

On the average, 33 percent of property damage accidents and 43 percent of personal injury accidents occurred on this subsection at night. Accidents per 10^6 VKT and accident cost rates (German Marks per 100 VKT) showed evident reductions on this subsection during day and night conditions (see Tables 3 and 4). Note that subsection 1 was continuously lighted during the periods A1 and A2 (see Table 1).

For day and night conditions, the development of the accident situation between the B, A1, and A2 periods is nearly parallel. This means that the reduction in accident rates and accident cost rates of accidents occurring during night conditions after the installation of lighting devices can be noticed also for day conditions. A comparison of the lighted subsection 1 with the unlighted subsection 3 reveals that accident rates and accident cost rates showed more favorable reductions, especially on subsection 1 during night conditions.

Furthermore, it is interesting to note that although the night-to-day ratio of accident rates for subsection 1 was increasing, the night-to-day ratio of accident cost rates, which accurately represents the severity of accidents, was decreasing between the B, A1, and A2 periods from 3.3 to 2.9 to 2.6 (German Marks per 100 VKT), respectively (see Table 4).

SUBSECTION 2

On the average, 29 percent of property damage accidents and 38 percent of personal injury accidents occurred on subsection 2 at night. Between the B and A1 periods, accident rates and accident cost rates were nearly cut in half. With the introduction of partial lighting on this subsection (see Table 1), it is interesting to see that between periods A1 and A2, accident rates and accident cost rates under night conditions remained at nearly the same level or even increased (see Tables 3 and 4).

Between periods B and A1, the reduction in accident rates and accident cost rates on this subsection is nearly parallel for day and night conditions. In contrast, between periods A1 and A2, the reduction continued under day conditions although it remained at nearly the same level or even increased under night conditions.

Comparing the night-to-day ratio of accident rates and accident cost rates in Tables 3 and 4, the preceding statement is supported by the decrease of the ratios from 1.8 to 1.5, respectively; and from 2.1 to 1.7 between the B (unlighted) and the A1 (lighted) periods. However, when partial lighting was introduced on this subsection in the A2 period, the night-to-day ratios showed a strong increase from 1.5 to 2.4, respectively, and from 1.7 to 2.7 (see Tables 3 and 4).

SUBSECTION 3

On the average, 30 percent of property damage accidents and 31 percent of personal injury accidents occurred on subsection 3 at night. This means that contrary to subsections 1 and 2, there was no evident difference between the accident categories, "property damage" and "personal injury" accidents. Although no lighting devices were present on this subsection (see Table 1), reductions in accident rates for the subsection can be noticed (see Tables 3 and 4). These reductions were, in general, not as strong as those on subsections 1 and 2. The night-to-day ratios reveal increases in accident rates and fluctuations in accident cost rates on the unlighted subsection 3.

THE EFFECTS OF PARTIAL LIGHTING

Although no clear conclusions have yet been drawn about the effectiveness of lighting, it should be mentioned here that further reductions in accident rates and accident cost rates under day conditions on subsection 2 could be observed between periods A1 and A2. In contrast, after several observations during night conditions, the reduction in accident rates and accident cost rates showed almost no decrease and, in some observations, even increased after the introduction of partial lighting (see Tables 3 and 4). Contrary to this development, accident rates and accident cost rates showed further decreases between periods A1 and A2 on subsection 1 under night conditions. The same is also true for the reduction in accident and cost rates on the unlighted subsection 3.

To draw more reliable conclusions about the effect of lighting on night accidents, the daytime period of 24 hr was divided into the time periods: Day, Dark 1, and Dark 2 (see Figure 2). (In Figure 2, period Day lasts from sunrise to sunset, period Dark 1 lasts from sunset until 10:00 p.m. and from 5:30 a.m. until sunrise, and period Dark 2 lasts from 10:00 p.m. until 5:30 a.m.)

Overall, the accident rates in Figure 2 appear to agree with the results of many findings, for example, that driving during the hours of day, and from sunset to about 10:00 p.m. and from about 5:30 a.m. to sunrise, is much safer than driving during the night hours between 10:00 p.m. and 5:30 a.m., regardless of the lighting conditions (1,2,11).

Under day conditions, Figure 2 shows that the reduction in accident rates is stronger on subsection 1 than on subsection 3. Different development could be noticed under Dark 1 conditions. For example, after the introduction of lighting in December 1973, a sharp decrease in the accident rates could be noticed on subsections 1 and 2 between the B and A1 periods. However, this reduction remained at nearly the same level between the A1 and A2 periods on both sections. In contrast, the accident rate on subsection 3 showed an increase between the B and A1 periods, although it remained at nearly the same level between the A1 and A2 periods.

Under Dark 2 conditions, the accident rate decreased strongly on subsections 1 and 2 between the B and A1 periods. While this reduction continued on subsection 1 (lighted) between the A1 and A2 periods, it increased strongly on subsection 2 (not

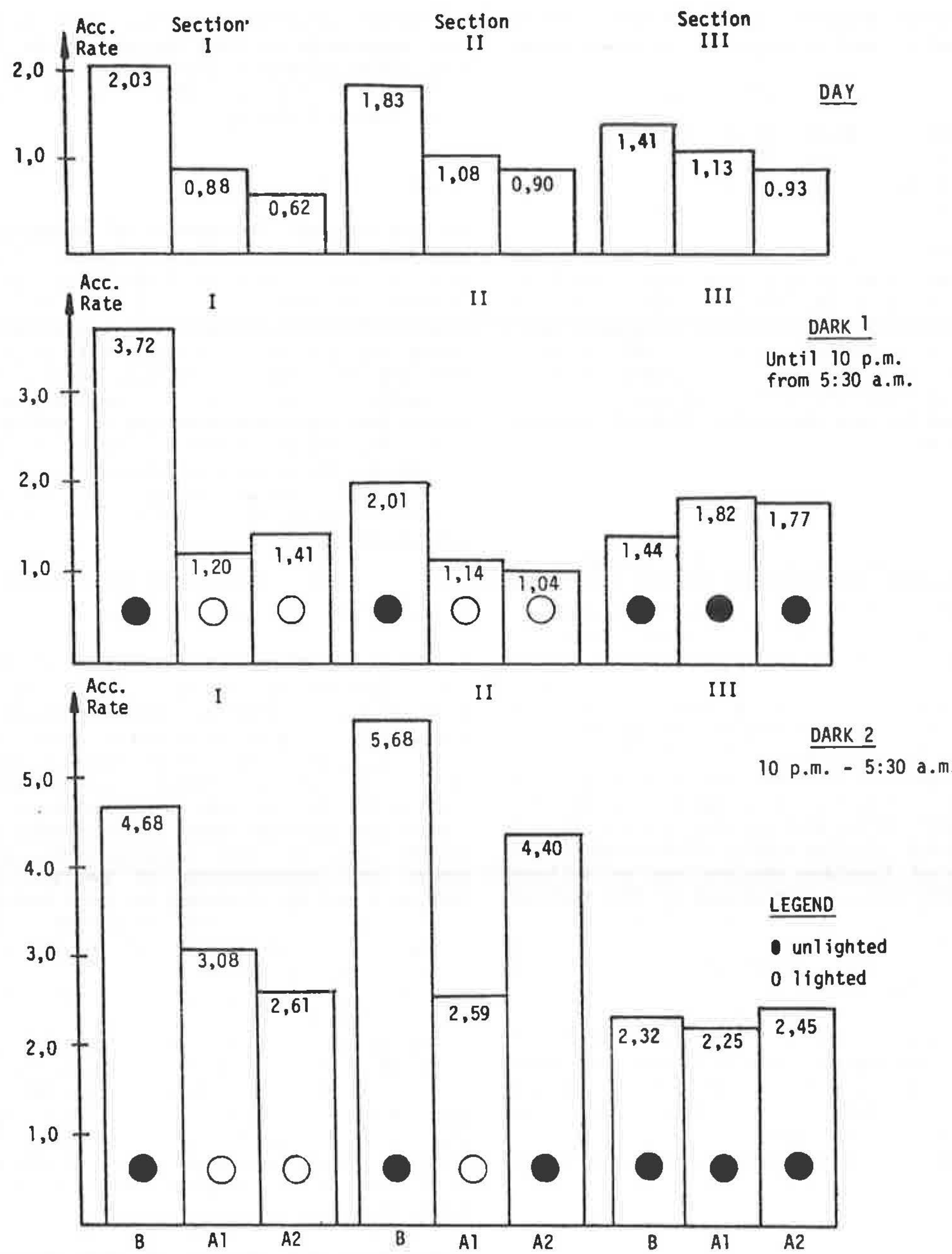

FIGURE 2 Accident rates for Day, Dark 1, and Dark 2 daytime periods with identification of lighting conditions.

lighted) between the same periods. During the 9-yr investigation period, the accident rate on subsection 3 revealed no significant changes among the 3 periods during the late night hours.

CONCLUSION

The case study revealed that positive effects of lighting on reducing accident rates and accident cost rates on freeways in suburban areas cannot be excluded, even if no real convincing results could be statistically proven. However, it should be noted that the positive results obtained through continuous lighting were lost in the case of partial lighting for energy conservation purposes. The increase in accident rates after switching off lights at night between 10:00 p.m. and 5:30 a.m. especially supports this statement. The savings in energy costs after switching off lights as compared with savings in accident costs could not be determined in this study.

It is obvious that this research is a small step toward determining the effectiveness of freeway lighting in terms of safety. Further research is needed to verify and add to the findings of this study.

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Detection of Reflectorized License Plates

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ABSTRACT

This paper contains data on the detection distances of reflectorized white license plates. Detection distances were obtained for a car heading angle and a driver's line of sight in 5 different treatments with 12 drivers. The order of presentation of the five treatments for a given heading angle was basically random and approximately balanced. Each driver sat in a stationary car on a 2,000-ft long runway and detected an approaching target configuration under low beam conditions against a background containing a number of luminaires and other light sources. There were three parallel approach paths on the runway and for each treatment, three approaches were made on each path toward a driver. The results of this study indicated that the average detection distance increase from treatment 1 to treatment 5 was 39 percent for the -3-degree heading angle and 85 percent for the 10-degree heading angle. Based on the detection distances obtained in this study and calculations that involve stopping sight distances and/or decision sight distances, the potential for significant safety benefits when using reflectorized license plates in addition to the red rear cube corner vehicle reflectors can be demonstrated. These potential safety benefits are especially significant for an 84-CIL license plate combined with two red rear reflectors.

Reflectors and reflectorized license plates have been in use for many years as a means of aiding the driver in the initial detection, recognition, and identification of stationary vehicles on or off the roadway at night with no lights on. Several studies have been conducted that compare accident rates of vehicles with reflectorized versus nonreflectorized license plates. Henderson, Ziedman, Burger, and Cavey reviewed and summarized a number of these studies (1). In the past, Hulbert and Burg, Cook, and Sivak and Olson reviewed license plate and reflectorization studies (2-4).

Although most of these studies indicated a reduction in the accident rate because of reflectorized license plates, some of the studies indicated that there was no statistically significant reduction. It could be argued, however, that if the reflectivity of the license plates used in these studies had been higher, the results would have been more positive. In the review cited previously, the authors state

> Almost every accident study reviewed showed a reduction in accidents when conspicuity was improved. This near-unanimity tends to outweigh the problems of interpretation of these studies. In addition, the finding that poor driver information processing is related to higher accident rates further strengthens a conclusion that improving conspicuity, and thereby reducing information processing loads, will reduce accidents (1).

On similar lines, Vanstrum and Kotnour state in their unpublished report on the Tennessee accident data and the effect of reflectorized license plates

> Despite the fact that state accident data in general is difficult to work with in establishing the effect of a single variable, a careful analysis shows that a small but significant accident reduction can be attributed to the introduction of reflective sheeting license plates in the state of Tennessee.

It is well established that drivers get their visual information through a series of discrete eye fixations at different objects and roadway features and, therefore, the initial detection of an object in the driving scene at night most often occurs a few degrees away from the fovea in the peripheral visual field. Eye scanning data for straight road driving at night such as that reported by Zwahlen (5) indicates that the range of horizontal eye fixations is approximately 13 degrees and the range of vertical eye fixations is approximately 6 degrees.

In spite of the fact that the initial detection of objects at night while driving will most likely occur peripherally rather than foveally, most visual conspicuity studies reported in the literature provide results for foveal detection, recognition, or identification only. Some authors, however, have recognized the importance of peripheral viewing such as Matson who stated: "The accuracy of identification of traffic signs increases as the angle between the axis of vision and the line drawn from the traffic sign to the motorist's eye decreases (6). He also suggested that the target should fall within a visual area of 10 to 12 degrees on the horizontal axis and 5 to 12 degrees on the vertical axis for better effectiveness. Recognizing the fact that the visual detection of objects while driving at night can occur either foveally or peripherally and taking into account the ranges of the horizontal and vertical eye fixations during night driving on straight roads, a car heading angle and a driver's line of sight of -3 degrees to the left was chosen as a representative condition for a near foveal detection, and a car heading angle and a driver's line of sight of 10 degrees to the right was chosen as a representative upper limit for a peripheral detection.

The objective of this study was to determine the detection distances at night for low beam conditions for near foveal (-3 degrees) and peripheral (10 degrees) detection for five experimental treatments. The five treatments were:

1. Two Chevette red rear reflectors,
2. One 24-CIL white license plate,
3. Two Chevette red rear reflectors, and 1 24-CIL white license plate,
4. One 84-CIL white license plate, and
5. Two Chevette red rear reflectors and 1 84-CIL white license plate.

METHOD

Subjects

Twelve subjects participated in the experiment; 9 males and 3 females. The average age of the subjects was 21.4 yr with a standard deviation of 2.35 yr. They had an average driving experience of 6 yr and drove an average 8,000 miles/yr, the respective standard deviations being 2.3 yr and 6,000 miles/yr. All the subjects were students, they all had normal visual acuity, normal reaction times, normal information processing capabilities, and were paid to participate in the experiment.

Apparatus

A 1979 Mercury Bobcat was used as the experimental car. The headlamps (General Electric 6052) of this car were 24.25 in. above the ground, and had a horizontal center-to-center distance of 48.45 in. The electrical system of the car operated at 14.15 volts. The theoretical location of the hottest spot of these headlamps (30000 cp at 12.8 volts, 55 watts) is approximately 2 degrees to the right and approximately 2.25 degrees down. The actual measured location of the hottest spot for the left low beam was 2.48 degrees to the right and 1.55 degrees down, and the actual measured location of the hottest spot for the right low beam was 0.95 degrees to the right and 1.72 degrees down. The average distance from the longitudinal vertical center plane of the car to the center of the subject's eyes in the driver position was 13.5 in. The average horizontal distance from the headlamps to the subject's eyes was 82.25 in. and the average subject eye height was 41.5 in. above the ground.

A black 5-horsepower Dune Kart was used as the target vehicle. On the front of this vehicle, two Chevette red rear reflectors and/or 1 white 24-CIL license plate or 84-CIL license plate was mounted in such a manner that its location and configuration were exactly identical to those on a 1979 Chevette. The center-to-center distance between the reflectors was 27.63 in. and the horizontal centerline was at a height of 26.75 in. above the ground. The reflectors were fixed in such a way that their reflecting surfaces made an angle of -10 degrees with the transverse axis of the Dune Kart to simulate the situation of a vehicle parked at a slight angle along a road. During the experiment, the target vehicle was driven by a person wearing dark clothing at a speed of about 10 mph.

License plates (size: 6 in. x 12 in.) of two levels of reflectivity were used: 24 CIL (measured 23.5 cd/fc per license plate at a 0.2-degree observation angle and -4 degrees entrance angle) and 84 CIL (measured 83.6 cd/fc per license plate at a 0.2-degree observation angle and -4 degrees entrance angle). The two Chevette red rear cube corner reflectors were randomly selected from 6 Chevette reflectors obtained from different Chevette vehicles from the year 1979. The two reflectors had a total red reflecting area of 0.047 ft^2. They were 4.25 in. x 3.063 in. with an inner nonreflecting rectangular area of 3.125 in. x 2 in. The left red reflector had a CIL value of 4.0 cd/fc and the right red reflector had a CIL value of 7.1 cd/fc (measured at a 0.2-degree observation angle and 0 degrees entrance angle).

Experimental Site

A 75-ft wide, 2,000-ft long section of a concrete airport runway no longer in use located at the edge of the city of Athens, Ohio, and near a shopping mall was used as the experimental site. A 2-lane state highway with moderate traffic was located parallel (about 200 ft away) to the runway. A number of luminaires, a few advertising signs, and other light sources were within the field of view (mainly in the left peripheral field) of the subjects. There were three approach paths parallel to the runway axis.

The front center of the test car was placed above the center line of the runway. Looking forward from the car, path 1 was 12.5 ft to the left of the runway centerline. Path 2 was 6.25 ft to the right of the runway center line. Path 3 was 25 ft to the right of the runway centerline. The purpose of having three paths was to determine how the lateral location of the approach path of the oncoming target configuration would affect, if at all, the detection distances. Moreover, the inclusion of three paths in the experiment was intended to introduce some uncertainty to the subject about the lateral location of the approaching target configuration. For a subject to fixate the eyes at an object in the -3- or 10-degree direction, two red cube corner reflectors mounted on stakes 3 ft above the ground were placed at appropriate locations in the grass on the left and right side of the runway (40 ft to the left of the runway centerline at 763 ft for -3 degrees, 80 ft to the right of the runway centerline at 454 ft for 10 degrees). Figure 1 shows the layout of the experimental site.

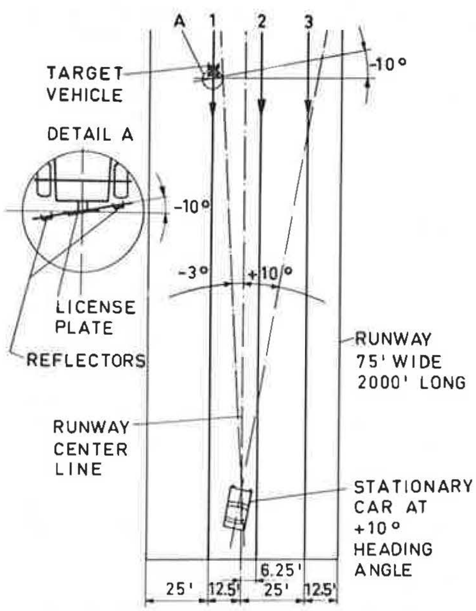

FIGURE 1 Layout of experimental site and arrangements.

Experimental Design

The independent variables for this experiment were

1. Two Chevette red rear reflectors,
2. One 24-CIL white license plate,
3. Two Chevette red rear reflectors and one 24-CIL white license plate,
4. One 84-CIL license plate, and
5. Two Chevette red rear reflectors and 1 84-CIL white license plate.

The dependent variable was the detection distance measured in feet.

Each subject was presented either all five treatments for the -3-degree heading angle first, or all five treatments for the 10-degree heading angle first. One-half of the subjects started with the -3-degree heading angle while the other one-half started with the 10-degree heading angle. The order of presentation of the five treatments for a given heading angle for each subject was basically random and approximately balanced considering that a perfect balancing scheme was not possible with 12 subjects and five treatments. Within a given treatment, nine observations were made. Each path approach was presented three times. The nine observations were grouped into three blocks of three observations each. Each path approach was presented randomly and only once within a block.

Procedure

The car was positioned on the runway by using plum bobs attached to the center of the front bumper and to the center of the rear bumper. Two 25-ft long lines were painted on the runway to indicate the direction of the car centerline for the -3- and 10-degree heading angles. The front center of the car was placed exactly above the runway centerline, and the car was placed to make an angle of either -3 or 10 degrees with the runway centerline. The subject sat comfortably in the driver's seat, and one experimenter sat beside the subject. At the beginning of each experiment, the subject's eye-height, the horizontal distance of the eyes to the headlamps, and other dimensions were measured.

To conduct the experiment, a group of experimenters positioned themselves at various locations along the side of the runway and signaled to the experimenter who was sitting in the car at the beginning of each trial, by using a flashlight. Another experimenter drove the target vehicle. At the beginning of the experiment, the experimenter sitting in the car briefed the subject about the purpose of the experiment and gave the subject a copy of the experimental instructions to read. During the experiment, the low beams of the car were always on, and the engine was kept idling. The experimenter in the car recorded the time, battery voltage, weather conditions, and subject responses. At the beginning of each trial, the subject was asked by the experimenter to start fixating the eyes at the red cube corner reflector positioned ahead either on the left (-3 degrees) or right (10 degrees) side. The subject was instructed to be prepared to detect the approaching target configuration while fixating the eyes at the reflector. The target vehicle would approach the stationary car along any one of the three paths. As soon as the subject had the initial sensation of detection of the target configuration in the peripheral or near foveal field of vision, he or she would switch immediately from the low beams to the high beams and keep the high beams on for a few seconds. As soon as the driver of the target vehicle noticed the high beams, he or she would drop a small sand bag on the runway that indicated the detection distance. The measurement crew would then measure and record the detection distance. They would also pick up the sand bag and return it to the target vehicle driver.

After everybody cleared the runway and the target vehicle had moved back to the end of the runway and was positioned in a perpendicular direction to the runway centerline, the measurement crew would give the signal to the experimenter sitting in the car indicating the beginning of the next trial. The correct approach path of the target vehicle and a subject's continuous eye fixation at the fixation point were checked by the experimenter sitting in the car. The experimenter also recorded for each trial the subject's response with regard to what the subject thought was actually detected first (e.g., red reflectors, license plate or both). The time to conduct the 45 trials (5 treatments x 9 observations) for 1 car heading angle condition usually took approximately 1 hr and 15 min.

RESULTS

Table 1 gives data on the group detection distance averages, standard deviations, minimums and maximums for all paths combined for all treatments for the -3- and 10-degree heading angles. Tables 2 and 3 give data on group detection distance averages, standard deviations, minimums and maximums for each path for all treatments for the -3- and 10-degree heading angles. Table 4 gives data on the percentage increments in average detection distances from lower to higher treatment combinations for the -3- and 10-degree heading angles.

Figure 2 shows the detection distance averages, standard deviations, minimums and maximums for all paths combined for all experimental treatments for the -3- and the 10-degree heading angles. Figures 3 and 4 show the detection distance averages, standard deviations, minimums and maximums for each path for each treatment for the heading angles of -3 and 10 degrees.

Figures 5 and 6 show the cumulative detection distance distributions for all paths combined for each treatment for the -3- and 10-degree heading angles. Figures 7 and 8 show the minimum recommended values for the decision sight distance (DSD) for the speeds of 25 mph, 35 mph, and 55 mph against the actual values of the average detection distances, and against the 50-percent values of the actual average detection distances (reduced to adjust for factors such as subject alertness, information processing load, driver age, cleanliness of the windshield, and so forth) for each treatment for the

TABLE 1 Group Detection Distances—all Treatments

	Treatment				
Heading Angles	T-1	T-2	T-3	T-4	T-5
-3 degree (left)					
Mean	1,293	1,480	1,571	1,750	1,794
Standard deviation	246	245	214	203	179
Minimum	742	881	1,021	1,115	1,205
Maximum	1,949	1,949	1,973	1,998	2,013
10-degree (right)					
Mean	480	552	680	799	890
Standard deviation	117	161	163	210	229
Minimum	276	329	322	402	464
Maximum	784	1,089	1,019	1,242	1,401

Note: For averages, standard deviations, minimums, and maximums for all 3 paths combined, and for -3- and 10-degree heading angles, N = 108; all distances are in feet; and T-1 is vehicle rear reflectors only, T-2 is 24-CIL license plate only, T-3 is 24-CIL license plate and vehicle rear reflectors, T-4 is 84-CIL license plate only, and T-5 is 84-CIL license plate and vehicle rear reflectors.

TABLE 2 Group Detection Distances—for Each Path, for -3 Degrees

Treatment	Path 1	Path 2	Path 3
T-1			
Mean	1,421	1,281	1,193
Standard deviation	232	227	215
Minimum	900	879	742
Maximum	1,949	1,635	1,668
T-2			
Mean	1,601	1,432	1,434
Standard deviation	214	217	253
Minimum	1,142	970	881
Maximum	1,949	1,823	1,818
T3			
Mean	1,668	1,506	1,516
Standard Deviation	218	196	202
Minimum	1,073	1,021	1,085
Maximum	1,973	1,738	1,783
T-4			
Mean	1,778	1,727	1,737
Standard deviation	207	216	190
Minimum	1,130	1,115	1,344
Maximum	1,998	1,972	1,982
T-5			
Mean	1,851	1,774	1,761
Standard deviation	159	174	193
Minimum	1,230	1,227	1,205
Maximum	2,012	2,013	1,997

Note: For averages, standard deviations, minimums, and maximums for each treatment, N = 36; all distances are in feet; and T-1 is vehicle rear reflectors only, T-2 is 24-CIL license plate only, T-3 is 24-CIL license plate and vehicle rear reflectors, T-4 is 84-CIL license plate only, and T-5 is 84-CIL license plate and vehicle rear reflectors.

TABLE 3 Group Detection Distances—for Each Path, for 10 Degrees

Treatment	Path 1	Path 2	Path 3
T-1			
Mean	476	472	562
Standard deviation	184	99	100
Minimum	248	348	357
Maximum	623	626	644
T-2			
Mean	561	626	745
Standard deviation	159	131	157
Minimum	303	384	399
Maximum	1,017	922	1,089
T-3			
Mean	596	668	778
Standard deviation	152	153	135
Minimum	386	376	652
Maximum	887	926	999
T-4			
Mean	739	763	908
Standard deviation	209	195	182
Minimum	402	403	557
Maximum	1,187	1,202	1,242
T-5			
Mean	840	840	992
Standard deviation	249	216	195
Minimum	448	541	600
Maximum	1,346	1,232	1,401

Note: For averages, standard deviations, minimums, and maximums for each treatment, N = 36; all distances are in feet; and T-1 is vehicle rear reflectors only, T-2 is 24-CIL license plate only, T-3 is 24-CIL license plate and vehicle rear reflectors, T-4 is 84-CIL license plate only, and T-5 is 84-CIL license plate and vehicle rear reflectors.

TABLE 4 Matrix Showing Percentage Increments in Average Detection Distances from Lower to Higher Treatment Combinations

Treatment	T-1	T-2	T-3	T-4	T-5
-3-Degree Heading Angle (left)					
T-1	–	14	22	35	39
T-2	–	–	6	18	21
T-3	–	–	–	11	14
T-4	–	–	–	–	3
T-5	–	–	–	–	–
10-Degree Heading Angle (right)					
T-1	–	15	42	66	85
T-2	–	–	23	45	61
T-3	–	–	–	18	31
T-4	–	–	–	–	11
T-5	–	–	–	–	–

Note: T-1 is vehicle rear reflectors only, T-2 is 24-CIL license plate only, T-3 is 24-CIL license plate and vehicle rear reflectors, T-4 is 84-CIL license plate only, and T-5 is 84-CIL license plate and vehicle rear reflectors.

heading angles of -3 and 10 degrees. Figures 9 and 10 show the recommended values for the stopping sight distance (SSD) for the same speeds against the actual values of the average detection distances and against the 50-percent values of the actual average detection distances (adjusted for subject alertness, and other variables) for the heading angles of -3 and 10 degrees.

DISCUSSION OF RESULTS

From Table 1 and Figure 2, it can be observed that the detection distances increase consistently from treatment 1 to treatment 5. The increase from treatment 1 to treatment 5 was 39 percent for the -3- degree heading angle and 85 percent for the 10-degree heading angle (from 1,293 to 1,794 ft for -3 degrees, from 480 ft to 890 ft for 10 degrees). The detection distance increases from any lower reflectivity treatment to any higher reflectivity treatment are all statistically significant at the 0.05 level with the exception of the increase from treat-

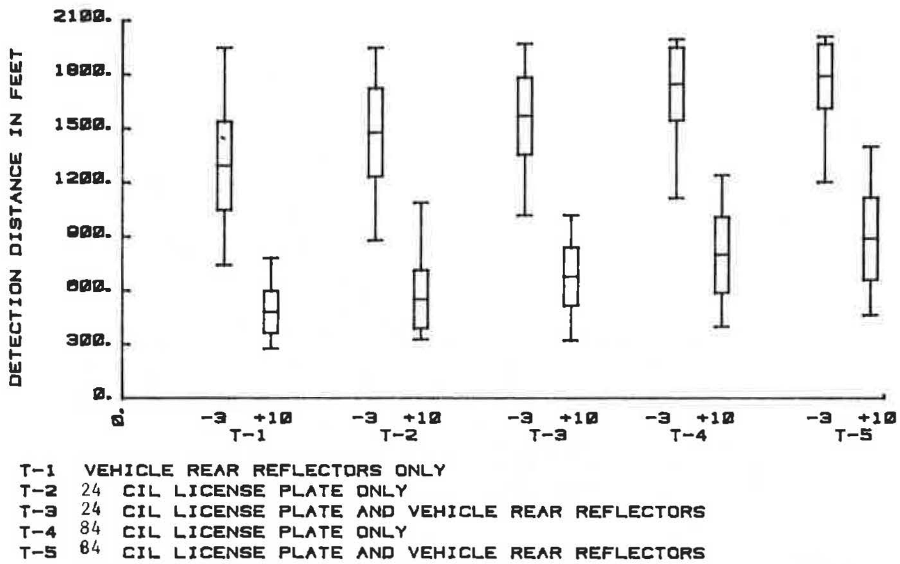

FIGURE 2 Detection distance means, standard deviations, minimums, and maximums for heading angles of -3 and 10 degrees.

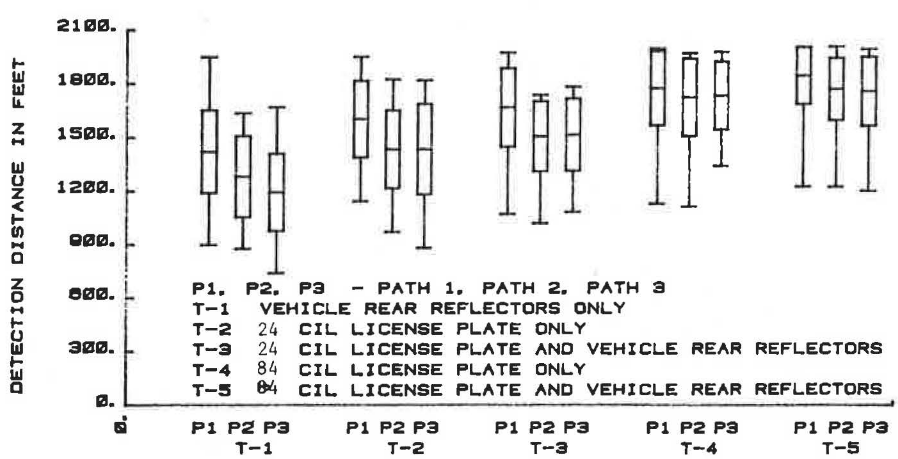

FIGURE 3 Detection distance means, standard deviations, minimums, and maximums for a heading angle of -3 degrees.

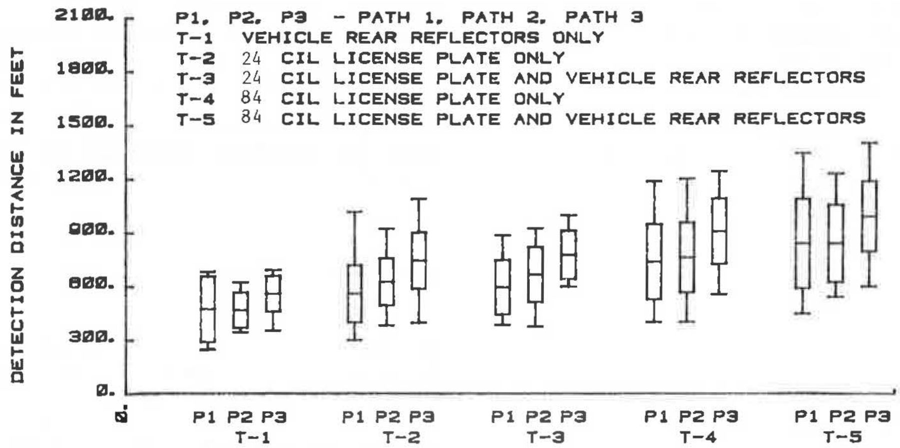

FIGURE 4 Detection distance means, standard deviations, minimums, and maximums for a heading angle of 10 degrees.

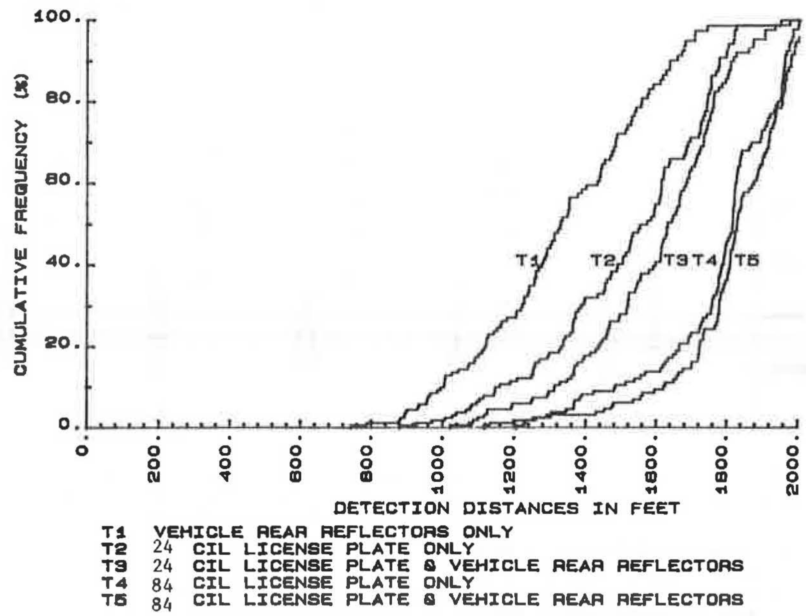

FIGURE 5 Cumulative detection distance distribution with all paths combined—heading angle of -3 degrees.

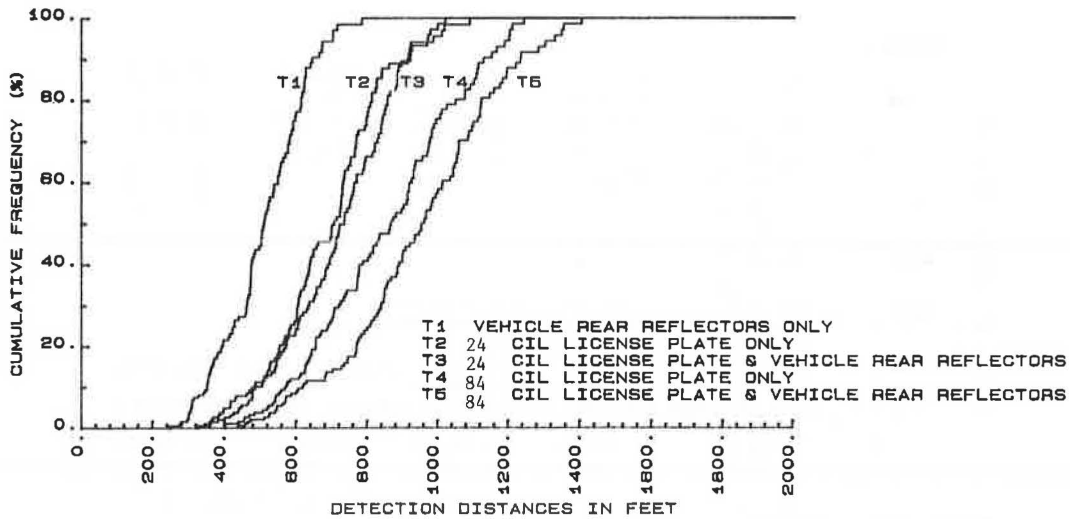

FIGURE 6 Cumulative detection distance distribution with all paths combined—heading angle of 10 degrees.

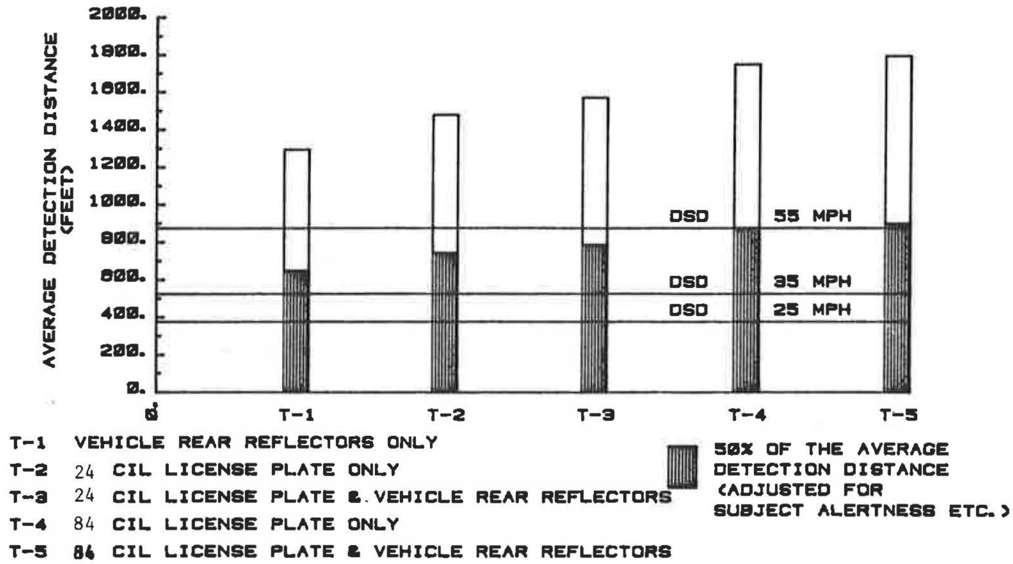

FIGURE 7 Comparison between average and 50 percent-detection distance for 25, 35, and 55 mph for a heading angle of -3 degrees.

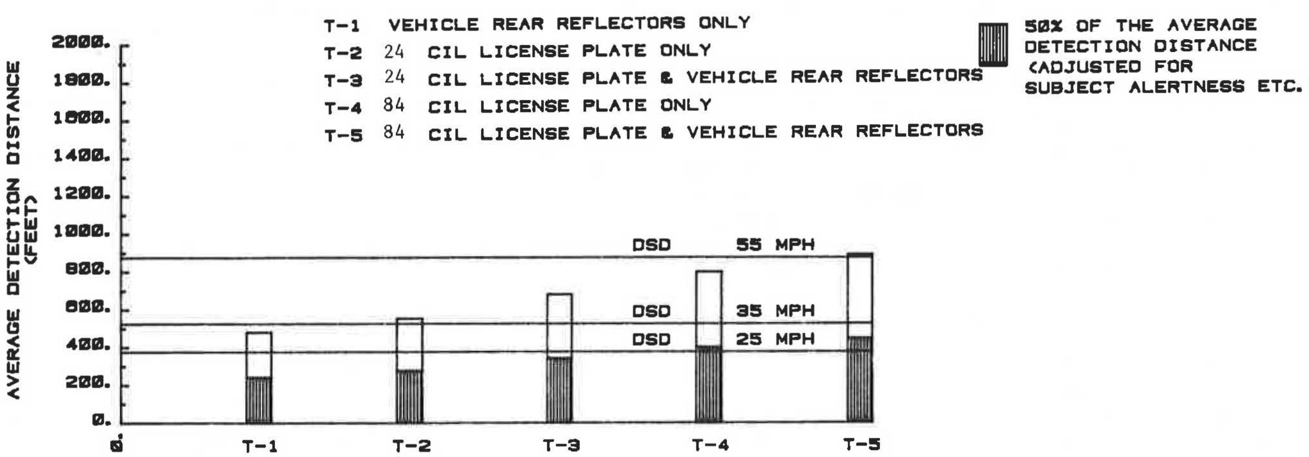

FIGURE 8 Comparison between average and 50 percent-detection distance for 25, 35, and 55 mph for a heading angle of 10 degrees.

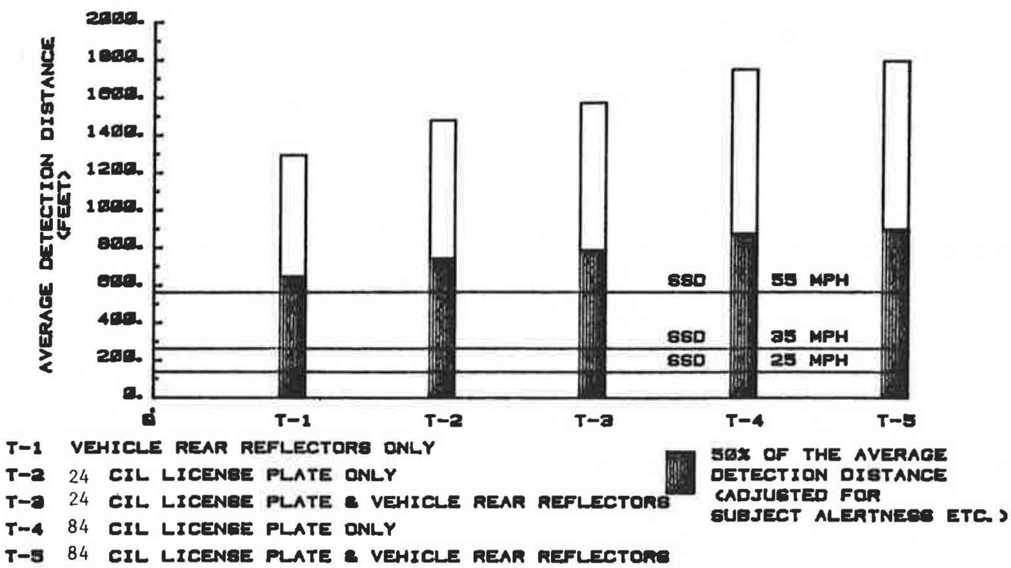

FIGURE 9 Comparison of average and 50 percent-detection distance and stopping sight distance for 25, 35, and 55 mph for the heading angle of -3 degrees.

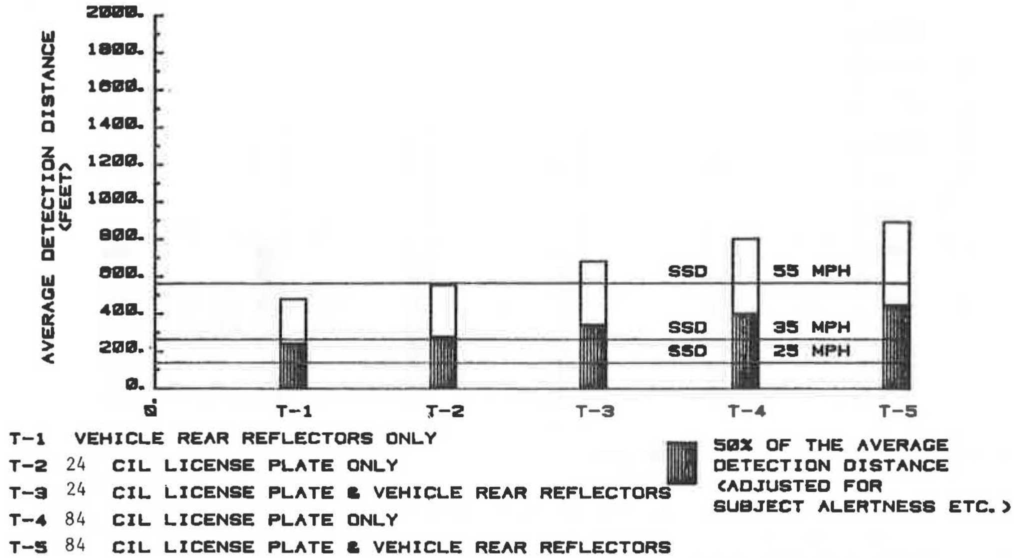

FIGURE 10 Comparison of average and 50 percent-detection distance and stopping sight distance for 25, 35, and 55 mph for the heading angle of 10 degrees.

ment 4 to treatment 5 for the -3-degree heading angle.

A runway longer than 2,000 ft would most likely have resulted in somewhat longer detection distances and somewhat longer and less truncated standard deviations and ranges for treatments 4 and 5 for the -3-degree heading angle and thus could have resulted in a statistically significant detection distance increase from treatment 4 to treatment 5. From Table 1 and Figure 2, the relatively large standard deviations and ranges for the detection distances can also be observed. Figure 2 especially shows the large variability that is typical for human detection of a reflectorized target configuration in a real-world urban night environment.

From Table 1 and Figure 2, it can be further observed that there was a consistent large increase in the detection distances for each treatment from the 10-degree heading angle to the -3-degree heading angle. For example, for treatment 1, the average detection distance increased approximately 2.7 times from 480 ft for 10 degrees to 1,293 ft for -3 degrees. The hottest point of the left low beam was actually aimed at an angle of 2.48 degrees to the right and 1.55 degrees down, and the hottest point of the right low beam was aimed at an angle of 0.95 degrees to the right and 1.72 degrees down. The effect of the aims of the two low beams was that when the car heading angle was -3 degrees to the left of the centerline, the low beams were practically aimed straight down the runway centerline providing just about the most optimal low beam conditions for the detection of a target configuration straight ahead. In this situation, the detection of the target took place only about 3 degrees away from the fovea or line of sight in a visual region, which is still efficient from a detection point of view when compared to the periphery.

Also, the relative high voltage level (14.15 volts) of the car's electrical system and the relatively high candle power intensity level of the two low beams might have contributed to the observed long detection distances for the -3-degree heading angle condition. On the other hand, the much shorter detection distances for the 10-degree heading angle are partly caused by the low beams pointing 12.48 degrees (left beam) and 10.95 degrees (right beam) to the right of the runway centerline. This fact, coupled with the significant fact that a subject had to detect the target at about 10 degrees in the periphery where the efficiency of the visual system with regard to detection is slightly lower when compared with the fovea.

It can also be observed from Table 1 and Figure 2 that there is always a small but consistent increase in the average detection distance when a white license plate was used in conjunction with the two Chevette red rear reflectors. These reflectors, themselves, produced considerably shorter detection distances (the increase for the -3 degree heading angle was 6.2 percent for the 24-CIL license plate and 2.5 percent for the 84-CIL license plate; the increase for the 10-degree heading angle was 23.2 percent for the 24-CIL license plate and 11.3 percent for the 84-CIL license plate). Zwahlen reported a similar phenomenon indicating that longer detection distances result when a reflective surface was cut in half and presented as two reflectors instead of one (5).

From Tables 2 and 3 and Figures 3 and 4, it can be observed that a rather consistent pattern exists among the detection distances for the three paths for each of the five treatments. In the case of the -3-degree heading angle detection distance results, the detection distances for path 1 (path 1 is on the left side of the runway centerline) are consistently the longest, while the direction distances for paths 2 and 3 (on the right side of the runway centerline) are consistently the shortest. These consistent patterns are the result of aiming the low beams practically straight down the runway. In the case of the 10-degree heading angle detection distance results, the detection distances for path 3 are consistently the longest, while the detection distances for path 1 are usually the shortest. Again, because the low beams are aimed at an angle of more than 10 degrees to the right of the runway centerline, it would be expected that the best detection performance would occur along path 3 and the worst detection performance would occur along path 1.

Turning to Figures 5 and 6, the large variability can be observed in the detection performance for each treatment. In Figure 5, it can clearly be seen that the cumulative detection distance distributions for treatments 4 and 5 are truncated at the longer detection distances. This truncation is attributed to the limited length of the runway (2,000 ft). Figures 5 and 6 are useful illustrations because they allow a reader to determine for a given detection distance the proportion of the population that has detection distances below this value. In addition, these figures can also be used to determine any set of percentile values of interest such as the detection distance value for which 95 percent of the population have equal or shorter detection distances.

It should be noted that the lower detection distance values shown in these cumulative detection distance distributions are the values where accidents are most likely to occur. It is therefore important to increase the level of reflectivity sufficiently to effect a significant increase in these lowest detection distance values. In looking at the cumulative detection distance distributions in Figures 5 and 6, it can clearly be observed that there exist the slight but consistent and significant increases between treatments 2 and 3 and between treatments 4 and 5. As was discussed earlier, these increases were somewhat unexpected and indicate that human detection does not simply follow optical and photometric calculations alone and has more than just an illumination dimension to it.

In Figure 7 (for the -3-degree heading angle), it can be observed that for the 50-percent adjusted average detection distances, only treatment 5 exceeds the minimum recommended DSD for 55 mph. (The DSD is the distance at which drivers perceive a potentially hazardous situation and react to the impending danger efficiently.) As given in the research report by McGee, et al., for a design speed of 25 mph, the recommended DSD is between 375 ft and 525 ft; for a design speed of 35 mph, it is between 525 ft and 725 ft; and for a design speed of 55 mph, it is between 875 ft and 1,150 ft (7). As observed in Figure 8 (for 10-degree heading angle), only treatments 4 and 5 for the 50-percent adjusted average detection distances exceed the minimum recommended DSD for 25 mph.

In Figure 9 (for -3 degree heading angle), it can be seen that even for the 50-percent adjusted average detection distances, all treatments exceed the recommended SSD for the 55 mph speed. The recommended values for SSDs for 25 mph, 35 mph, and 55 mph are 137 ft, 263 ft, and 563 ft, respectively. In looking at Figure 10 (for 10-degree heading angle), it can be observed that for the 50-percent adjusted average detection distances, all treatments with the exception of treatment 1 exceed the recommended SSD for the 35 mph speed. Figures 7 through 10 are useful in providing the reader with close-to-ideal and 50-percent adjusted average detection distances for each treatment that can then be evaluated in terms of either the minimum recommended DSDs or the SSDs for the three speeds from 25 to 55 mph.

CONCLUSIONS AND RECOMMENDATIONS

This study clearly demonstrates that reflectorized license plates with 24-CIL or especially 84-CIL specific intensity levels do increase the conspicuity and the detection distances of a car parked along a highway at night in a statistically and practically significant manner. The obtained longer detection distances mean that a driver will detect earlier a parked car with no lights on at night, and will therefore have more time for recognition, decision making and proper control actions. On the basis of the results of this study and calculations involving SSDs and DSDs, it can be demonstrated that the potential exists for significant safety benefits when using reflectorized license plates in addition to the vehicle red rear reflectors. The potential for these safety benefits is especially significant for the 84-CIL license plate combined with two red rear reflectors. Therefore, an increase of the initial reflectivity level of license plates to 84 CIL is highly recommended, and with such a level of reflectivity a decrease in reflectivity as a result of wear and exposure over time would be assured and would still result in an adequate conspicuity level, which would lead to significant safety benefits.

Also on the basis of the results of this study, it may be concluded that having a second reflectorized license plate of 24 CIL, or especially 84 CIL, attached to the front of a car will greatly increase the conspicuity and detection distance of a parked car along a road at night in the case where the front end of such a car faces an approaching vehicle. The probability of early detection leading to potential safety benefits in such a case is greatly enhanced because there are usually no vehicle reflectors placed on the front of cars.

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A Microcomputer Program for Use with the American National Standard Practice for Roadway Lighting

L. ELLIS KING

ABSTRACT

Recommended roadway lighting practices are set forth in the 1983 American National Standard Practice for Roadway lighting. In the past, horizontal illuminance has been recognized in the 1983 Standard Practice as the basis for design of roadway lighting. However, lighting engineers have long known that pavement luminance and veiling luminance criteria provide a better correlation with roadway lighting as perceived by the driver. In the 1983 Standard Practice, luminance is recognized as the primary and preferred basis for design, and values for luminance and veiling luminance are recommended. Illuminance criteria are retained as an acceptable alternative. A microcomputer program has been developed for use with the Standard Practice. The program calculates values for illuminance, luminance, and veiling luminance by using input data that include pavement directional reflectance factors, lamp/luminaire candlepower arrays, and geometry of the lighting system. These values are calculated at regularly spaced test points between two adjacent luminaires. For luminance and veiling luminance calculations, the observer moves through the system viewing the roadway at a fixed distance ahead. All calculations are carried out by using formulas and procedures recognized in the Standard Practice, and the output includes values for both the illuminance and luminance design criteria that are contained in the Standard Practice. Written in Microsoft BASIC-80, the program requires a control program for microcomputers, a disk operating system, a RAM of at least 64K, and two disk drives.

The principal purpose of roadway lighting, as stated in the 1983 American National Standard Practice for Roadway Lighting, is "to produce quick, accurate, and comfortable seeing at night" (1). The ability to see at night contributes to the safe and efficient flow of traffic on highways during the hours of darkness. However, in many instances, limitations of the human eye prevent vehicle headlights alone from completely satisfying visual nighttime driving requirements. In these cases, fixed roadway lighting aids the driver by providing earlier warnings of hazards on or near the highway. The driver can then use this early information to formulate his response to any unsafe condition. Fixed roadway lighting also contributes to a more pleasant and comfortable night-driving environment, which, in turn, reduces driver fatigue and improves driver efficiency.

Recommended roadway lighting practices for North America are set forth in the Standard Practice. The Standard Practice is revised approximately every 5 years under the sponsorship of the Illuminating Engineering Society of North America. The latest version (1983) has been revised from the previous 1977 version to include a luminance method for design that also considers veiling luminance (glare). Although the new luminance method is the preferred method for design, it is recognized in the Standard Practice that because of complexity, the calculation and measurement of pavement luminance may be difficult and burdensome for some agencies. For this reason, the older illuminance design procedures have been retained in the 1983 revision as an acceptable alternative.

Also in the Standard Practice, pavement luminance is recognized as the critical design variable as follows:

> The criteria for roadway lighting in North America have been based on horizontal illuminance. However, it is known that pavement luminance and veiling luminance (glare) criteria provide a better correlation with the visual impression of roadway lighting quality. It is possible to satisfy illuminance criteria and fall far short of the luminance criteria.

The importance of pavement luminance in roadway lighting design has been known to illuminating engineers for many years and has been the subject of numerous research projects and reports, many of which are listed in the bibliography of the Standard Practice. For readers unfamiliar with the illuminance versus luminance concept, the following section provides an update of a previous report on the subject (2).

ILLUMINANCE VERSUS LUMINANCE

Illuminance is the measure of the amount of light flux striking a surface. It is independent of (a) the direction from which the light comes, (b) the number of light sources and their locations, (c) the type of light source, and (d) the type of surface it strikes. A surface may be illuminated to a given level by one concentrated light source placed perpendicular to the surface or by several less intense sources placed at an angle to the surface. The illuminance is the same whether the surface is a traffic-polished asphalt pavement or a new, rough-finished concrete pavement. For roadway lighting, illuminance is a widely understood and easily calculated quantity. However, it has very little value in describing the actual observed highway situation.

Luminance is a measure of the amount and concentration of light flux leaving a surface and is the only light by which an object is seen. It is the luminance that controls the magnitude of the sensation that the brain receives of an object. The luminance of a surface depends on all of the quantities of which illuminance is independent, including the direction from which the light comes, the directions from which the surface is viewed, and the light-reflecting characteristics of the surface itself. The amount of light falling on a small area of a surface may be measured as the illuminance on that area. For a highway pavement, this incident light is generally reflected in all directions and its directional distribution is determined by the properties of the surface and the manner in which the light strikes the surface. The apparent luminance of the surface is determined by the amount of light reflected toward the observer's eye.

All surfaces, including roadway surfaces, may be classified into three major groups according to the way in which they reflect light. The ideal specular surface is one that reflects all the luminous flux received by a point at an angle of reflection exactly equal to the angle of incidence. The reflected ray, the normal to the surface at the point of incidence, and the incident ray all lie in the same plane. An observer looking at a perfect specular surface along the direction of the reflected light will see an undistorted image of the object, and the image will be the same size as the object. The luminance of the image will be proportional to the luminance of the object. Some practical surfaces, such as mirrors, highly polished metal surfaces, and the surface of liquids, closely approximate the ideal specularly reflecting surface.

The perfectly diffuse surface is at the opposite pole from the ideal specular surface. The diffuse (or mat) surface reflects light as a cosine function of the angle from the normal, regardless of the angle of incidence. Because the luminance of the surface is equal to the intensity divided by the projected area that is also a cosine function of the angle from the normal, the perfectly diffuse surface appears equally bright to an observer from any viewing angle. The luminance of this surface is nearly independent of the luminance of the source of light but proportional to the illuminance of the surface. Photometric test plates exhibit the characteristics of almost uniform diffusion for most practical purposes.

Many surfaces, such as a mirror or highly polished steel plate, closely approximate the ideal specular surface, and many surfaces, such as white mat-finished paper or walls finished with flat white paint, would appear to closely approximate the perfectly diffuse surface at first glance. However, closer inspection reveals that these surfaces behave as diffuse surfaces only if the angle of incidence is close to 0 degrees as measured from the normal to the surface. Large angles of view will also cause these surfaces to exhibit properties unlike those of a diffuse surface.

Most surfaces encountered in everyday life fall between the ideal specular and ideal diffuse surfaces and exhibit properties of mixed reflection. These surfaces form no geometric image but act somewhat as a diffuse surface, showing some preference as to direction of reflection. The luminance of such a surface changes with changes in angle of incidence and observer viewing angle. The larger these angles become, the more noticeable are their effect.

Roadway surfaces where observer viewing angles and angles of incident light (as measured from the normal) range from 86 to 89 degrees and from 0 to 87 degrees, respectively, exhibit characteristics of mixed reflection. A single luminaire suspended over a roadway produces a single luminous patch on the surface of the roadway. To the observer traveling on the roadway, this luminous patch has the form of a T with the tail extending toward the observer. The luminous patch is almost completely on the observer's side of the luminaire because the reflecting properties of the pavement surface are such that only a small amount of the light striking the surface in a direction away from the observer is reflected back toward the observer. The tail of the T always extends toward the observer regardless of his position on the roadway. The size, shape, and luminance of the T depends to a great extent on the surface characteristics of the pavement. For a mat surface, the head of the T predominates, and only a short tail is evident; a surface polished smooth by traffic, however, exhibits a long tail and a small head. On a wet roadway, the head may completely disappear and the tail may become very elongated.

VEILING LUMINANCE

Roadway lighting designers must also take into consideration the veiling luminance (glare) produced by the lighting system itself. A discontinuity of brightnesses within the field of view is caused by the luminaires for most roadway lighting conditions. This results in stray light within the eye, which, in turn, produces a veiling luminance that is superimposed on the retinal image of the object to be seen, thus reducing the apparent brightness of the object as well as the background against which it is viewed. The ability of the driver to perform visual tasks is thereby reduced (1).

ROADWAY LIGHTING CALCULATIONS

Levels of illuminance are relatively easy to determine either by measurement or calculation. In the past, the derivation of roadway luminance data from photometric data required time-consuming and tedious measurement of pavement reflectance factors as well as a great number of calculations. However, recent technical developments have greatly simplified the data collection task and laboratory reflectance data are now available for a wide variety of pavement surfaces (3). Calculation procedures and computer programs have also been developed and reported together with methods for determining glare (4-8). However, the programs have often been limited to mainframe computers, and there has been no universal agreement with regard to computational methods.

The Standard Practice includes procedures and formulas for calculating illuminance, luminance, and veiling luminance. The critical expressions are given in the following text. (Appendixes B and C of the Standard Practice may be referred to for their derivation and a more detailed discussion.)

The unit of measurement of illuminance (E) is the lux, which is equal to 1 lumen per m^2. As previously stated, illuminance is the measure of the amount of light flux striking a surface. When the incident light strikes the surface an at angle, the horizontal component of the illuminance (E_h) can be expressed as

$$E_h = [I (\cos \gamma)]/D^2 \qquad (1)$$

where

I = luminous intensity in candelas,
γ = angle of incidence, and
D = distance from the source.

The surface luminance (L) is the luminous flux per steradian reflected by a unit area of surface in the direction of an observer. In general terms, the surface luminance can be expressed as

$$L = [(1/\pi) E_h] \, [q(\beta, \gamma)] \quad (2)$$

where E_h is the horizontal illuminance in lux, and $q(\beta, \gamma)$ is the directional reflectance coefficient for angles of incidence β and γ.

The preceding equations may be applied to the typical roadway lighting situation shown in Figure 1. For the single luminaire shown, the horizontal illuminance at point P can be expressed as

$$E_h = [I(\phi, \gamma)] \, [\cos \gamma] / D^2 \quad (3)$$

and

$$D^2 = H^2 / (\cos^2 \gamma) \quad (4)$$

where H is the mounting height of the luminaire. Substitution for the D^2 value in Equation 3 gives

$$E_h = [I(\phi, \gamma)] \, (\cos^3 \gamma)/H^2 \quad (5)$$

Further substitution of E_h of Equation 5 into Equation 2 gives the following expression for calculating luminance:

$$L = (1/\pi)[q(\beta, \gamma)] \, [I(\phi, \gamma)] \, (\cos^3 \gamma)/H^2 \quad (6)$$

In practice $[q(\beta, \gamma)][\cos^3 \gamma]$ is expressed as a single luminance coefficient r, and is usually given in tabular form for various types of road surfaces. The luminous intensity $[I(\phi, \gamma)]$, may be determined from published candela tables or obtained from luminaire manufacturers. A simplified expression for L can then be written as:

$$L = (1/\pi) \, [r(\beta, \gamma)] \, [I(\phi, \gamma)] / 10{,}000 \, H^2 \quad (7)$$

The angles β, γ, and ϕ are as shown in Figure 1. The 10,000 in the denominator of Equation 7 reflects the fact that tabulated r values are multiplied by 10,000 for ease of manipulation.

For a typical roadway lighting situation, both the illuminance and luminance at point P is contributed to by several luminaires. When this is the case, the illuminance and luminance values at point P represent the sum of contributions from all luminaires.

Appendix C of the Standard Practice gives a brief description of both discomfort glare and disability glare, which are the two types of glare encountered in most roadway lighting systems. While discomfort glare produces a sensation of ocular discomfort, it does not reduce the ability to see. No system for evaluating discomfort glare has been universally adopted and there is no widely accepted procedure for calculating discomfort glare. However, agreement has been reached with regard to calculating disability glare or veiling luminance (6). The veiling luminance (L_v) for the single luminaire shown in Figure 2 can be expressed as follows:

$$L_v = 10 \, E_v / (\theta^2 + 1.5 \, \theta) \quad (8)$$

where E_v is the vertical illuminance in the plane of the pupil of the observer's eye, in luxes, and θ is the angle between the line of sight and the luminaire, in degrees.

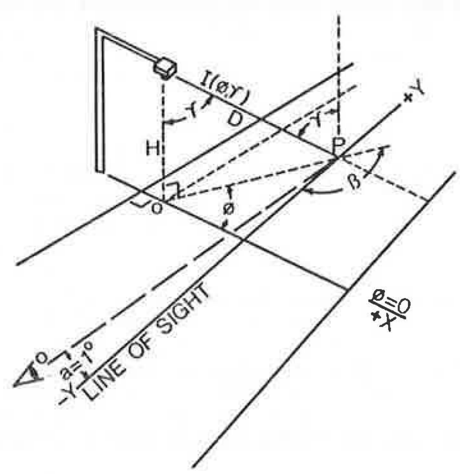
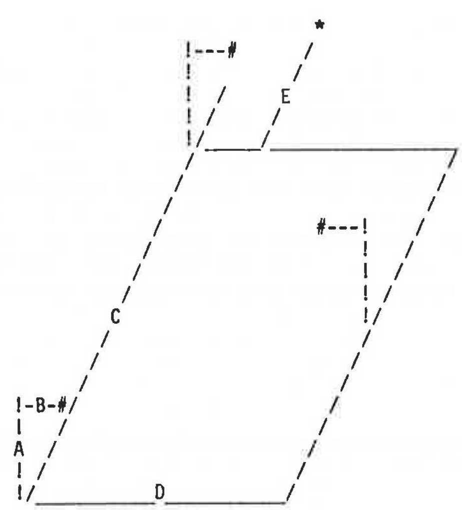


FIGURE 2 Veiling luminance angles.

This empirically derived expression can then be used to calculate the total veiling luminance for a system of luminaires by summing the individual contributions.



A Mounting Height
B Overhang
C Spacing
D Roadway Width
E Viewing Distance
* Observer
# Lamp

FIGURE 1 Reflectance angles.

THE MICROCOMPUTER PROGRAM

The previously mentioned formulas have been used to develop a roadway lighting program that is compatible with the design and evaluation procedures recommended in the Standard Practice. The program calculates values for illuminance, luminance, and veiling luminance by using input data that includes pavement directional reflectance factors, lamp/luminaire candela arrays, and geometry of the system. These values are determined at regularly-spaced calculation points between two adjacent luminaires for a dynamic observer moving through the system while maintaining a fixed viewing distance of 83m.

Written in Microsoft's BASIC-80 language, the program requires a control program for microcomputers (CP/M), a disk operating system, a minimum random-access memory (RAM) of 64K, and two disk drives. The Microsoft BASIC and program are stored on the first disk while the second disk contains pavement reflectance factor tables and candela arrays. The program's versatility and flexibility are shown by the functions it performs:

1. The program calculates values for the parameters: illuminance; luminance; and veiling luminance, including average values, maximum and minimum values, and ratios of these.
2. A grid system of calculation points on the roadway is established for the moving observer in accordance with Standard Practice procedures.
3. The lighting systems being evaluated may include one side, opposite, or staggered luminaire arrangements. A single luminaire can also be evaluated for area lighting.
4. Roadway width, lane width, and number of lanes may be specified and a median may be included.
5. Values for luminaire spacing, mounting height, and overhang may be specified as well as a light loss factor.
6. The results of the calculations are presented in the form of a printout of the highway grid calculation points and summary tables.

On initiation, the program gives an introduction and then displays the roadway lighting system diagram shown in Figure 3. A brief description of each

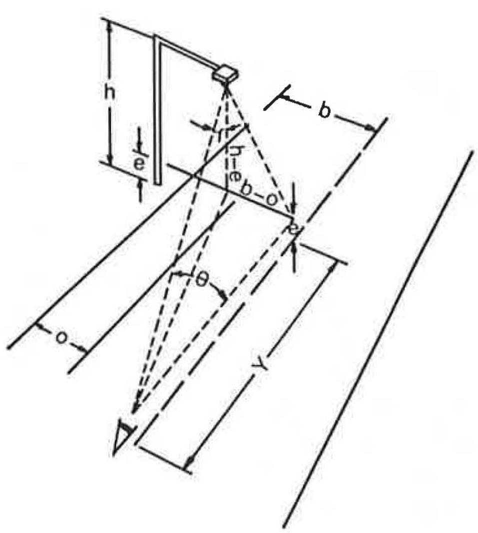


FIGURE 3 Lighting system diagram.

of the input parameters is given and the user is asked to supply a value for each. If needed, the lighting system diagram can be recalled at any time for review. After all input parameters have been assigned a value, they are listed as shown in the following table:

Input Parameter	Assigned Value
Road surface	R3
Luminaire number	1
Light loss factor	0.8
Configuration	Opposite
Spacing	45
Mounting height	12.0
Overhang	3.0
Roadway width	24.0
Lane width	4.0
Number of lanes in direction of travel	3
Viewing distance	83
Longitudinal grid spacing	4.5

If so desired, the user may change any of the values before proceeding. The following input parameters are defined in accordance with the Standard Practice, and metric units are used exclusively:

1. Road surface--The user may select any of the four standard road surfaces for which reflectance data are given in the Standard Practice. The corresponding r-Tables are stored within the program and additional tables may be added.
2. Luminaire number--The example luminaire/lamp intensity distribution shown as Table B5 in the Standard Practice is stored within the program. The user may add additional candela arrays in the same format.
3. Light loss factor--The total light loss factor should include all factors that reduce the original output of the selected luminaire/lamp. This factor may range from 0.1 to 1.0.
4. Configuration--One-sided, staggered, and opposite arrangement, as shown in Figure 2 of the Standard Practice, are provided for within the program. The median arrangement is not directly provided for, but can be produced by manipulation of the one-sided arrangement. A single luminaire configuration is also included for area lighting.
5. Spacing--Luminaire spacing is the longitudinal distance measured between adjacent luminaires, as shown in Figure 2 of the Standard Practice. This distance may range from 5 to 300 m.
6. Mounting height--Mounting height is measured from the luminaire light center to the pavement surface, as shown in Figure 2 of the Standard Practice. This distance may range from 3 to 20 m. At the present time, no provision is made for high mast cluster lighting.
7. Overhang--Luminaire overhang is measured transversely from the pavement edge or curb, as shown in Figure 2 of the Standard Practice. This distance may range from 0 to 15 m.
8. Roadway width--The roadway width is the transverse distance between pavement or curb edges. If a median is present, it is included in this total distance, which may range from 2 to 60 m.
9. Lane width--All lanes are assumed to be of equal width, ranging from 1 to 60 m. The total width of all lanes, plus any median, cannot exceed the roadway width upper limit of 60 m.
10. Number of lanes in direction of travel--This parameter provides for highways with an unequal number of lanes in each direction.

The following two parameters are not input by the user; however, they are included in the preceding table as a reminder.

1. Viewing distance--The observer viewing distance is fixed at 83 m. This corresponds to an eye height of 1.45 m and a line of sight downward 1 degree below horizontal that is parallel to the

roadway edge along quarter-points as shown in Figure B7 of the Standard Practice.

2. Grid spacing--The number shown here is the longitudinal distance between calculation points. In accordance with the Standard Practice, it is determined by dividing the space by 10, not to exceed 5m between points, with a minimum of 10 points.

All calculations are carried out by using procedures and formulas recognized in the Standard Practice. Illuminance values for a given point are calculated with the user-specified lamp/luminaire candela array and the user-specified system geometry. At least 1 luminaire behind and at least 3 luminaires ahead of a test point are considered to contribute to the illuminance at that point. Pavement luminance values are calculated by using reflectance values for the user-specified road surface, geometry of the lighting system, and Standard Practice observer position. As in the illuminance calculations, at least 1 luminaire behind and at least 3 luminaires ahead of a calculation point are considered to contribute to the luminance of the point. The calculations for veiling luminance (a) are based on the total amount of light from all luminaires directed toward the eye as the observer moves through the system, and (b) include consideration of any shielding provided by the roof of the automobile. The program calculates (a) values for individual test points between adjacent luminaires, and (b) average values for the test points, and (c) various ratios of interest.

PROGRAM RESULTS

Figures 4 through 6 show the program's output for the roadway lighting system listed in Table 1. The luminaires are located on a 24-m-wide, 6-lane roadway in an opposed arrangement. The spacing between luminaires is 45 m, the mounting height is 12 m, and the overhang is 3 m. The moving observer is viewing the roadway 83 m ahead as he travels through the system. The light loss factor is 0.8 and calculation points are at longitudinal intervals of 4.5 m. The candela array is from Table B5 of the Standard Practice and the road surface is Standard Surface R3 from Table 1 of the same document. This is a typical system for a major street located in a commercial area.

The illuminance values shown in Figure 4 are acceptable for a major street in a commercial area, as recommended in Table 2 of the Standard Practice.

ILLUMINANCE (LUX)

PAVEMENT NO = 3 LUMINAIRE NO = 1 LIGHT LOSS = 0.8

OPPOSITE ARRANGEMENT DYNAMIC OBSERVER

LUMINAIRE SPACING	MOUNTING HEIGHT	OVERHANG	ROADWAY WIDTH	LANE WIDTH
45.0	12.0	3.0	24.0	4.0

TRANSVERSE DISTANCE

	Lane # 1		Lane # 2		Lane # 3	
LONGITUDINAL DISTANCE	1.0	3.0	5.0	7.0	9.0	11.0
0	27	29	30	35	35	34
4.5	18	25	30	33	30	31
9	15	18	22	22	20	18
13.5	12	13	14	14	14	12
18	9	11	11	12	12	11
22.5	7	10	10	11	12	12
27	9	11	11	12	12	11
31.5	12	13	14	14	14	12
36	15	18	22	22	20	18
40.5	18	25	30	33	30	31
45	27	29	30	35	35	34

AVERAGE ILLUMINANCE = 20
MAXIMUM ILLUMINANCE = 35
MINIMUM ILLUMINANCE = 7
MAXIMUM / MINIMUM = 4.7
AVERAGE / MINIMUM = 2.7

FIGURE 4 Illuminance values.

LUMINANCE (CD/SQ.M.)

PAVEMENT NO = 3 LUMINAIRE NO = 1 LIGHT LOSS = 0.8

OPPOSITE ARRANGEMENT DYNAMIC OBSERVER

LUMINAIRE SPACING	MOUNTING HEIGHT	OVERHANG	ROADWAY WIDTH	LANE WIDTH
45.0	12.0	3.0	24.0	4.0

TRANSVERSE DISTANCE

	Lane # 1		Lane # 2		Lane # 3	
LONGITUDINAL DISTANCE	1.0	3.0	5.0	7.0	9.0	11.0
0	0.731	0.730	0.936	1.058	1.334	1.383
4.5	0.502	0.552	0.747	0.988	0.983	1.024
9	0.613	0.672	0.865	0.957	0.928	0.884
13.5	0.605	0.659	0.750	0.808	0.839	0.755
18	0.570	0.625	0.705	0.753	0.803	0.785
22.5	0.579	0.756	0.793	0.884	0.935	0.992
27	0.840	1.063	1.008	1.092	1.135	0.965
31.5	1.002	1.120	1.151	1.185	1.167	1.138
36	0.876	0.919	1.159	1.241	1.238	1.287
40.5	0.718	0.791	1.065	1.339	1.357	1.449
45	0.731	0.730	0.936	1.058	1.334	1.383

AVERAGE LUMINANCE = 0.9
MAXIMUM LUMINANCE = 1.4
MINIMUM LUMINANCE = 0.5
MAXIMUM / MINIMUM = 2.9
AVERAGE / MINIMUM = 1.9

FIGURE 5 Luminance values.

VEILING LUMINANCE (CD/SQ.M.)

PAVEMENT NO = 3 LUMINAIRE NO = 1 LIGHT LOSS = 0.8

OPPOSITE ARRANGEMENT DYNAMIC OBSERVER

LUMINAIRE SPACING	MOUNTING HEIGHT	OVERHANG	ROADWAY WIDTH	LANE WIDTH
45.0	12.0	3.0	24.0	4.0

TRANSVERSE DISTANCE

	Lane # 1		Lane # 2		Lane # 3	
LONGITUDINAL DISTANCE	1.0	3.0	5.0	7.0	9.0	11.0
0	0.119	0.142	0.127	0.097	0.088	0.095
4.5	0.171	0.205	0.171	0.129	0.118	0.121
9	0.244	0.289	0.222	0.163	0.138	0.111
13.5	0.318	0.337	0.257	0.180	0.119	0.084
18	0.017	0.025	0.030	0.028	0.019	0.012
22.5	0.022	0.030	0.036	0.033	0.024	0.017
27	0.032	0.036	0.042	0.038	0.029	0.021
31.5	0.039	0.046	0.051	0.042	0.034	0.029
36	0.059	0.068	0.069	0.057	0.049	0.048
40.5	0.084	0.095	0.091	0.073	0.065	0.071
45	0.119	0.142	0.127	0.097	0.088	0.095

AVERAGE VEILING LUMINANCE = 0.095
MAXIMUM VEILING LUMINANCE = 0.337
MINIMUM VEILING LUMINANCE = 0.012
MAX VEILING/AVG LUMINANCE = 0.359

FIGURE 6 Veiling luminance values.

TABLE 1 Calculated Luminance Values for Standard Road Surfaces

Luminance	Pavement Classification R1 (cd/m^2)	R2 (cd/m^2)	R3 (cd/m^2)	R4 (cd/m^2)	Recommended Value
Average	0.9	0.8	0.9	1.2	1.2
Maximum	1.4	1.3	1.4	1.9	
Minimum	0.4	0.5	0.5	0.6	
Maximum to minimum	3.2	2.8	2.9	2.9	5 to 1
Average to minimum	2.0	1.8	1.9	1.9	3 to 1
Veiling to average	0.395	0.411	0.359	0.280	0.3 to 1

The average illuminance of 20 lux is greater than the recommended value of 17 lux for an R3 pavement classification and the average-to-minimum illuminance uniformity ratio of 2.7 is less than the recommended ratio of 3 to 1. On the basis of an illuminance method for design, this would be an acceptable lighting system for the specified conditions; however, the luminance and veiling luminance values shown in Figures 5 and 6 do not meet the Standard Practice recommended values. Although the maximum to minimum luminance value of 2.9 and average-to-minimum luminance value of 1.9 are acceptably less than the recommended ratios of 5 to 1 and 3 to 1, respectively, the average luminance of 0.9 cd/m^2 is unacceptable when compared to the recommended value of 1.2 for an R3 pavement classification. In addition, the veiling luminance-to-average luminance value of 0.359 exceeds the recommended ratio of 0.3 to 1.

COMPARISON OF STANDARD SURFACES

In view of the previously mentioned results and as a further demonstration of the program, the output for these results and three additional runs are summarized in Table 1. The geometry of the system and the luminaire/lamp were held constant and only the road surface was varied for each run. In each case, the lighting system would be acceptable based on an illuminance method for design; however, acceptable levels of luminance would be produced only with Standard Pavement Surface R4. This is attributable to the mostly specular mode of reflectance for the smooth textured asphalt surface as compared with the slightly specular, mixed, or diffuse mode of reflection for the other three standard surfaces.

SUMMARY

With the standardization of computational methods and procedures and the ever-increasing availability of the microcomputer, it is now possible to eliminate compromises and shortcut procedures previously used in designing and evaluating roadway lighting systems. Thus, it is practical for engineers involved in roadway lighting design and evaluation to base their recommendations on luminance considerations, rather than on illuminance. The microcomputer program presented here combines readily available lamp/luminaire candela arrays with pavement directional reflectance factor data to calculate illuminance, luminance, and veiling luminance in accordance with the Standard Practice. It is anticipated that this design and evaluation program will be of use to the practicing engineer in providing a better night driving environment through improved roadway lighting.

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