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Contents

Foreword

The papers contained in this Record are from the 1992 Annual Meeting of the Transportation Research Board and are concerned with traffic control devices, rail/highway grade crossing safety and research, and highway visibility.

Readers with an interest in traffic signing and signalization will find papers related to alternative methods for providing lane assignment information on urban guide signs, legibility distance associated with fiber-optic changeable lane-use signs, effectiveness of active advance warning signs at high-speed signalized intersections, appropriate traffic signal display during malfunction of traffic signal control equipment at signalized intersections, comparison of service volumes and delay resulting from different types of left turn phasing strategies, the use of fuzzy set theory to analyze driver behavior during the signal change interval, and research into low-cost innovative traffic control devices at passive rail/highway crossings on low-volume roads.

Highway visibility and illumination are also discussed. The reader will find papers related to the nighttime legibility and conspicuity of reflective license plates and the development and field testing of a system for measuring light levels from vehicle headlights traveling on highways.

The papers presented in this Record were sponsored by the following TRB committees: Traffic Control Devices, Visibility, and Railroad-Highway Grade Crossings.

Study of Urban Guide Sign Deficiencies

QUINN BRACKETT, R. DALE HUCHINGSON, NADA D. TROUT, AND KATIE WOMACK

A survey administered to a sample of 662 volunteers at the 1990 Houston Auto Show compared alternative methods for providing lane assignment information on urban guide signs. Signing elements that were studied included the white down-arrow used for optional lane usage; the black down-arrow used in the Exit Only lane; the organization of route numbers and destinations; a comparison of the *Manual on Uniform Traffic Control Devices* diagrammatic guide sign and modified diagrammatics with separate arrows for each lane and with arrow shafts exceeding the number of lanes; and the use of Next Left and Next Right on conventional guide signs. The conventional diagrammatic guide sign was found to be less effective in communicating lane assignment than the modified diagrammatic signs tested. Downward arrows failed to communicate the intended optional usage message. The communication of optional exit lanes was confused by the number and position of arrow shafts displayed. Common routes displayed on an exit guide sign were less effectively displayed side by side than vertically arrayed. The array of information on diagrammatic signs was determined to equal in importance the information on the diagrammatic. Next Right and Next Left were interpreted as mandatory exits by a significant portion of the respondents.

Increased traffic has generated greater demand on existing freeways and their information systems. For example, exit ramps in many urban areas are no longer capable of handling the traffic for which they were designed. The demand today calls for two-lane exit ramps. However, signing for two-lane exits has not been established as a standard practice. In some cases, terminology used on freeway signs is not consistent. Freeways often have a local name and a route number, and mixed use is confusing to the motorist unfamiliar with it. Beltway, loop, circle, belt, and bypass are all descriptors used in various parts of the country to describe certain freeways. Drivers must read and respond to trailblazers for arterial routes while usually operating under a heavy driver work load. Geometric features such as bifurcations are complex and drivers require a sufficient amount of information processing time to respond correctly.

In this study, a survey was conducted to gain more information about several guide sign deficiencies that had been identified in previous research. It was intended that this survey compare alternative methods for providing lane assignment information on guide signs.

STUDY METHOD

Data were collected during the Houston Auto Show held March 24–April 1, 1990. Each survey consisted of an individual presentation of seven depictions of guide signs and associated questions. The questions were in the form of statements. Respondents were to indicate agreement or disagreement with statements regarding which lane or lanes could be used to exit or to continue on the interstate. On each computer-generated sign the route numbers and destination cities were fictional to prevent respondents from recognizing a specific sign. However, the stimulus material presented consisted of representations of guide sign formats that are actually being used and are in compliance with the *Manual on Uniform Traffic Control Devices* (MUTCD).

Two different sets of surveys, Set A and Set B, were each administered to approximately half of the respondents. Four signs and their associated questions were duplicated on both sets of surveys. Each of the two sets was further divided into four subsets. The subsets differed only in the order of questions. This procedure was to ensure that no carryover effects influenced results. Otherwise, the signs and associated questions were identical.

Topics Investigated

Signs A, B, and C (Figure 1) were used to compare understanding of signing elements and overhead lane positions on two-lane exiting guide signs. Of specific interest was the white down-arrow for optional lane usage, the black down-arrow in the Exit Only lane, organization of route numbers and destinations, and overhead lane position.

Signs D and E (Figure 2) compared understanding of the MUTCD diagrammatic guide sign and the modified diagrammatic with separate arrows for each lane. Sign F (Figure 3) dealt primarily with understanding of Next Left on conventional guide signs. Signs G and H (Figure 4) dealt with understanding the modified diagrammatic when the number of arrow shafts exceeded the number of lanes shown. Also of interest were how the detail designs of signing elements affected understanding of a single-lane, optional left exit guide sign and two-lane, Pull-Through arrowheads.

Signs G and H tested the effect of a right-hand guide sign sharing the sign bridge with a four-lane modified diagrammatic. Of specific interest was the understanding of Next Right in conjunction with the fourth arrow.

Because the modified diagrammatic is less common, another objective was to test the effect of a prior explanation of how to interpret its meaning. It was predicted that a brief explanation would greatly enhance the understanding of lane usage. Five signs (D, G, H, I, and J) included the modified diagrammatic. Half of the respondents were termed "the informed group." They were shown a sample diagrammatic, not used in the survey, and a printed explanation of how to read and interpret it. The other half of the participants, termed

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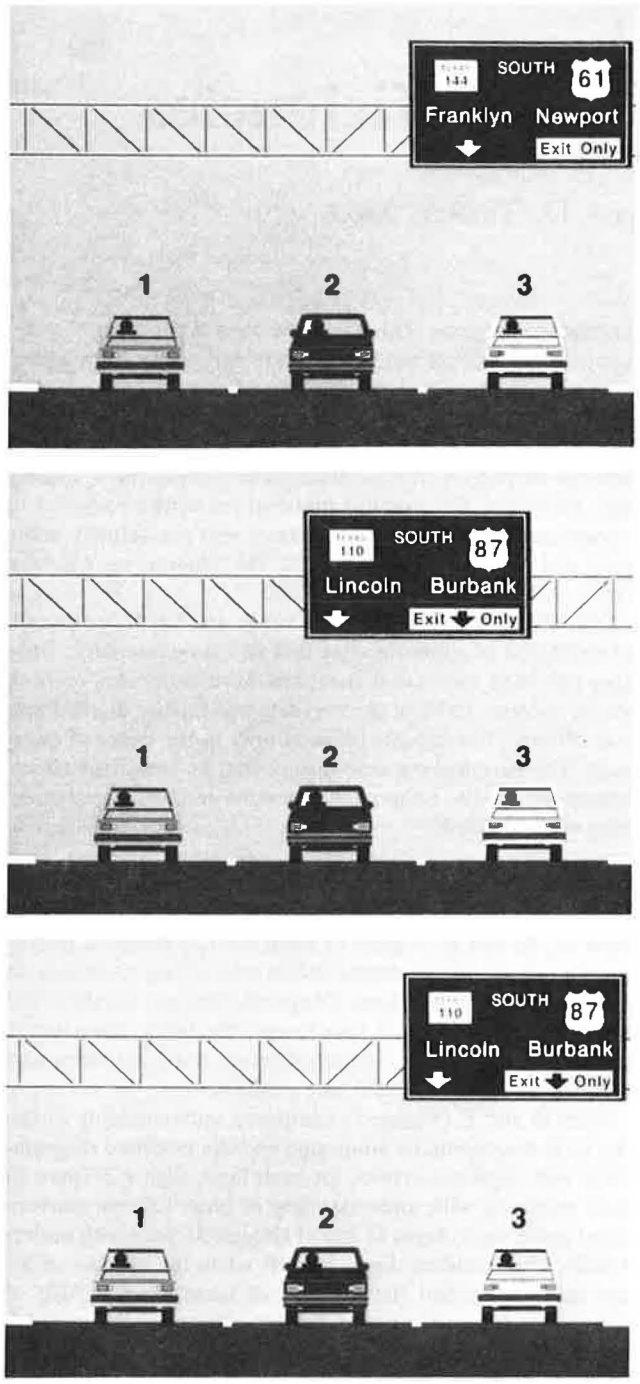

FIGURE 1 Signs A, B, and C (*top*, *middle*, and *bottom*, respectively) were used to compare comprehension of sign elements and positioning on two-lane exit signs.

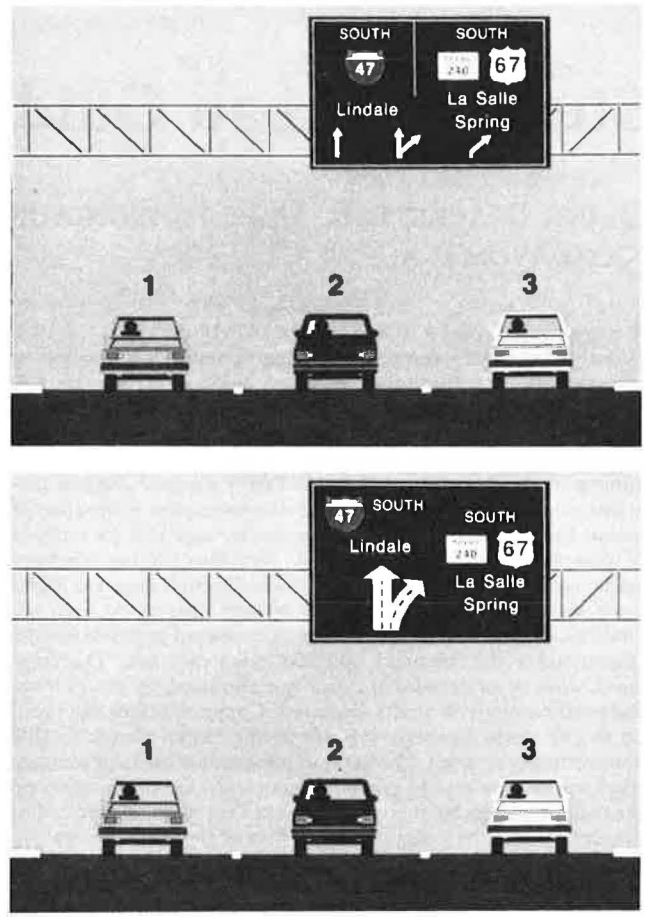

FIGURE 2 Signs D and E (*top* and *bottom*, respectively) were used to compare comprehension of MUTCD and modified diagrammatics.

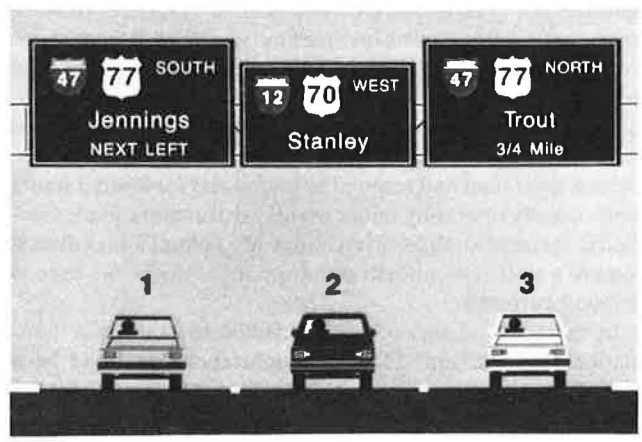

FIGURE 3 Sign F was used to determine comprehension of Next Left on conventional signs.

"the uninformed group," received no explanation. Each group was given the same signs and questions.

Procedure

Two staff members were present to administer the survey. One person was in charge of administering Set A, while the other administered Set B. Informed and uninformed surveys were given in sets of fours. Hence, four uninformed surveys were followed by four informed surveys. The participants were told that there were no right or wrong answers and that their responses would be confidential. They were then shown the depictions of guide signs and asked to check each statement they deemed true based on their understanding of the

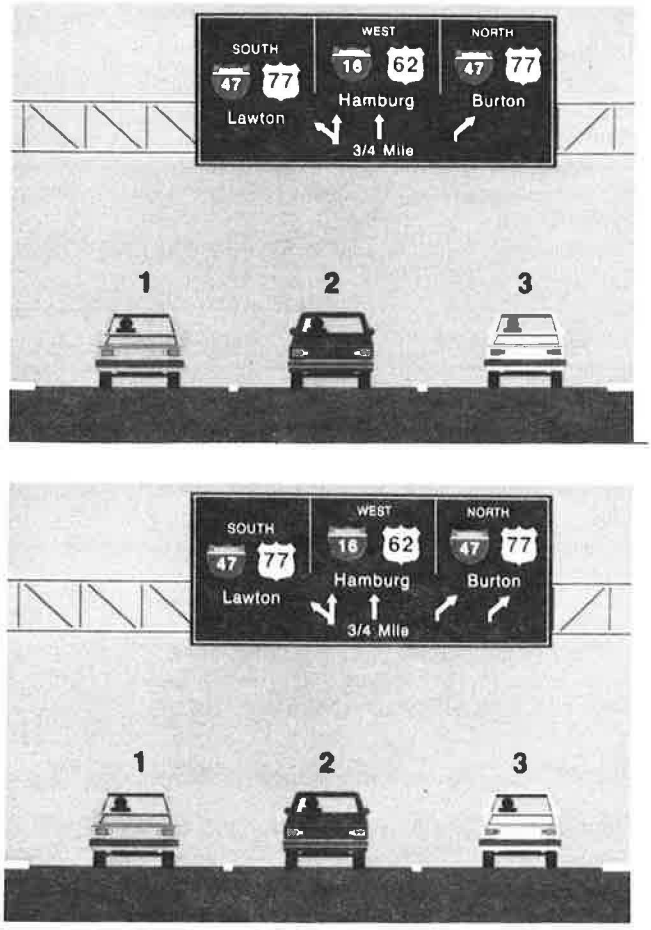

FIGURE 4 Signs G and H (*top* and *bottom*, respectively) were used to determine comprehension when number of arrow shafts exceeded number of lanes.

signs. It was explained that there could be more than one true statement for a set of statements relating to a given figure. A total of 662 surveys were completed.

Problems in Administration

Several problems were encountered during the survey. One major problem was the noise that was generated from the surrounding booths. The traffic safety section at the automobile show consisted of 18 different booths, including two seat belt convincers, an air compressor to refill air bags, and a singing puppet show. This made it difficult to hear, and it may have affected the respondents' ability to concentrate while completing the survey. It should be noted that the noise factor may have also affected the comparison results of the uninformed and informed participants. Originally the staff members were going to read aloud the paragraph informing the motorist how to read a diagrammatic sign properly. Because of the loud noise from surrounding booths, it was impossible to do so. Since motorists have varying literacy skills, this may be a factor to consider in the analysis.

It should also be noted that several of the participants may not have understood the meaning of the words "urban" and "downstream." "Urban" was used in the second question in the demographic section. The question was, "How often do you travel on urban freeways?" The word "downstream" was used for Sign G only, and the question was, "What do you think happens downstream that made this difference possible?"

Data Analysis

The data collected at the automobile show were placed into a database file and later converted into Statistical Analysis System (SAS) format for further analysis. The SAS program converted frequencies into percentages and determined if a significant difference existed between groups in answering the same question.

Description of Respondents

In all, 662 visitors to the Auto Show volunteered to participate in the survey. Demographic data were collected but were not used as a basis for selection other than to ensure that all were frequent travelers on urban freeways. The characteristics of the sample are presented in Table 1.

RESULTS FOR SIGNS A, B, AND C

Objectives

The objectives of this comparison were as follows:

1. To determine drivers' understanding of the downward white arrow on the left as an indicator of optional usage of Lane 2,

TABLE 1 Description of Respondents

Gender	%	Education	%
Male	69.9	Less than High School	5.5
Female	30.1	High School	19.2
		Some College	30.9
		College Degree	44.4
Ethnicity		Years Driving	
Anglo	76.6	Less than 1	4.7
Black	5.3	1 to 5	16.0
Hispanic	9.6	More than 5	79.3
Other	8.5		
Age		Urban Freeway Driving	
Less than 25	28.3	Occasionally	17.3
25 to 55	65.7	Often	82.7
Over 55	6.1		

2. To determine if the black down-arrow embedded in the Exit Only message clarified the optional usage of Lane 2 (by emphasizing Lane 3 is for exit),
3. To determine if the position of the overhead sign was a factor in understanding, and
4. To determine if the side-by-side array of route numbers and destinations led to the assumption that Lanes 2 and 3 led to different routes.

Questions Pertaining to Signs A, B, and C

Participants were asked to respond to the following statements. The results are shown beneath each statement.

1. Lane 2 may be used to exit to Texas 144 to Franklyn (Texas 110 to Lincoln), but not to US-61 to Newport (US-87 Burbank).

Sign	*True*	*False*	*Sample Size* ($p < .05$)
A	51.60	48.40	651
B	64.35	35.65	331
C	68.62	31.38	325

2. Either Lane 2 or 3 may be used to exit to US-61 South (US-87 South).

Sign	*True*	*False*	*Sample Size* ($p < .02$)
A	12.90	87.10	651
B	18.13	81.87	331
C	11.08	88.92	325

3. If you are in Lane 3 you *must* take the next exit.

Sign	*True*	*False*	*Sample Size*
A	90.00	10.00	651
B	91.24	8.76	331
C	90.15	9.85	325

4. Lane 2 may be used to continue on IH-47 South. (Respondents were instructed that they were driving south on IH-47.)

Sign	*True*	*False*	*Sample Size* ($p < .001$)
A	75.50	24.50	649
B	74.85	25.15	330
C	62.15	37.85	325

Discussion of Results

Question 1 addressed the fourth objective: to determine if a side-by-side array of route numbers and destinations lead to the assumption of route separation. A majority of drivers believed that Lane 2 led only to Franklin (Texas 144). When the downward black arrow appeared (Signs B and C) this seemed to increase confusion, as if Lane 3 were reserved for US-87 to Burbank. This result may be because some drivers spatially clustered information with each arrow. Thus, the information on the left side of the sign is associated with the left arrow, while the information on the right side of the sign is associated with Exit Only or Exit Only with an arrow.

Question 2 addressed the first objective. Over 80 percent did not understand that the white down-arrow meant Lane 2 could be used as an exit. Moving the sign such that the white arrow was over Lane 2 (Sign C) did not improve understanding. In fact, there was significantly poorer understanding of Sign C than B.

For Question 3, 90 percent understood that Lane 3 traffic must exit. For Question 4, 75 percent agreed that Lane 2 traffic could continue when the sign was over Lane 3, but only 62 percent agreed when the sign was over Lane 2. Changing the sign position had a negative impact on understanding.

In summary, the data suggest that the white arrow does not connote optional usage. The black arrow did not aid in connoting optional usage. Positioning the sign over Lane 2 led to more misunderstanding than when it was over Lane 3. A side-by-side array of route numbers and destinations may be confusing when the down-arrow appears directly under one or both routes.

RESULTS FOR SIGNS D AND E

Objectives

The objectives of this set of questions were as follows: to compare the effectiveness of the conventional diagrammatic (Sign E) and the modified diagrammatic (Sign D) and to determine the degree to which instructions on how to read diagrammatic signs improved performance.

Questions Pertaining to Signs D and E

1. If you are in Lane 3 you *must* take the next exit.

Sign	*True*	*False*	*Sample Size* ($p = .001$)
D	82.07	17.93	329
E	71.43	28.57	322

2. Lane 2 may be used to exit to Texas 240 to LaSalle, but not to US-67 to Spring.

Sign	*True*	*False*	*Sample Size*
D	14.59	85.41	329
E	11.15	88.85	323

3. Either Lane 2 or 3 may be used to exit to US-67 South.

Sign	*True*	*False*	*Sample Size* ($p < .001$)
D	88.75	11.25	329
E	75.85	24.15	323

4. Lane 2 may be used to continue on IH-47 South.

Sign	*True*	*False*	*Sample Size*
D	92.71	7.29	329
E	90.09	9.91	323

Discussion of Results

Question 1 asked if Lane 3 traffic must exit. Significantly more respondents understood the modified diagrammatic for this application. Training had no effect on the percentages. Thus, the separate, modified up-arrow over Lane 3 better communicated that Lane 3 must exit. However, the level of understanding was below that reported for Signs A, B, and C where Exit Only appeared over Lane 3.

Question 2 was analogous to Question 1 in the previous set of signs. Over 85 percent understood that Lane 2 applied to both routes (Texas 240 and US-67). Note that here destinations are arrayed one above the other. There was no difference between the modified and conventional diagrammatic for this application.

Question 3 tested whether respondents thought Lane 2 could be used to exit to US-67. With the modified diagrammatic, significantly more respondents (13 percent) were correct than with the conventional diagrammatic. Again, training did not significantly improve the performance with the modified diagrammatic sign.

Question 4 asked if Lane 2 could be used to continue on the interstate. Ninety percent of both groups agreed. Training had no effect. It is interesting to note that these signs had pull-through route designations unlike those in Signs A, B, and C. This information coupled with the modified diagrammatic arrows increased the correct responses by at least 17 percent.

In summary, the modified diagrammatic, with separate arrows for each lane, resulted in better performance when applied to whether Lane 3 must exit or whether Lane 2 may be used to exit. When the two exit destinations were not side by side and did not have down-arrows, as in Signs A, B, and C, drivers were less likely to assume that Lane 2 led to one route and Lane 3 to another.

Training on how to read diagrammatic signs was predicted to increase understanding. However, instruction appeared to have little or no effect. One possible explanation for this finding could be the conditions of administration, which required the respondents to read the explanation and not orally demonstrate understanding. The exercise failed to teach many drivers the basic principles.

RESULTS FOR SIGN F

Objective

The objective of this set of questions was to determine if a sign over Lane 1 that displayed Next Left would imply that exiting was optional or mandatory.

Questions Pertaining to Sign F

1. Lane 1 traffic *must* exit to IH-47 South.
2. Lane 2 traffic may continue on IH-12 West.
3. Lane 1 traffic may exit to IH-47 South or may continue on IH-12 West.

Discussion of Results

The results are given in the following table:

Question	*True*	*False*	*Sample Size*
1	29.30	70.70	651
2	86.80	13.20	651
3	64.50	35.50	651

In answer to Question 1, 70.7 percent understood that exiting was not required, but a surprising 29.3 percent thought Lane 1 was for exiting only. Question 3 was essentially the same question restated in a different form. Here 64.5 percent understood that Lane 1 exiting was optional.

Question 2 asked if Lane 2 traffic could continue on the interstate. Although 86.8 percent were correct, one might have expected near perfect performance.

In summary, Next Left over Lane 1 was misinterpreted by almost a third of the drivers as being mandatory.

RESULTS FOR SIGNS G AND H

Objectives

One objective of this set of questions was to determine the effects of displaying modified diagrammatic arrows when there are more arrows than lanes shown. The actual situation was one of an added right-hand lane downstream of the overhead sign. The drivers were not given this information but were asked to speculate on why there were more arrows than lanes. Another objective was to determine the extent to which poor formatting of information and overhead placement of information in the wrong lane affects interpretation of a left, optional usage exit shown by a modified diagrammatic.

Questions Pertaining to Signs G and H

1. Lane 2 traffic may exit to IH-47 *North*.

Sign	*True*	*False*	*Sample Size* ($p = .0001$)
G	42.86	57.14	329
H	14.51	85.49	324

2. Lane 1 traffic may continue on IH-16 West.

Sign	*True*	*False*	*Sample Size*
G	56.10	43.90	328
H	54.01	45.99	324

3. Lane 1 traffic may exit to IH-47 *South*.

Sign	*True*	*False*	*Sample Size*
G	77.81	22.19	329
H	79.63	20.37	324

4. Lane 1 traffic *must* exit to IH-47 *South*.

Sign	*True*	*False*	*Sample Size*
G	25.84	74.16	329
H	24.07	75.93	324

5. Lane 2 traffic *must* continue on IH-16 West.

Sign	*True*	*False*	*Sample Size* ($p = .001$)
G	44.07	55.93	329
H	57.10	42.90	324

6. Lane 3 traffic *must* exit to IH-47 *North*.

Sign	*True*	*False*	*Sample Size*
G	85.41	14.59	329
H	84.57	15.43	324

Discussion of Results

Question 1 addressed the first objective. With only one up-arrow over Lane 3 (Sign H), 85.5 percent understood that the middle arrow referred to Lane 2 and that Lane 2 could not exit to IH-47 North. However, with two up-arrows (Sign G) 42.9 percent thought that Lane 2 traffic could exit. It is surmised that counting from the *right*, they assumed the second arrow referred to Lane 2.

Skipping to Question 5, drivers were asked if Lane 2 traffic must continue on the interstate. Only 57 percent of the responses to the Sign H group were the correct answer and even less (44 percent) of the responses to the Sign G group were correct.

Examining the elements of the sign provides several possible explanations for the poor performance. For the four-arrow group (Sign G), respondents may have assumed that both the second and third arrow referred to Lane 2. If so, traffic would have had an option to exit or continue and "must" continue was incorrect. This explanation would not apply to the three-arrow group (Sign H). The elements of both the sign and the question need be considered. The question gave only the route number (IH-16) and not the destination, "Hamburg," so the driver had to locate the small IH-16 shield. Another possibility is consistent with the findings of the first set of questions (Signs A, B, and C) in which a majority of drivers thought that the two routes displayed had separate exit lanes and the arrows accentuated this misinterpretation. Generalizing, some may have assumed that the second arrow referred to US-62 and the first arrow referred to IH-16. Regardless of the reason, performance was unexpectedly poor for both groups.

Questions 2, 3, and 4 all addressed the second objective. As expected, there was no significant difference in responses between the Sign G and H data because the issue of one or two right exit arrows did not apply to questions related to Lane 1.

When asked if Lane 1 traffic may continue on the interstate, over 40 percent answered negatively (Question 2). Evidently, respondents were not counting lanes from the left and identifying this as an optional usage lane. For Questions 3 and 4, understanding was much improved. Over 75 percent grasped the idea that Lane 1 had the option of exiting, but was not required to do so. It was somewhat paradoxical that they believed Lane 1 did not have to exit, yet 40 percent did not believe that Lane 1 traffic could continue either. Without fully answering this paradox, it is important to note the many misleading and confusing elements in this sign. First, the vertical lines suggest that the information in the middle part refers to Lane 2 only. Second, the optional usage arrow is over Lane 2 only. Third, the amount of information displayed is overloading. One must search to locate the small IH-16 interstate shield and read it. Also, single lane, left-side, optional exits may be less familiar to many drivers.

In summary, participants better understood that exiting was optional than that continuing was optional, suggesting many drivers may have been overwhelmed and confused by the formatting of the information.

The last question asked if Lane 3 traffic must exit to IH-47 North. About 85 percent of both groups answered correctly. Note that correct responses to Sign G were given equally by respondents who thought the third or fourth arrow applied to Lane 3.

Two concluding questions were asked. One question was, "Note that there are more arrows than lanes. Do you find this confusing?" The second question asked, "What do you think happened downstream that made this difference possible?"

Of 333 Set A respondents, 198 (60 percent) reported the four-arrow sign was confusing. To the write-in question about what was happening downstream, the responses were highly variable: 137 (41.1 percent) gave no answer; other responses were classified into three categories in the data analysis. The associated frequencies and percentages are as follows:

1. Partially correct—80 (24.0 percent),
2. Exactly correct—15 (4.5 percent), and
3. Ambiguous, irrelevant, or incorrect—101 (30.3 percent).

A partially correct tally was assigned for statements such as, "a lane was added on the right," "Lane 3 split into 2 lanes," "the road widens on the right," or words to this effect. Respondents grasped the notion of another lane but did not state that this lane had incoming traffic.

An exactly correct answer used verbs such as "merging, feeding in, or entering" to describe the new lane. A few stated there was a ramp or feeder road. Ambiguous responses were ones that indicated a possible lack of understanding. Irrelevant or incorrect comments included, "missed an exit," "it feeds to another road," "several forks leading to different highway," "road narrows," or mention of Lane 1 and 2. Some incorrectly said there was another exit upstream of the routes on the sign given; a few even mentioned a narrowing of the highway.

Less than 30 percent understood the meaning that four arrows indicated an added exit lane upstream on the right. And only 4.5 percent recognized that the lane would have traffic on it entering from a ramp. A majority felt it was confusing. Even those who did not report confusion were largely incorrect.

The display of more arrows than lanes in Sign G was confusing in terms of whether Lane 2 traffic could exit. Twenty-eight percent more were incorrect with four arrows as when there was one arrow per lane. An optional usage, a modified diagrammatic referring to Lane 1, failed to communicate that traffic in that lane could continue. Several explanations were offered. Failure to understand that Lane 2 could continue

with the three-arrow group was unexpected and may relate both to the question and the signing elements.

RESULTS FOR SIGNS I AND J

Objectives

The objectives of this comparison were as follows:

1. To determine if adding a guide sign over the fourth lane affected driver understanding of the Lane 3 and 4 exiting requirement [Sign I is the control group for Sign J in this comparison (Figure 5)], and
2. To determine the degree to which drivers misinterpreted Next Right as referring to a mandatory exit.

Questions Pertaining to Signs I and J

1. Lane 1 traffic *must* continue on US-83 South (US-79 South).

Sign	*True*	*False*	*Sample Size* ($p < .001$)
I	90.90	9.10	651
J	84.80	15.20	652

2. Lane 1 traffic may continue on US-83 South (US-79 South) or exit to IH-40 West (IH-60 West).

Sign	*True*	*False*	*Sample Size* ($p < .001$)
I	11.80	88.20	651
J	18.90	81.10	652

3. Lane 2 traffic *must* continue on US-83 South (US-79 South).

Sign	*True*	*False*	*Sample Size*
I	5.40	94.60	651
J	4.40	95.60	652

4. Lane 2 traffic may continue on US-83 South (US-79 South) or exit to IH-40 West (IH-60 West).

Sign	*True*	*False*	*Sample Size*
I	93.10	6.90	651
J	94.00	6.00	652

5. Lane 3 traffic *must* exit to IH-40 West (IH-60 West).

Sign	*True*	*False*	*Sample Size* ($p = .001$)
I	87.40	12.60	650
J	80.70	19.30	652

6. Lane 3 traffic may continue on US-83 South (US-79 *North*) or exit to IH-40 West (IH-60 West).

Sign	*True*	*False*	*Sample Size* ($p < .001$)
I	8.30	91.70	651
J	21.00	79.00	652

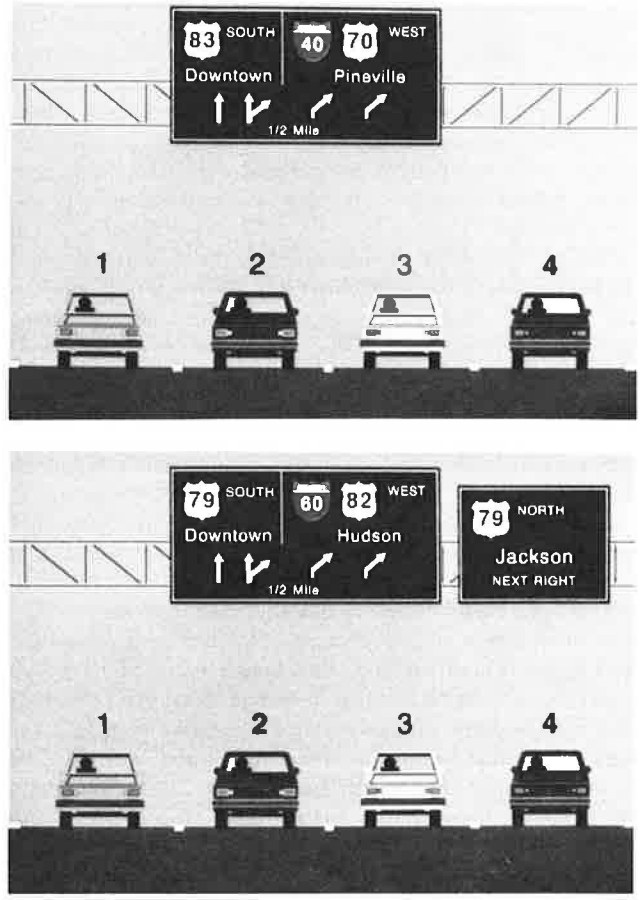

FIGURE 5 Signs I and J (*top* and *bottom*, respectively) were used to determine if a sign over Lane 4 changed comprehension of exiting requirements.

7. Lane 4 traffic *must* continue on US-79 North.

Sign	*True*	*False*	*Sample Size*
J	50.00	50.00	652

8. Lane 4 traffic may exit to US-79 North or may continue on IH-60 West.

Sign	*True*	*False*	*Sample Size*
J	44.40	55.60	651

Discussion of Results

The first two questions referred to Lane 1. Correct responses were high for both groups, but significantly higher for Sign I than Sign J. Regarding the optional usage lane, correct responses of both groups (trained and untrained) were in excess of 80 percent.

Regarding Lane 3, correct responses varied significantly (6.7 percent) between Signs I and J as to whether this lane must exit (Question 5). The guide sign in the right lane appeared to be exerting some effect on distinguishing lanes.

Because of editing errors it is not possible to compare the corollary question of whether or not Lane 3 is optional. For Sign I, 92 percent said that Lane 3 was optional (Question 6). However, Question 6 for Sign J listed the options as IH-60 West and US-79 North (rather than US-79 South). Thus, the correct response called for knowing a vehicle could negotiate into Lane 4 from Lane 3 and exit. The 79 percent correct is high, but it is not the same issue addressed in Question 6 for Sign I.

Questions 7 and 8 were asked for Sign J only and addressed Objective 2: understanding Next Right. Question 8 data are usable but Question 7 data are not because Question 7 used the word "continue" rather than "exit" for the exit to US-79 North. Compounding this problem, in the previous questions US-79 *South* was the continuing "downtown" route. Thus, if the reader did not see the cardinal direction (US-79 North) and translate "continue" as "exit," the question would be missed.

However, Question 8 was stated correctly, and 56 percent did not interpret Next Right as being optional. Recall that there was a similar although less pronounced misinterpretation of Next Left for Sign F, Question 1.

In summary, a large percentage of drivers misinterpreted Next Right as implying that Lane 4 must exit. The guide sign over Lane 4, particularly if it is viewed as an exit lane, may have exerted some influence over interpretation of the Lane 3 arrow, but had no impact on the Lane 1 and 2 arrows. The modified diagrammatics over Lanes 1 and 2 performed very well, possibly because they were simpler than those investigated in the previous questions.

SUMMARY AND CONCLUSIONS

A questionnaire, administered to a sample of volunteers at the Houston Auto Show, was designed to study several variables identified previously as being major sources of confusion in overhead guide signs. The lane assignment issues related to various signing elements, formatting of information, and overhead placement.

Previous survey research had identified high frequency problem areas. This research attempted to isolate the elements as potential contributors to misunderstanding and to measure understanding by a series of true-false questions. Questions referred systematically to each interstate lane and asked if traffic could exit from the lane, was required to exit, was required to continue on the freeway, or had a choice.

In general, the level of understanding was not as high as anticipated, particularly for signs that had been in use in Texas for many years (e.g., the white down arrow for optional usage and Next Left or Next Right messages). The large sample size and the demographics of the sample suggest that the findings are reliable. The volunteers were younger, better educated, and more experienced in freeway driving than the driving public in general. So if there was a measurement error, it would be in the direction of underestimating the true extent of misunderstanding.

One of the major findings of the study related to the conventional diagrammatic sign. Although previously suspected of having a shorter legibility distance compared to the modified separated lane arrows, the present study demonstrated that the conventional diagrammatic did not communicate lane assignment information as well, even when legibility was not an issue. Other major findings are as follows:

1. The downward white arrow on the left side of an exit sign was misinterpreted by 80 percent as an indicator that a lane has optional usage. A black down-arrow embedded in the Exit Only message did not improve understanding.
2. Moving the sign so that the downward white and black arrows are over the appropriate lanes did not improve understanding of the optional usage and, in fact, increased misunderstanding.
3. Two common routes appearing side by side on an exit guide sign misled many drivers to think that they referred to different routes to be accessed by different lanes. Adding the second black down-arrow accentuated this confusion. Arraying destinations under one another (Sign D) resulted in 85 percent responding that they were a common route.
4. The modified diagrammatic was 10 percent better than the conventional diagrammatic in indicating that the third (right-hand) lane must exit and was 13 percent better regarding an optional usage lane. The two were equally effective in connoting that the optional lane could continue.
5. A Next Left sign over a lane was misinterpreted by 30 percent as indicating a mandatory, single-lane exit.
6. When the number of arrow shafts on a modified diagrammatic exceeded the number of lanes displayed, drivers were confused about optional usage. When the number of arrow shafts equaled the number of lanes (Sign H) performance was 28.5 percent better regarding exiting from an optional usage lane. This suggests that the added lane downstream should not have been displayed on the advance sign.
7. When a modified diagrammatic was used to indicate an optional usage left-lane exit and when the arrow and other information was clustered over Lane 2, about 20 percent did not understand that Lane 1 traffic could exit and 25 percent thought traffic must exit. However, 45 percent thought traffic could continue. It is speculated that the location of the information overhead was misleading and that vertical lines accentuated the conclusion that the information did not apply to Lane 1. Too many secondary routes were displayed, forcing the reader to extract the small relevant route number from a mass of information. Diagrammatic signs need to be simplified to display only the primary routes.
8. On this same sign, misinterpretation that Lane 2 could continue on the interstate was unexpected; one explanation is in terms of the problem identified in Item 3 above. The high degree of understanding of the modified diagrammatic in Signs H and J suggests that it is not the diagrammatic itself but the array of information on the sign that may be leading to some confusion.
9. Next Right signs were misinterpreted by 56 percent of the respondents as mandatory exit. An improved message is required.
10. Although some data were lost because of miswording of two questions, some evidence supports the position that guide signs should not appear on the same bridge with a diagrammatic.
11. The effects of an educational paragraph on interpreting modified diagrammatics was not assessed because of poor conditions of administration. This issue remains unanswered.

ACKNOWLEDGMENT

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Dynamic Lane Assignment Using Fiber-Optic Signs

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The legibility distance for various fiber-optic changeable lane-use displays was evaluated. Legibility distance for the arrow shafts of the various displays was approximately 800 ft. Legibility for the word messages was near 200 ft. The word messages were identified as being a possible source of confusion to the driver, which would effectively reduce the overall efficiency of the sign in providing information. An inverse relationship between legibility and target value was identified for the sign. Decreasing the light output of a display improved the legibility but reduced the target value at practical light output settings. The selection of operational settings for a given installation must optimize this relationship to obtain acceptable levels of legibility and target value. A small legibility distance was associated with the word messages in this study. This was partly because of the width of the individual letters, the letter spacing, and the light output within the area of the word message. Design procedures should be developed that will promote uniform application of fiber-optic technology to the field of traffic signing.

Maintaining uncongested traffic conditions at intersections that exhibit wide variations in turning movements is a challenging problem for transportation agencies in many metropolitan areas. Limitations in the amount of available right-of-way (ROW) causes expansion of these facilities to become both physically and economically unfeasible. New technologies must be identified and developed that will allow the existing geometry at these locations to be utilized more efficiently.

Lane-use information at intersections is presently conveyed to drivers via pavement markings and overhead reflective signs. Problems occur at intersections that use these traffic control devices for lane-use assignments when wide variations in turning movements exist. The static nature of these devices does not allow lane usage to be optimized based on the traffic demand. The use of changeable message signs would provide a more efficient means of responding to cyclical variations in turning movements.

A sign's effectiveness is dependent on the legibility and target value of the sign. Legibility and target value of signs vary with the contrast between the sign legend and background as well as the contrast between the sign and its surroundings. Factors that contribute to a sign's effectiveness are external illumination, whether the sign reflects or emits light, and the size of the sign and its legend (*1*). Although design procedures for reflective signs have been documented throughout transportation and human factors engineering journals, the design and operation of internally lighted displays depend on basic rules of thumb and experience. Design procedures for changeable message signs are not yet well established, due largely to the rapid development of changeable message signs (*2*). Design procedures must be developed that take into account the limitations of driver visibility in both daytime and nighttime driving conditions. Liability issues further mandate that changeable message signing conform as closely as possible to requirements of the *Manual on Uniform Traffic Control Devices* (MUTCD) for signing (*2*).

Fiber-optic technology provides a viable alternative to many other types of changeable message signs. Fiber-optic displays are typically associated with the provision of higher levels of resolution, very uniform light output between individual pixels, and lower costs than are associated with other types of internally illuminated signs (*3*). A large amount of work has been done by European companies to quantify the light output of this type of sign and to develop design procedures that limit the number of pixels used to form a display based on the average pixel output (*3*). Procedures that provide engineers with the ability to design displays that can be discerned at specific distances must be developed. The development of national standards for the design of fiber-optic displays is essential to ensure that future transportation systems continue to provide information to drivers in a safe, effective, and uniform manner.

Table 1 shows the range of pixel sizes that are typically used in fiber-optic displays as well as the function for which these displays are used. The largest pixels shown in Table 1 are typically smaller than the lighting elements used in other types of internally lighted signs. Smaller pixel sizes allow symbols and words to be formed with greater resolution so that a more continuous appearance is obtained. The application of a single light source to the common end of a bundle of fiber-optic strands produces highly uniform light output for individual pixels. The high light output and intensity associ-

TABLE 1 Typical Fiber-Optic Pixel Sizes

PIXEL DIAMETER[1]	TYPICAL USE/APPLICATION
0.055"	Pedestrian Signals
0.068"	Lane Assignment/Regulatory Signing
0.090"	
0.125"	Lane Control Signing
0.177"	Word Message Signing
0.238"	Used for turn angles that need wide angle of dispersion.

[1]Obtained from the National Sign and Signal Company

W. L. Gisler, Traffic Engineers, Inc., 8323 Southwest Freeway, Suite 200, Houston, Tex. 77074. N. J. Rowan and M. A. Ogden, Texas Transportation Institute, CE/TTI Tower, College Station, Tex. 77843-3135.

ated with these pixels eliminates the "phantom effect" exhibited by other types of internally illuminated signs.

Several trade-offs must be identified and addressed when considering the use of internally lighted fiber-optic signs rather than conventional reflective signs. The ability of fiber-optic signing to produce light can be both an advantage and a disadvantage. Although fiber-optic signs provide more target value than do reflective signs, the amount of light produced by fiber-optic signs must be adjusted to ensure that the sign is legible sufficiently in advance of the point where the information is needed. A variety of information can be presented using fiber-optic displays, whereas conventional reflective signs provide only one message.

Several disadvantages associated with internally illuminated signs also exist. These disadvantages make the decision to use this type of sign highly dependent on the benefits that can be gained at the facility. These benefits include higher capital, maintenance, and operating costs. Backup systems that provide redundancy must be designed so that, in the event of a mechanical breakdown, bulb failure, or power outage, the sign will still be capable of providing a message. Internally lighted signs are also heavier than conventional reflective signs and require the development of special, more substantial supports and mast arms to accommodate the increased weight.

STUDY DESIGN AND METHODOLOGY

Research focused on the evaluation of a fiber-optic lane-use sign developed by the Texas Transportation Institute (TTI) and the National Sign and Signal Company. The sign was tested at the Texas A&M University Riverside Campus. The scope of this research was limited to a quantitative analysis of the nighttime legibility distance associated with the sign under controlled viewing conditions. A model was also developed to relate the luminous output of the displays to the voltage applied to the sign.

Laboratory Design

The laboratory was designed to provide realistic viewing conditions for a three-lane approach to an intersection. The laboratory layout shown in Figure 1 was developed through various subjective analyses by various TTI and Texas Department of Transportation (TxDOT) traffic and transportation officials of preliminary laboratory arrangements. A sign tower was used to support two three-lens traffic signal heads with 12-in. lenses, two overhead high-intensity grade retroreflective signs, and the overhead fiber-optic changeable message sign. The distance between the right edge of the signals and the sign to the left of the signals was 3 ft. External illumination was provided by placing Type II, 250-watt, high-pressure sodium luminaries 120 ft in front of and behind the sign tower.

Design Characteristics of the Fiber-Optic Sign

The sign used in this research was purchased for the purpose of evaluating (a) the light output characteristics of fiber optics, (b) the legibility distance associated with the sign displays, and (c) the target value associated with each display.

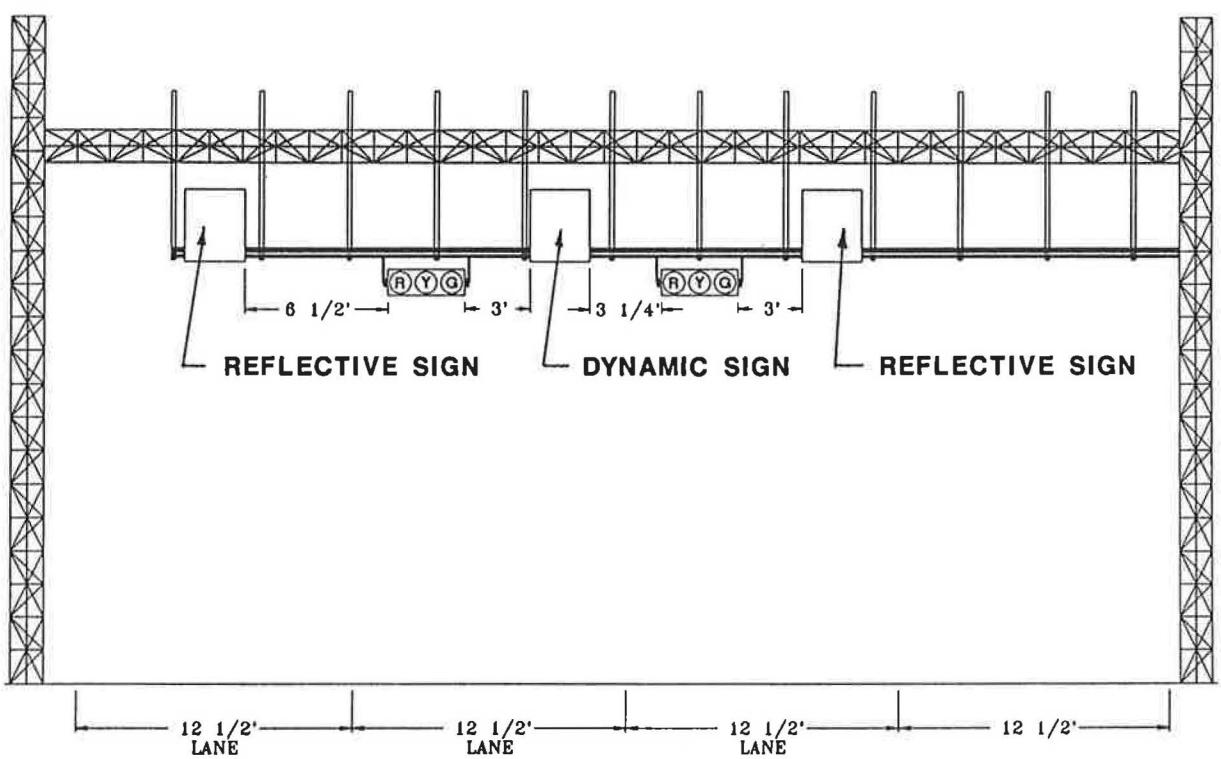

FIGURE 1 Laboratory layout used for legibility studies.

The sign was used to produce the eight different displays shown in Figure 2. Figure 3 shows a detailed layout of the face of the fiber-optic sign. Each pixel is part of a line or group of fiber-optic pixels that make up a single element of a sign display. A total of 14 lamps and transformers powered the individual lines used to form various displays. Lines were grouped as necessary to form specific displays. Two overlapping lines were used to form an arrow shaft. Pixels were arranged such that pixels from each line that formed a shaft alternated on ½-in. centers. This allowed both ½- and 1-in. pixel spacings to be evaluated.

The pixel layout allowed both a single-row and a bold or outline arrow shaft to be produced. The height and stroke width associated with bold arrow displays were designed to parallel those of arrow shaft designs used in reflective signing. The radius of the left-turn arrow was slightly larger than that of a standard R3-6L retroreflective sign. This radius was increased so that the light output would be spread over a wider expanse of the sign. This reduced the concentration of light across the sign face and provided a more legible display with higher target value than would have otherwise been obtained.

Two different stroke widths for word messages were also evaluated. The letters were 5 in. tall and conformed to letter design and spacing for standard Series-E lettering (*4*). The major concern with the design of the lettering was that the word messages not interfere with the arrow shafts because they were intended to supplement the information conveyed by the arrow shafts.

Development of Relationships Between Luminous Output and Voltage

Analytic measurements of the luminous output at specific voltages across the sign were taken. These measurements were used to develop a relationship between the voltage applied to the sign and the luminous output of each display.

Study Methodology

Three types of studies were used in this research. A pilot study was first conducted to relate the limitations of the human eye to light output levels of the light-emitting components at the laboratory. This was accomplished by evaluating the threshold intensity of the traffic signals and the point of irradiation for the fiber-optic displays. This threshold intensity corresponds to the light output of the signals that caused disability glare with respect to a person's ability to view the fiber-optic display. The point of irradiation corresponded to the voltage level that caused irradiation to occur for a given fiber-optic display.

The results of the pilot study were analyzed to determine a range of voltages for both the traffic signals and the fiber-optic displays that provided acceptable viewing conditions for the study participants. Voltage levels selected for the evaluation of the traffic signal viewing condition corresponded to the mean voltage associated with the pilot study viewings and a voltage setting used in previous subjective observations. Subjective evaluations of the fiber-optic displays were made for 35-, 50-, and 65-volt settings.

Additional subjective observations were conducted to limit the number of display factors that varied during the evaluation of legibility distance. Several variables identified prior to this survey were evaluated concerning their effect on legibility and target value. These variables included the following:

- The effects of pixel spacing,
- Differences in the formation of arrow shafts,
- The effectiveness of the sign at different levels of light output, and
- The effectiveness of the word messages.

General comments and observations concerning the overall effectiveness of the sign were also solicited.

Quantitative Evaluation of Glance Legibility

The purpose of this portion of the study was to evaluate the legibility associated with different elements of the fiber-optic

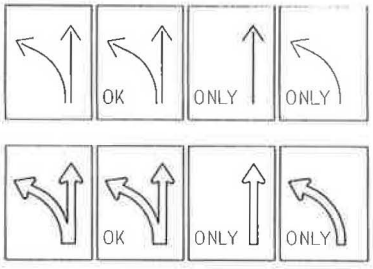

FIGURE 2 Fiber-optic test displays.

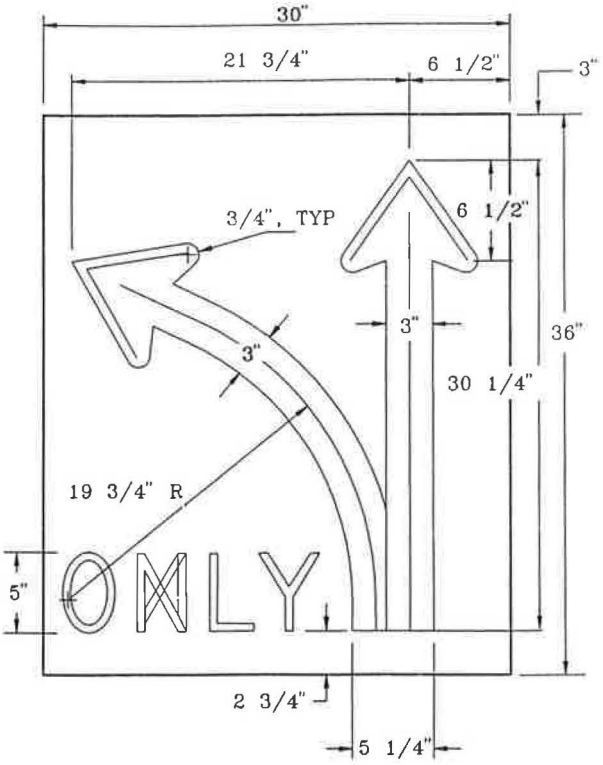

FIGURE 3 Detailed layout of the fiber-optic sign face.

sign. Glance legibility was evaluated in an attempt to provide results that would more closely represent actual driving conditions (*5*). The displays shown in Figure 2 were presented to study participants at distances from the sign of 800, 600, 400, and 200 ft. Specific settings used for the laboratory components during the legibility studies were selected based on results from the subjective observations and the pilot study. These settings included the following:

- ½-in. pixel spacings,
- No adverse weather conditions,
- 50 volts across the traffic signals,
- A constant level of external illumination, and
- Voltage settings corresponding to specific light outputs for the fiber-optic displays, as determined by subjective observations.

The study participants were instructed to travel to a distance of 800 ft from the face of the sign and park the test vehicle in the lane that lined up with the fiber-optic sign. Each participant was then informed that eight different displays would be presented on the sign and remain visible for a total of 3 sec. Each participant was instructed to view the sign long enough to identify the visual image, then look away and begin drawing the display exactly as it appeared to them. The remaining displays were presented to the subject in the same manner. The participant was then instructed to proceed to 600 ft where this procedure was repeated. Displays were presented in a random order to ensure that each display was given proper consideration by the study participants. Analysis of the data involved "grading" the drawings from each study participant to determine the glance legibility distance associated with specific elements that make up the different displays.

RESULTS

Subjective Analysis

Several factors associated with the design of the fiber-optic sign were subjectively evaluated during this part of the study. These factors were evaluated using a survey provided to the study participants. The factors that were evaluated included the following:

- The effect of luminous output on legibility and target value,
- The type of arrow design, and
- The design of word messages.

Evaluation of the legibility and target value of the sign at different voltage levels illustrated the relationship of these variables to light output. Legibility of the displays was found to decrease as the light output increased. Target value, however, was found to increase with increasing light output. The relationships between legibility, target value, and voltage indicate the existence of an inverse relationship between target value and legibility.

Subjective evaluations were made by professional observers from a distance of 300 ft. The consensus of these observers indicated a preference for the single-row arrow design rather than the bold arrow design. The difference in target value for the two types of arrow shaft designs was not believed to provide a significant advantage for the bold arrow indications over the more legible single-row alternative.

Two different word messages were viewed during this part of the study. Each word message was displayed using two different stroke widths. Different stroke widths were formed using one row (single-stroke) and two rows (double-stroke) of pixels. Double-stroke word messages could not be discerned. Some letters of the single-stroke word messages were partially legible at the 300-ft viewing distance. The observers did not, however, think that the words could be discerned if the effects of dynamic visual acuity were taken into account. The consensus of the group was that the word messages should be enlarged and possibly repositioned so that the legibility of these messages would be improved. The group preferred the single-stroke words to the double-stroke words. The double-stroke words were subsequently eliminated from further observations.

Analysis of Analytic Measurements of Luminous Output

Preliminary analysis of the analytic measurements of the luminous output for each display indicated consistent overestimation of the luminous output at low voltage levels and underestimation of the luminous output at high voltage levels. The reoccurrence of these discrepancies indicated the possible presence of power losses within the system. Consequently, regression analysis was performed using the model shown in the equation below.

$$I = m * V^2 + n * V \quad (1)$$

where

I = intensity at the sign face (candelas),
m = constant for each display that includes (a) conversion between power and intensity and (b) the resistance associated with the display,
V = voltage measured across the fiber-optic display (volts), and
n = a constant that corresponds to energy that is either not converted to light or is otherwise lost.

The coefficients of regression were equal to 1 at three significant figures. Visual analysis of the data indicates that this model slightly overestimates the luminous output at lower voltage levels. The magnitude of these discrepancies, however, did not have any practical significance (i.e., this difference in luminous output could not be detected). The discrepancy between the actual and calculated values were attributed to fluctuations in voltage for the electric generator and to the relative magnitude of these losses at low levels of light output. Consequently, the model was believed to provide an acceptable means of estimating the luminous output of each display across the range of voltage settings used in the remainder of the study.

Results of Glance Legibility Studies

Studies required participants to view eight displays at 800, 600, 400, and 200 ft. After viewing these displays, participants were required to draw the display exactly as it had appeared to them. These drawings were evaluated to determine when the participants were able to distinguish the following:

- The general format of the sign,
- The appearance of the word messages, and
- The difference between single-row and bold arrow shafts.

This information was meant to provide an estimate of the overall effectiveness of the sign with respect to glance legibility. Analysis of individual elements also allowed statements to be made concerning the effectiveness of the design of these elements.

Specific settings were utilized for the laboratory components for the purposes of evaluating glance legibility associated with each fiber-optic display. The voltage setting placed across the traffic signals was 50 volts. Glare associated with this setting provided a more comfortable viewing condition without significantly reducing the target value as compared to the 65-volt setting. Light output levels associated with 35- and 50-volt settings were selected for the evaluation of glance legibility for the fiber-optic displays. Two settings were selected so that the significance of the difference in light output at these voltages could be evaluated.

Figure 4 illustrates at what point study participants were able to discern the general format of the sign. This point was determined when word messages and arrows were identified as separate elements. Identification of this point is important since, prior to this point, word messages were typically perceived as a second or third arrow by the majority of the study participants at both voltage levels tested. At the point that participants realized the word message was a separate element, they were also able to recognize that it was not an arrow. Prior to this point the message was typically represented as a blur until it was correctly identified.

Approximately 85 percent of the participants who viewed the displays were able to discern the separation between the word message and the arrow shaft(s) at a distance of 400 ft from the face of the sign for both voltage levels. The use of smaller distance intervals would have provided a more uniform distribution for the observations. Little difference in the ability of the participants to discern this separation, therefore, is believed to have existed at the two different voltage settings that were tested.

Glance legibility distance was also evaluated for the OK and Only word messages. The size and spacing of the word messages corresponds to standard Series E lettering used for reflective signing. The arrows that make up the displays are the primary information to be conveyed to the driver. The purpose of the word messages was to supplement the information provided by the arrows. Figure 5 indicates that approximately 10 percent of the participants were able to discern the message OK farther than 200 ft away. A slightly higher number of the participants was able to read the message Only prior to this distance (see Figure 6). No difference in legibility is believed to have existed at the two different voltages for the OK message. The 35-volt setting appeared to provide slightly more legibility for the Only word message than did the 50-volt setting.

The reason for the difference in legibility between the OK and Only messages is believed to lie in the length of the word and the legibility of the "o" and the "y" letters in Only. The legibility of the "o" in both word messages and the "y" in Only was much better than that of the "k," "n," and "l."

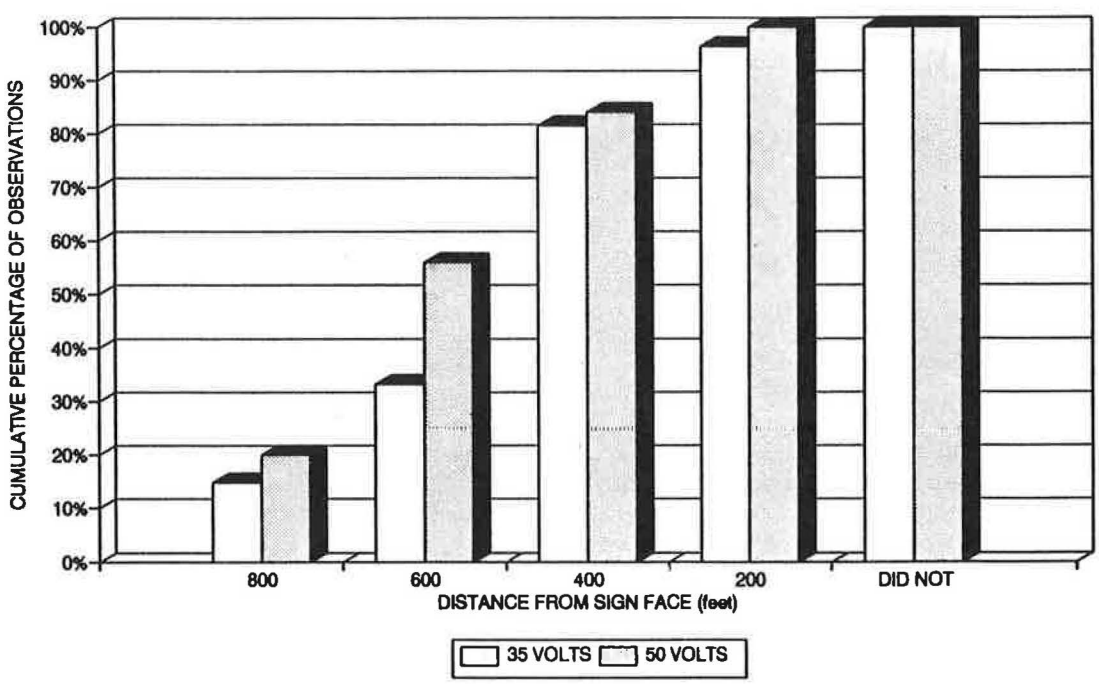

FIGURE 4 Glance legibility for general format of sign.

These letters had better legibility because of their width, outside position in the message, and because of their simple shapes, especially at the 35-volt setting. Once participants were able to distinguish the "o" and "y" in the Only indication, it is believed that they inferred the remaining letters within this message based on their previous experience. Because the OK word message was only two letters long and because of the complexity of the design of the letter "k," participants were believed to have had more difficulty in making these inferences. Several participants identified the word messages as saying "On" and "Off" prior to the distance at which they were actually able to discern this meaning. These

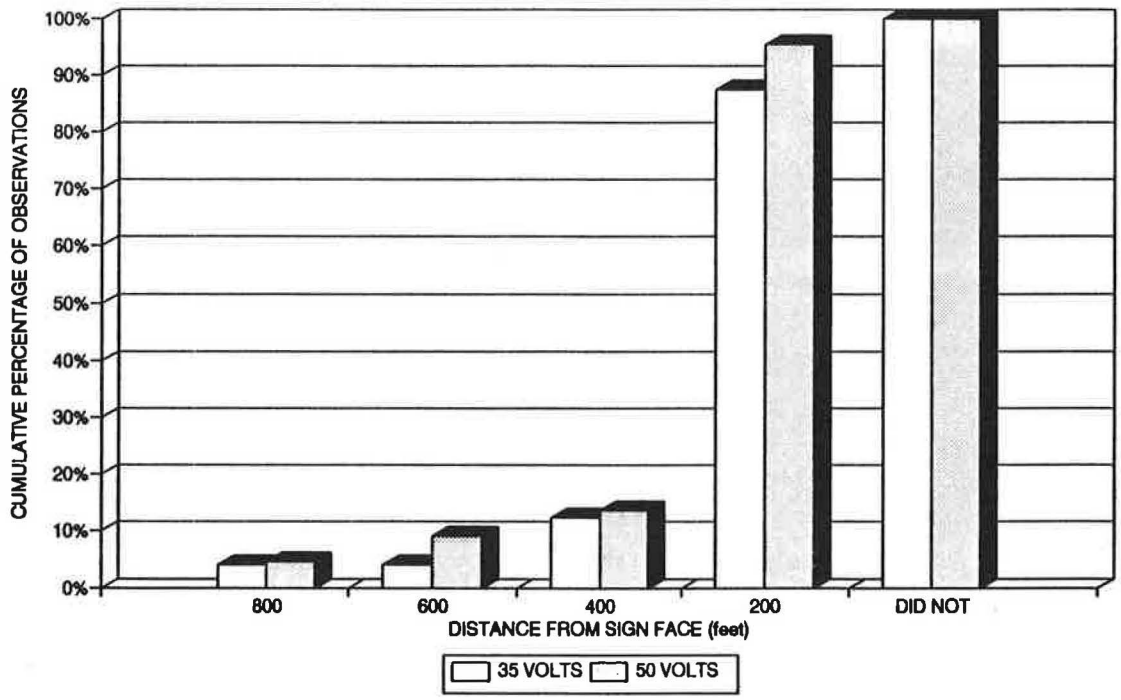

FIGURE 5 Glance legibility distance for OK word message.

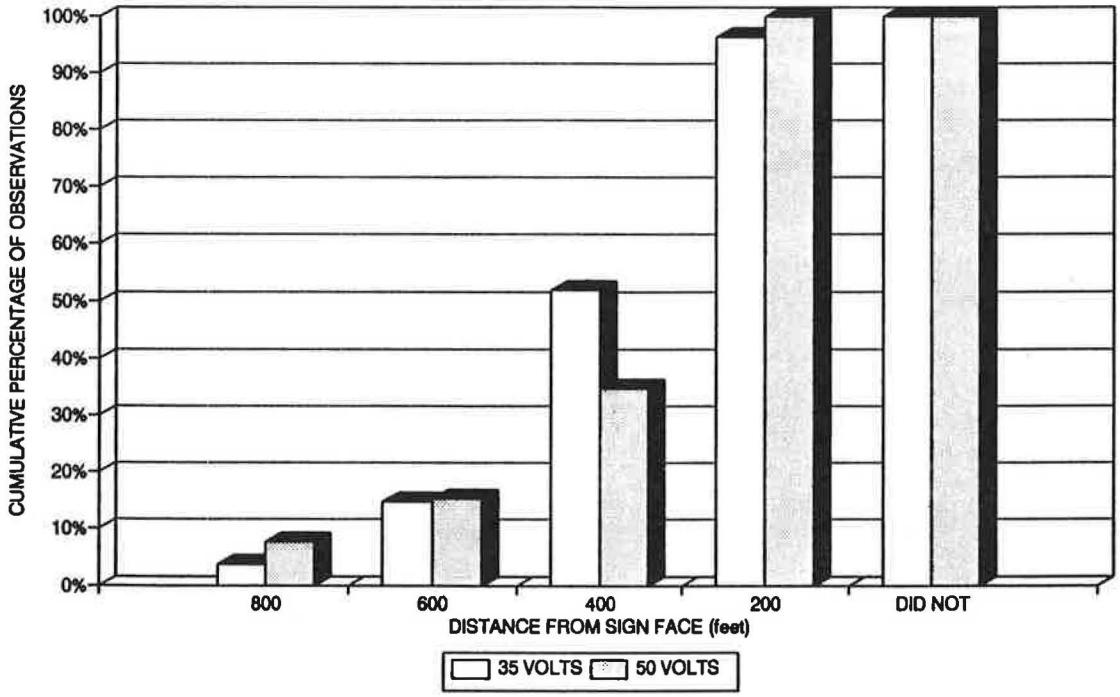

FIGURE 6 Glance legibility distance for Only word message.

findings indicate that the spacing of word messages needed to account for the concentration of light within the message, which is different from that typically used in the design of reflective displays.

The distance at which people were able to detect the difference between single-row and bold arrow shaft design was also analyzed. The ability to discern this difference was originally intended to be used in evaluating the minimum visual acuity of the participants. The ability of the participants to discern this level of detail was not consistent for either voltage. Although no single distance could be identified as the point at which this level of detail became evident, virtually all participants identified a difference in shaft design at or before 200 ft. These results indicate, therefore, that the ability to identify fine details in symbol designs is highly dependent on the capabilities of an individual's eye to deal with the light emitted by the sign.

FINDINGS AND RECOMMENDATIONS

This research has shown that fiber-optic signs are equally or more effective in conveying the messages of lane assignment at intersections when compared on the basis of target value and legibility. Further, the changeable aspects of the fiber-optic sign provide the added dimension of achieving dynamic lane assignment (i.e., altering the lane assignment display to fit the desired operational pattern at a given time).

This research indicated that a relationship exists among light output, target value, and legibility of the fiber-optic sign. Target value was found to increase with light output while legibility decreased. The selection of voltage settings for operating the fiber-optic signs should involve optimizing the relationship between these variables with respect to surrounding or ambient illumination conditions.

For the relatively dark environment of the experimental test facility, it was found that the best viewing conditions for night operation were achieved when the fiber-optic sign was operated between 35 and 65 volts based on a nominal 120-volt supply. Thirty-five volts provided the best legibility, but 65 volts provided the best target value. The principal importance of this finding is that all fiber-optic sign circuits should contain a variable voltage supply so that the voltage level can be adjusted to fit the ambient light conditions.

The placement of the traffic signals relative to the fiber-optic sign was found to be critical to the proper operation of both components. The interaction of the signals and fiber-optic sign was dependent on the operational settings for and the visual separation between each piece of equipment. These devices provide equally important but very different types of information. Operational settings and the amount of visual separation should, therefore, provide adequate target value for both the traffic signals and the fiber-optic signing without hindering the legibility of the fiber-optic sign.

The findings of the glance legibility studies indicate a strong dichotomy in the legibility of the symbols versus the words. In general, the subjects could discern the shape of the arrows at 800 ft, but the word messages remained a blur until they were in the range 200 ft from the sign. For the word messages to have a desirable effect, their legibility needs to be increased. Letter size and spacing are key factors and need to be explored further in conjunction with light output in lieu of stroke width.

These findings raise the question of whether the word messages contribute to or detract from the effectiveness of the sign in transmitting information to the driver sufficiently in advance of the intersection. Past studies have shown that signs that use symbols exclusively are more efficient in providing information to the driver than are signs that mix words and symbols or signs that use words exclusively (*6,7*). This statement is supported by the increased use of symbols in traffic signs over the last 30 years. Consequently, it is believed that the lane-use information presented by the displays would be conveyed in a more safe and effective manner through the use of arrows exclusively.

Research is needed to develop design procedures for fiber-optic displays that relate light output and visual acuity to legibility distance. The difference in legibility between the letters of the word messages examined in this research was attributed to their proximity to other letters in the word and to the complexity of their shape. This indicates that minimum visual acuity of an object is related to the total light output per unit area within the object. The procedures used in this research to identify the point of irradiation for the fiber-optic displays could be used to relate minimum visual acuity, light output per unit area, and the overall dimensions of letters and symbols. This information could be used to develop a standard letter series for use with fiber-optic signs.

The number of rows, either one or two, used to form the arrows does not have any appreciable effect on legibility. Neither does pixel spacing; however, the closer pixel spacing provides an aesthetic quality in smoothness of the symbol. From the standpoint of continuity of service, pixel spacing should be maintained closer than needed and every other pixel should connect to an alternate light source. In this manner, two lamps will be used to form an arrow or line of a symbol. When one of the lamps expires, then, the symbol is maintained even with half the pixels operational.

It is recommended that the fiber-optic lane assignment displays be evaluated under actual traffic operating conditions. A location for such a study has been selected and the signs have been designed and procured. The results of this research were used in developing the design of the new signs. Single-row pixels for the symbols, 0.70-in. spacing of pixels, and 6-in. letters with single-row pixels were specified.

The research to be performed in the field study includes integrating and coordinating the dynamic lane assignment display with the signal timing and phasing plan. Transition patterns from one lane assignment configuration to another constitutes a major concern. These various aspects of the project will be studied at the TTI sign laboratory at the Texas A&M Riverside Campus prior to installation at the study location.

Ultimately, the dynamic lane assignment concept will be integrated with the overall plan for transportation system management.

ACKNOWLEDGMENT

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Active Advance Warning Signs at High-Speed Signalized Intersections: Results of a Study in Ohio

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The effectiveness of several active advance warning signs at high-speed signalized intersections in Ohio was evaluated. The signs included the Prepare To Stop When Flashing (PTSWF), Flashing Symbolic Signal Ahead (FSSA), Continuously Flashing Symbolic Signal Ahead (CFSSA), and Passive Symbolic Signal Ahead (PSSA) signs. The research was designed as a before and after study with control sites. The measures of effectiveness included vehicular speeds at various segments of the intersection approach, vehicle conflict rates, and ratings by drivers. The study found that the effects of the signs on drivers varied among intersections with tangent and curved approaches. The PTSWF or FSSA signs generally encouraged high speed when the flasher was inactive and the signal indication was either green or yellow. Fewer motorists related the PTSWF sign to the traffic signal. In general, active advance warning signs should be discouraged at high-speed signalized intersections, particularly at intersections with tangent approach. At high-speed signalized intersections with curved approach, the CFSSA sign seems to be preferable to the PTSWF sign for reducing speed. Further study to examine the possible use of the FSSA sign in providing a better alternative at locations where the PTSWF sign cannot be effective is recommended.

A high accident potential exists at high-speed signalized intersections where an area close to an intersection, called a decision or dilemma zone, often poses a problem to a driver in stopping safely during the yellow clearance or in proceeding through the intersection before the beginning of the red interval. Traditionally, state highway departments have used active advance warning signs such as the Prepare to Stop When Flashing, Red Signal Ahead, or Signal Ahead signs to inform the driver of the presence of a signal and the fact that it is red or about to turn red. Generally, the signs are activated near the end of the green interval and remain active until the end of the red interval. No specific standards exist, however, for the design, use, and operation of active advance warning signs at high-speed signalized intersections. The lack of a standard has made many agencies increasingly concerned about possible tort liability claims arising out of any ineffective traffic control systems.

The objective of this study was to examine the effectiveness of selected active advance warning signs at high-speed signalized intersections in Ohio. The signs included the Prepare to Stop When Flashing (PTSWF), Flashing Symbolic Signal Ahead (FSSA), Continuously Flashing Symbolic Signal Ahead (CFSSA), and Passive Symbolic Signal Ahead (PSSA) signs. The intersections were located on rural or suburban highways where signals are generally unexpected or hidden by curves.

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BACKGROUND

A survey of practicing traffic engineers by the West Virginia University found that the three most commonly used types of active advance warning devices were the flashing Red Signal Ahead sign, the PTSWF sign and its variations, and flashing strobe lights (*1*). A study by the Maryland Department of Transportation found that the Red Signal Ahead sign had the potential to be an effective device in reducing right angle accidents (*2,3*). However, the study to evaluate the effectiveness of the experimental flashing red strobe sign was inconclusive. A study by the Minnesota Department of Transportation assessed the use of and experience with advance warning devices through a survey of state traffic officials (*4*). Among the 39 states responding to the survey, 29 (74 percent) reported using some form of advance warning device at high-speed signalized intersections. The most common sign was the PTSWF sign or its variations. Also fairly common among these states were blank-out messages and W3-3 Signal Ahead signs with flashers activated for signal change intervals.

A Kentucky study suggested that the use of active advance warning signs should be considered at problem locations at which a large number of avoidable accidents have occurred (*5*). A study by FHWA examined driver responses to several active advance warning signs on the highway driving simulator (HYSIM) (*6*). Eight different signs were examined at two problem locations: intersections hidden by horizontal curves and unexpected intersections at rural highways. The signs included the following:

1. PTSWF [(a) ground mounted, diamond shaped, (b) overhead, diamond shaped, (c) ground mounted, rectangular shaped, and (d) overhead, rectangular shaped],
2. Flashing Red Signal Ahead,
3. FSSA,
4. Signal Ahead, and
5. PSSA.

The study indicated that the FSSA sign was the most desirable sign. The PTSWF sign was the most incorrectly identified sign. Driver preference for the PTSWF signs was in general the lowest among the signs.

Field validation of the HYSIM results is needed. The results of the HYSIM study are applicable only to nighttime driving, however, because subject drivers on the HYSIM were only exposed to nighttime driving.

In summary, the literature review showed that field studies were needed to examine the effects of active advance warning signs at high-speed signalized intersections. This paper presents the results of an evaluation of the effectiveness of several advance warning signs at high-speed signalized intersections in Ohio.

RESEARCH APPROACH

The study was performed by collecting and analyzing field data at several high-speed signalized intersections in Ohio. The following measures of effectiveness were utilized:

1. Vehicle speeds in advance of the warning sign, in advance of the decision zone, and in advance of the stop line;
2. Vehicle acceleration or speed change rate;
3. Vehicle conflict rate; and
4. Driver survey.

These measures are further discussed in the later sections.

The research was designed as a before and after study with control sites. Several geometric and traffic characteristics, including approach alignment, number of lanes, and posted speed limit, were used to select the control and study sites. However, accident rates were not examined because of the excessively long time (at least 3 years after the installation of a sign) that must elapse before any meaningful conclusion can be made from an analysis of accident data.

The following four types of ground-mounted, diamond-shaped advance warning signs were selected for the study (Figure 1):

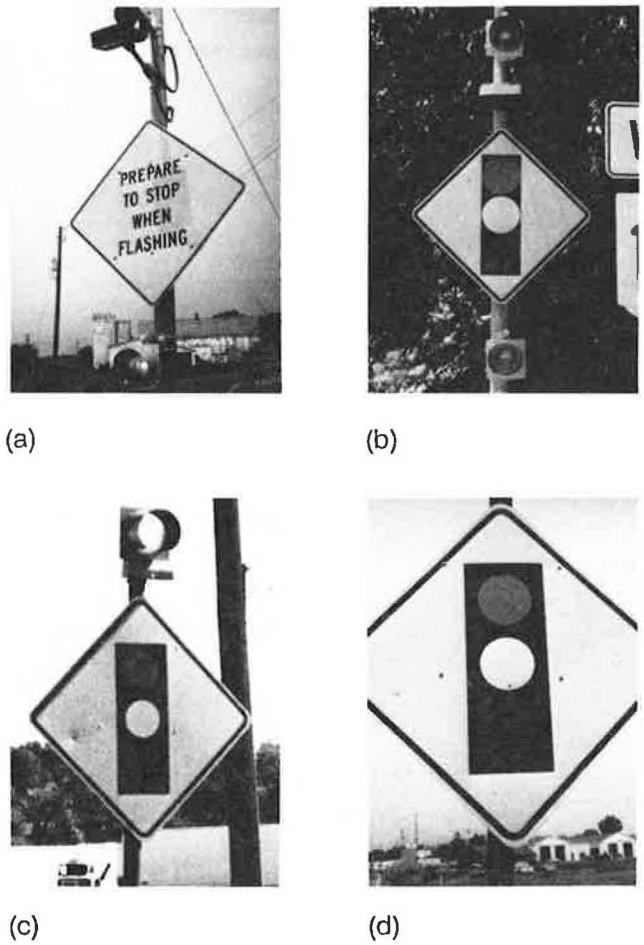


FIGURE 1 Advance warning signs: (*a*) PTSWF, (*b*) FSSA, (*c*) CFSSA, and (*d*) PSSA.

1. PTSWF sign—The PTSWF sign is the most commonly used advanced warning sign at high-speed signalized intersections in Ohio.
2. FSSA sign—The FSSA sign was selected for the study because of the general trend in the nation toward using symbolic signs for traffic control and operations. This sign was never used in Ohio before.
3. CFSSA sign—In Ohio, the CFSSA sign is often used at intersections with curved approach.
4. PSSA sign—The PSSA is only the Signal Ahead sign and no flashers are used. It is the most commonly used sign at signalized intersections.

The PTSWF and FSSA signs had yellow flashers at top and bottom that were activated near the end of the green interval and remained active until the end of the red interval. The CFSSA sign had one to two flashers that were active all the time.

The intersections were divided into two categories: intersections with tangent approach and intersections with curved approach.

To control for the effects of external factors that were not accounted for by the study, the experiments were performed at three control sites (intersections) and four study sites. The sites were so selected that the geometric conditions and posted speed limits at the control and study sites were similar. At the control sites, the advance warning signs consisted of a PSSA sign at intersections with tangent approach and a CFSSA sign at intersections with curved approach during both the before and after periods. At the study sites, the existing advance warning signs during the before period consisted of the same type of signs as those at the control sites. After the necessary data were collected during the before period, the Ohio Department of Transportation installed PTSWF or FSSA signs at the study sites. A minimum of 6 months were allowed for the motorists to become familiar with the new signs before the data for the after period were collected. The PTSWF sign was tested on two-lane highways and the FSSA sign on both two-lane and four-lane divided highways.

DATA COLLECTION

Each intersection approach was divided into three segments (called "zones" in the following discussions) as follows:

1. Zone 1 is the segment of the intersection approach just upstream of the advance warning sign.

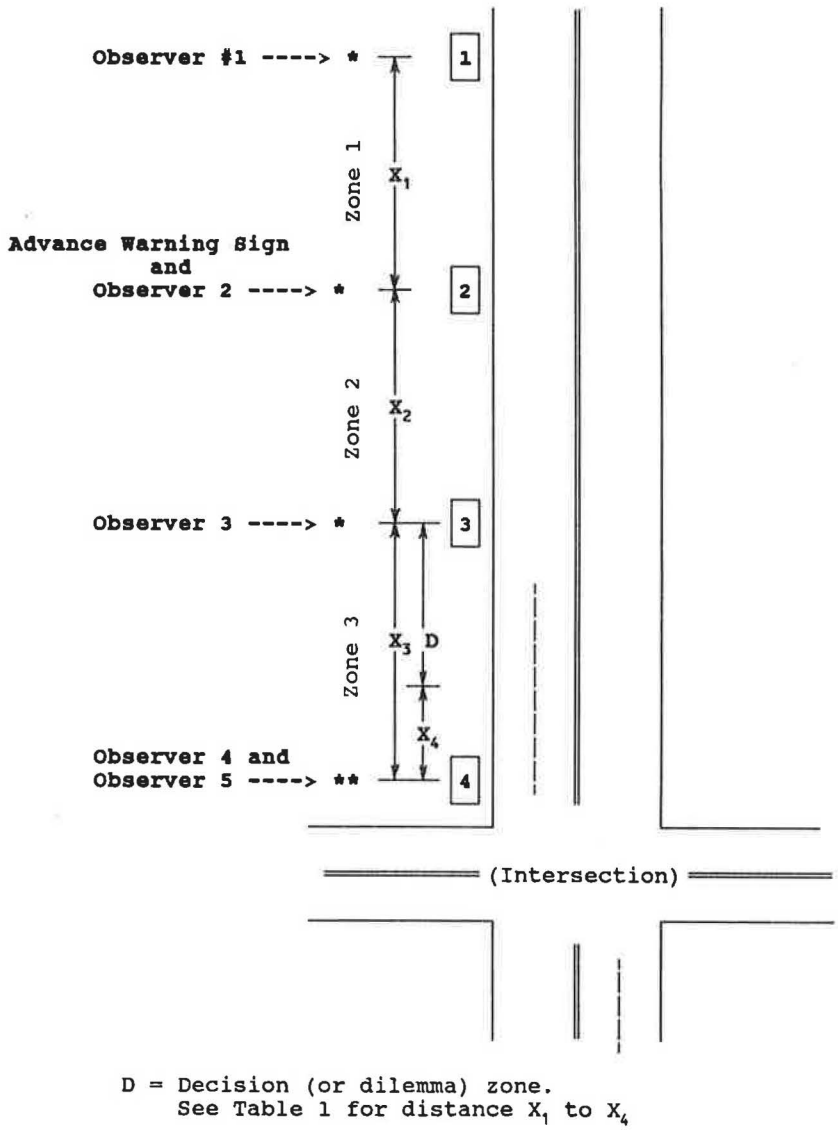


FIGURE 2 Data collection with three speed zones.

2. Zone 2 is the segment of the intersection approach just past the advance warning sign but in advance of the decision zone.
3. Zone 3 is the final segment of the intersection approach, measured from the beginning of the decision zone to the stopline.

A schematic representation of the three speed zones is shown in Figure 2. The boundaries of the decision zones (*7*) and the locations of observers for data collection are shown in Table 1.

At the study intersections where a PTSWF sign was installed during the after period, a PSSA sign existed at an upstream location [as per the Ohio Department of Transportation's (ODOT's) current practice]. Hence an additional speed zone called Zone 4 (see Figure 3) was used for collecting the data at these intersections.

A total of five or six observers manually collected the data at each intersection. The observers carried a previously syn-

TABLE 1 Location of Observers

INTERSECTION	X_1 (ft)	X_2 (ft)	X_3 (ft)	X_4 (ft)	X_5 (ft)
SR 37 at US 40	640 (640)[a]	640 (280)	384 (384)	233 (233)	NA[b] (360)
US 33 at US 127	919 (919)	919 (271)	384 (384)	233 (233)	NA (648)
SR 126 at Loveland-Madeira Rd	360	360	325	150	NA
SR 4 at Liberty-Fairfield Rd	918	918	384	233	NA
SR 36 at SR 235	615	615	384	233	NA
US 127 at SR 725	911	911	351	170	NA
US 68 at Moorfield Rd	436	436	384	233	NA

[a]*Values in parentheses are for "after" period*
[b]*Not applicable*

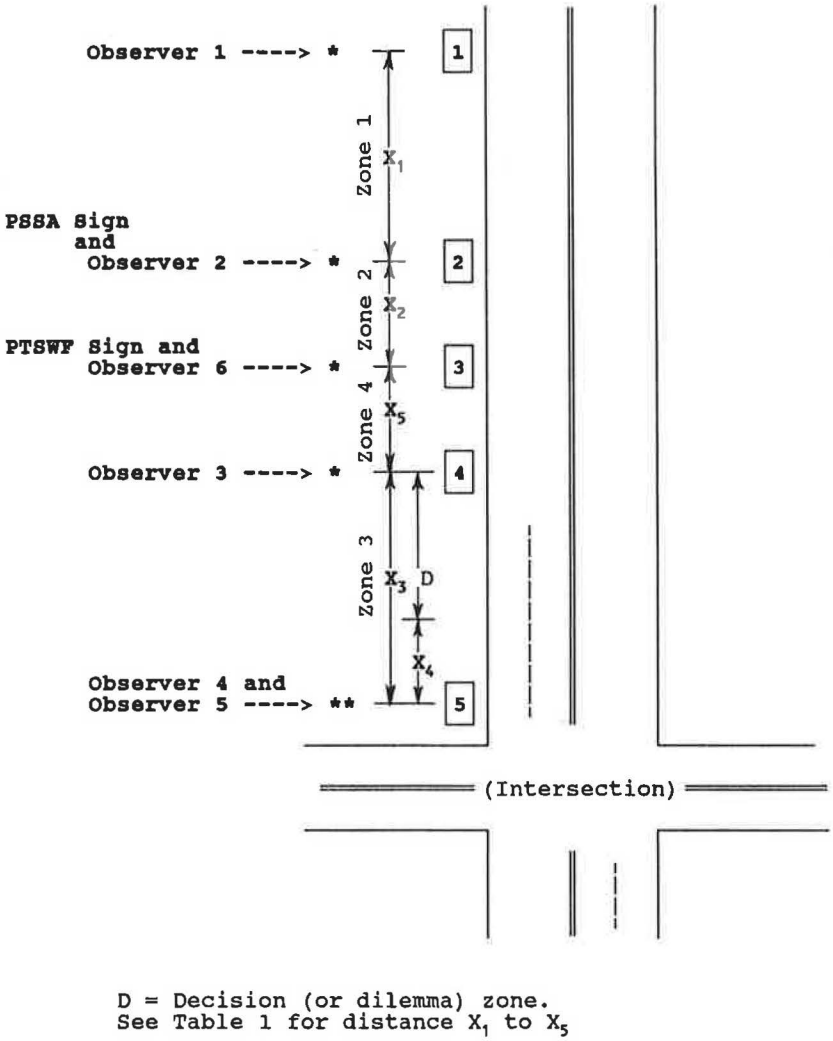


FIGURE 3 Data collection with four speed zones.

chronized electronic stop watch with an accuracy of 1/100 sec. and a walkie-talkie for communication with each other. A previous study has shown that speed measurement techniques using stop watches are capable of yielding individual speed measurements to an accuracy of 1 mph (*9*). Observer 1 randomly selected the vehicles (Figures 2 and 3). If vehicles arrived as a platoon, the first vehicle was sampled. Upon receiving command from Observer 1, the remaining observers located at the respective positions closely recorded the vehicle's movement until it crossed the intersection. The following information for each vehicle was recorded:

1. Time of arrival of the vehicle at Positions 1 to 5;
2. Vehicle type—Light vehicle (passenger car, van, and pickup) or heavy vehicle (truck, bus, and recreational vehicle);
3. Flasher Indication 1—If the approach had a sign with a flasher, whether the flasher was active when the vehicle arrived at the CFSSA or FSSA sign, or at the PSSA sign if the approach had a PTSWF sign;
4. Flasher Indication 2—Whether the flasher was active when the vehicle arrived at the PTSWF sign;
5. Signal Indication 1—Traffic signal indication when the vehicle entered into the decision zone;
6. Signal Indication 2—Traffic signal indication when the vehicle reached the stopline;
7. Stop—Whether the vehicle proceeded through the intersection or stopped;
8. Conflict—The following type of conflicts (*8*) were recorded:
   a. Run red light—A proceeding vehicle was upstream of the stopline when the signal turned red;
   b. Abrupt stop—A driver decided at the last instant to stop; the deceleration, particularly within 100 ft of the stopline, caused the front end of the vehicle to dip noticeably;
   c. Acceleration through yellow—The driver "guns" the engine to clear the intersection; and
9. Direction—Through, left turn, or right turn.

The speed sampling periods were 7:00–9:00 a.m., 10:00–11:30 a.m., 1:00–2:30 p.m., 3:00–6:00 p.m., 9:00 p.m.–midnight.

Driver Survey

A questionnaire was prepared to obtain drivers' subjective responses to the advance warning signs. Several techniques, including personal interview, mailing, and distribution of the questionnaire to employees of nearby business facilities, were employed. The information obtained from the respondents included the following:

1. Driver characteristics—age, sex, education, driving experience, familiarity with site;
2. What the sign meant to the respondent;
3. What action the respondent took when he/she saw the sign;
4. Respondent's action (if any) when the traffic signal turned yellow;
5. Respondent's ratings of the sign on a scale of 0 to 10 based on adequacy of information, of time available to read and understand message, of ease with which message is read and understood, and overall effectiveness.

The average sample size per location was about 50.

Traffic Volume

Traffic volume is not a measure of effectiveness. It is related, however, to vehicle conflict rate at the intersection. The traffic volume at the intersection, categorized by light and heavy vehicles, was manually recorded.

DATA ANALYSIS AND RESULTS

The data were used to calculate the mean vehicular speed in each zone. In general, when vehicles traveled from Zone 1 to Zone 3 in an intersection approach, the mean speed was gradually reduced until the vehicles crossed the intersection. The magnitude of the speed reduction seemed to be related to the geometric condition, type of advance warning sign, flasher indication (if applicable), and signal indication. Acceleration and deceleration rates between adjacent zones were calculated and compared with standard rates (*10*) that motorists are expected to conform to when they are not required to react rapidly. An acceleration or deceleration rate in excess of the standard rate may indicate a potential problem for the motorists at the intersection. The analysis showed that none of the acceleration or deceleration rates observed during the before and after periods exceeded the standard rate.

To test the null hypothesis that there was no difference in speed change rate during the before and after periods, *t*-tests were performed at 0.05 level of significance. The alternate hypothesis stated that there existed a difference in speed change rate. For a meaningful test, the effects caused by the difference in original speed should be eliminated. For example, if the speed of a vehicle changed from 60 mph to 30 mph, and the speed of another vehicle changed from 30 to 15 mph over the same distance, the net difference is 30 mph for the former and 15 mph for the latter. But the speed change rate is the same for both vehicles, because speed change rate = (60 − 30)/60 = .05 or (30 − 15)/30 = 0.5. The effect of the difference in original speed was eliminated by performing *t*-tests on speeds in which the logarithm of speed, instead of absolute speed, was employed. Thus, speed difference = log(speed 1) − log(speed 2). Hence for the two vehicles in the above example, log(speed 1) − log(speed 2) = log(60/30) or log (30/15) = log(2). The results of the *t*-test for which the speed change rates were found significant are discussed in the respective sections below.

A sample of the before and after speed data for the intersection on US-33 at US-127 is presented in Tables 2 and 3. The mean speeds of passenger cars for different signal con-

TABLE 2 Mean Speed on US-33 at US-127 During "Before" Period

Speed Zone	Signal1[a]	Signal2[b]	Proceed[c]	Speed (MPH)	Acceleration (ft/s/s)
1	Green	Green	Yes	54.9	-0.54
2	Green	Green	Yes	50.5	-1.85
3	Green	Green	Yes	37.8	
1	Green	Yellow	Yes	61.8	-1.21
2	Green	Yellow	Yes	52.8	-2.80
3	Green	Yellow	Yes	33.0	
1	Green	Red	No	57.4	-1.67
2	Green	Red	No	43.2	-2.50
3	Green	Red	No	18.7	-1.96
1	Yellow	Red	No	58.5	-0.91
2	Yellow	Red	No	51.4	-3.54
3	Yellow	Red	No	22.3	-2.79
1	Red	Green	Yes	58.1	-1.19
2	Red	Green	Yes	48.6	-2.83
3	Red	Green	Yes	25.4	
1	Red	Red	No	58.4	-1.00
2	Red	Red	No	50.6	-3.41
3	Red	Red	No	22.2	-2.76

[a]*Signal indication when a vehicle arrives at the "decision zone"*
[b]*Signal indication when the vehicle arrives at the stop line*
[c]*Does vehicle proceed through the intersection without stopping?*

TABLE 3 Mean Speed on US-33 at US-127 During "After" Period

Speed Zone	Flasher 1[a]	Flasher 2[b]	Signal 1[c]	Signal 2[d]	Proceed Intersection[e]	Speed (MPH)	Acceleration (ft/s/s)
1	Inact.	Inact.	Green	Green	Yes	60.9	-0.89
2	Inact.	Inact.	Green	Green	Yes	55.3	-2.29
3	Inact.	Inact.	Green	Green	Yes	45.6	0.18
4	Inact.	Inact.	Green	Green	Yes	46.2	
1	Inact.	Inact.	Green	Yellow	Yes	58.1	-0.65
2	Inact.	Inact.	Green	Yellow	Yes	53.9	-2.96
3	Inact.	Inact.	Green	Yellow	Yes	40.5	1.53
4	Inact.	Inact.	Green	Yellow	Yes	45.9	
1	Inact.	Active	Red	Red	No	55.6	-0.11
2	Inact.	Active	Red	Red	No	54.9	-2.69
3	Inact.	Active	Red	Red	No	43.2	-4.35
4	Inact.	Active	Red	Red	No	23.3	-3.04
1	Active	Inact.	Green	Green	Yes	57.2	-0.75
2	Active	Inact.	Green	Green	Yes	52.2	-2.21
3	Active	Inact.	Green	Green	Yes	42.2	-0.88
4	Active	Inact.	Green	Green	Yes	38.9	
1	Active	Active	Red	Red	No	59.0	-0.81
2	Active	Active	Red	Red	No	53.8	-2.47
3	Active	Active	Red	Red	No	42.9	-4.14
4	Active	Active	Red	Red	No	24.1	-3.25

[a]*Signal indication when the vehicle arrives at the "decision zone".*
[b]*Signal indication when the vehicle arrives at the stop line.*
[c]*Flasher indication when the vehicle arrives at the PSSA sign.*
[d]*Flasher indication when the vehicle arrives at the PTSWF sign.*
[e]*Does vehicle proceed through the intersection without stopping?*

ditions were plotted graphically. Samples of the graphs are presented in Figures 4 and 5. Data for other sites could not be included in this paper for space reasons.

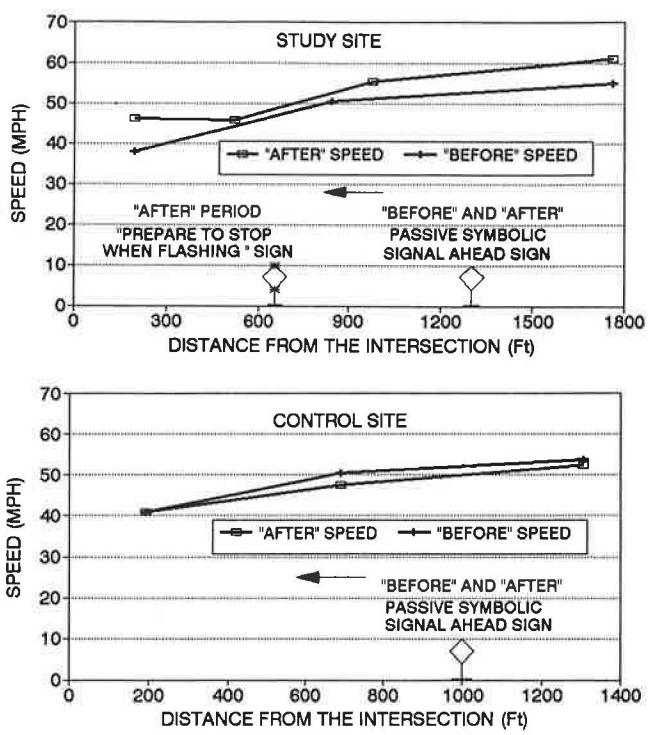
EFFECTS OF PTSWF SIGN

The effects of the PTSWF sign seemed to vary among intersections with tangent and curved approaches.

PTSWF Sign at Tangent Approach

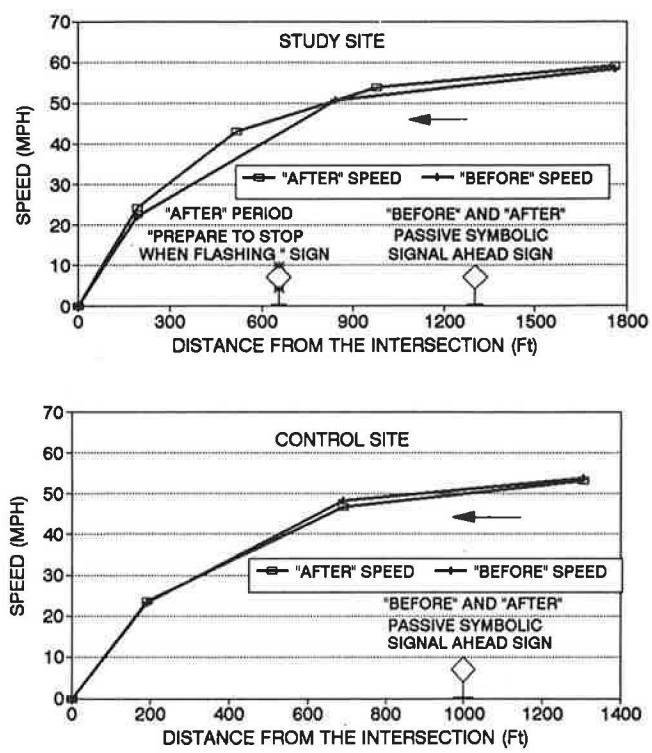
A PSSA sign existed at the intersection approach during both periods. In addition, during the after period, a PTSWF sign existed at a downstream location on the intersection approach. As shown in Figure 3, four zones were used for calculating speed during the after period, as compared to three zones during the before period. The result showed that mean speeds were particularly influenced by the condition of the flasher on the PTSWF sign when vehicles arrived at the PSSA and PTSWF signs. When the flasher was inactive during the time of vehicle arrivals at the PSSA and PTSWF signs and the signal was green when vehicles arrived at the stopline, the mean speeds in Zone 1 and Zone 3 increased by 6 and 8 mph, respectively, indicating a substantial increase in mean speeds on the intersection approach during the after period. The *t*-test showed that the speed change rate in Zone 3 was significant at the 5 percent level of significance. In contrast, the mean speeds changed little at the control site during the same period. Contrary to the normal pattern typically observed at signalized intersections, mean speed in Zone 3 was higher than in Zone 2 during the green interval, indicating that motorists were speeding up when they arrived in the decision zone. This is not a desirable trend, especially if the speed increase is great, because it is likely to create a difficult situation for motorists taking any corrective or evasive action that becomes necessary. This trend was not observed during the before period when only the PSSA sign existed at the intersection approach.

Additionally, when the flasher was inactive during the time of vehicle arrivals at the PSSA and PTSWF signs, an undesirable trend was observed among vehicles proceeding through the intersection during the yellow interval. For example, the mean speed in Zone 1 was observed to be 58 mph, which decreased to 41 mph in the next zone but increased to 46 mph in Zone 3. During the *t*-test, the speed change rate was found to be significant at a 5 percent level of significance. The speed increase in Zone 3 is contrary to normal expectations, indicating that motorists were speeding up in the yellow interval during the after period. Again, this type of trend was not observed at the intersection during the before period.



(1) Signal is green when vehicle arrives at the decision zone
(2) Signal is green when vehicle arrives at the stop line
(3) Vehicle proceeds through intersection

FIGURE 4 Mean speed on study site and control site (condition: green-green-proceed).



(1) Signal is red when vehicle arrives at the decision zone
(2) Signal is red when vehicle arrives at the stop line
(3) Vehicle stops at the stop line

FIGURE 5 Mean speed on study site and control site (condition: red-red-stop).

The speed pattern was different when the flasher was active during the time of vehicle arrivals at the PSSA sign and inactive at the PTSWF sign. There was no appreciable increase in the mean speeds before the motorists crossed the intersection. It seemed that motorists understood the information received from the flasher and proceeded through the intersection without increasing speed.

The study site had a conflict rate of 82 conflicts per 1,000 vehicles during the before period that was reduced to 31 conflicts per 1,000 vehicles (or by 62 percent) after the installation and operation of the PTSWF sign (Table 4). However, the control site that had 27 conflicts per 1,000 vehicles during the before period also experienced a 62 percent decrease (to 10 conflicts per 1,000 vehicles) during the after period. Hence, the conflict reduction at the study site cannot be attributed to the PTSWF sign.

When motorists were surveyed about the PSSA sign during the before period, 90 percent of the respondents indicated that the sign means, "There is a signal ahead" (Table 5). During the after period, however, only 60 percent of the respondents related the PTSWF sign with the signal. This result supports the findings of the FHWA study on the HYSIM in which drivers commented that the PTSWF sign had a limited and inadequate relationship to a traffic signal. However at this site, an overwhelming proportion of the respondents had favorable comments about the PTSWF sign because many area residents were concerned with the relatively high percentage of trucks on the highway. In response to the statement, "Overall, the sign was helpful to me in terms of my driving through or stopping at the intersection," on a scale of 0 to 10 the rating increased from 7.6 for the PSSA sign to 8.6 for the PTSWF sign.

PTSWF Sign at Curved Approach

During the before period, a CFSSA sign existed at the intersection approach. During the after period, the CFSSA sign was converted to a PSSA sign, and a PTSWF sign was installed at a downstream location at the intersection approach as per ODOT's current practice. When the flasher was inactive during the time of vehicle arrivals at the PSSA and PTSWF signs, and the signal was green when vehicles reached the stopline, no appreciable difference in mean speeds between the two periods was found. The *t*-test showed that the speed change rate was not significant at the 5 percent level of significance. This result is in sharp contrast with the previously described intersection with tangent approach where the mean speeds were found to increase by 6 to 8 mph. It indicated that the effects of the PTSWF sign vary between intersections with tangent and curved approaches.

An important change in speed pattern, similar to the pattern observed at the tangent approach, was observed when vehicles proceeded through the intersection during the yellow interval. The speed in Zone 3 increased by 7 mph for vehicles that had reached the PSSA and PTSWF signs when the flasher was inactive. The *t*-test showed that the speed change rate was significant at the 5 percent level of significance. It seemed that when the light turned yellow, motorists were either unable or unwilling to reduce speed or stop at the intersection. The increase in speed may not be desirable, especially at an intersection approach with curvature. If reducing speed at the intersection during yellow interval is an objective of the advance warning sign, the PTSWF sign is less effective than the CFSSA sign.

The analysis showed that the vehicle conflict rate had increased by 15 percent during the after period. Considering that the conflict rate at the control site had declined by 36 percent during the same period, the real increase in conflict rate at the study site is larger than the 15 percent rate observed at the intersection.

The result of the driver survey showed that the PTSWF sign received a rating of 8.1–8.7 on a scale of 0 to 10, in contrast to the ratings of 6.8–7.2 for the CFSSA sign, indicating that motorists generally preferred the PTSWF sign to the CFSSA sign. However, in response to the question, "What does the sign mean to you?," fewer motorists seemed to relate the PTSWF sign to the traffic signal, because the percentage of motorists who thought the PTSWF sign meant there was a traffic signal ahead dropped from 76 percent during the before period to 60 percent during the after period. The number of motorists who indicated they slow down when they see the sign increased from 56 percent during the before period to 70 percent during the after period.

TABLE 4 Traffic Volume and Vehicle Conflict Rates[a]

INTERSECTION	TOTAL TRAFFIC[b] (VEH)	RUN RED	SPEED UP ON YELLOW	ABRUPT STOP	TOTAL CONFLICT
SR 36 "BEFORE"	868	2.3	23.0	1.5	26.8
SR 36 "AFTER"	905	1.1	9.1	0.0	10.2
CHANGE[c]	-4.3%	2.2%	60.4%	100%	61.9%
SR 33 "BEFORE"	719	8.5	66.6	7.0	82.1
SR 33 "AFTER"	594	1.7	28.0	1.8	31.5
CHANGE	17.4 %	80.0%	58.0%	74.3%	61.6%

[a]*Vehicle conflicts are expressed per 1000 vehicles*
[b]*Traffic volume during 11 hour period*
[c]*Change = (Before's - After's)/Before's*

EFFECTS OF FSSA SIGN

As with the PTSWF sign, the effects of the FSSA sign seemed to vary among intersections with tangent and curved approaches.

FSSA Sign at Curved Approach on Two-Lane Highway

During the before period, two CFSSA signs were at the intersection approach (one on each side of the roadway) that were replaced by an FSSA sign during the after period. The result showed that the mean speed in Zone 3 remained unchanged during the after period, indicating no difference in the effects of the two signs.

TABLE 5 Driver Survey

US 33 AT US 127	Average (before)	Average (after)	Comparison[a]
HOW OFTEN DO YOU DRIVE ON THE HIGHWAY ?			
About once a day	13 %	20 %	-58 %
More than once a day	58 %	36 %	38 %
About once or twice a week	25 %	33 %	-33 %
About once or twice a month	4 %	10 %	-159 %
WHAT DOES THE SIGN MEAN TO YOU ?			
There is a signal ahead	90 %	60 %	32 %
Slow down	46 %	52 %	13 %
Be ready to stop if necessary	67 %	84 %	25 %
Does not mean any thing	0 %	0 %	0 %
Others	4 %	2 %	50 %
WHAT ACTION DO YOU TAKE WHEN YOU SEE THE SIGN ?			
Slow down	73 %	80 %	10 %
Become alert	75 %	63 %	16 %
Do nothing until I see the traffic light	4 %	2 %	50 %
Others	4 %	2 %	50 %
DO YOU TAKE ANY ACTION WHEN LIGHT TURNS YELLOW ?			
Try to stop	50 %	50 %	0 %
Speed up to enter intersection before red	4 %	2 %	50 %
Depends on how close I am to the intersection	54 %	58 %	- 7 %
Others	8 %	6 %	25 %
RATINGS[b]			
Sign Adequately Alerts Me to Stop or Drive Through	7.46 (3.97)[c]	8.63 (1.92)	-16 %
Time Adequate to Read or Understand Message	7.85 (3.36)	8.78 (1.21)	-12 %
The Ease with Which Message Is Read	7.75 (4.35)	8.74 (1.23)	-13 %
Overall, The Sign Was Helpful	7.66 (5.05)	8.61 (2.32)	-11 %
AGE 1=(<20),2=(20-29),3=(30-39),4=(40-49) 5=(50-59),6=(60-69),7=(70 and above)	4.8	4.9	-3 %
SEX Male	81 %	78 %	4 %
Female	19 %	22 %	-16 %
DRIVING EXPERIENCE 1=(<1), 2=(1-2), 3=(3-5), 4=(6-10), 5=(>10)	4.8	5.0	- 3 %
MILES DRIVEN/YEAR 1=(<5000), 2=(5000-10000), 3=(>10000)	2.6	2.5	5 %
LAST SCHOOL GRADE ATTENDED (1-17)	12.5	12.5	0 %
VISION			
Not wear glasses or other lenses	37 %	39 %	-5 %
Wear lasses	23 %	28 %	-22 %
Wear bifocals	15 %	35 %	133 %
Wear contact lenses	4 %	11 %	175 %
Sample Size	49	46	6 %

[a]*Comparison = (before's - after's)/(before's) x 100%*
[b]*Rating on 0-10 scale*
[c]*Number in parentheses represents standard deviation*

When the signal indication was red and vehicles stopped at the intersection, mean speeds in Zone 3 decreased by 4 mph during the after period. The *t*-test found the speed change rate to be significant at the 5 percent level of significance. Drivers seemed to understand that, when the FSSA sign was active, the signal was likely to remain red when they reached the intersection. On the other hand, the CFSSA sign provided no advance information to the drivers about possible signal indication upon arrival at the intersection.

The results showed that the vehicle conflict rate had declined by 8 percent (from 31 conflicts per 1,000 vehicles during the before period to 29 conflicts during the after period). But the control site had experienced a 36 percent reduction in vehicle conflict (from 42 conflicts per 1,000 vehicles to 26 conflicts) during the same period. Hence the FSSA sign did not seem to be effective in reducing vehicle conflict.

The result of the driver survey showed that few differences in the ratings for the CFSSA and FSSA signs. In response to some specific questions during the after period, 46 percent more respondents seemed to understand that the sign meant, "There is a signal ahead," 48 percent more indicated it meant, "Slow down," and 80 percent more indicated they would "become alert" when they saw the sign. Overall, drivers' responses to the FSSA sign were positive.

FSSA Sign at Tangent Approach on Four-Lane Divided Highway

During the before period, two PSSA signs were posted, one on each side of the roadway. For the after period, these signs were removed and replaced by two FSSA signs at a downstream location on the intersection approach. In general, when the flasher was inactive and the signal indication was green, the mean speeds in the three zones were generally 2 to 4 mph higher during the after period. When the flasher was active, and the signal was red, the mean speed of vehicles was 4 to 7 mph higher at the upstream zone, which narrowed down to 0.5 to 2 mph in Zone 3. The result showed that motorists reduced speed only after they passed the FSSA sign, indicating that the location of the FSSA sign had an important impact on speed before motorists stopped at the intersection.

The analysis showed little difference in vehicle conflict rates during the two periods. The result of the driver survey showed that the overall rating of the FSSA sign was slightly higher than that of the PSSA sign. Two important changes in drivers' responses were noted during the after period. First, 85 percent of the respondents indicated that the FSSA sign meant "be ready to stop if necessary," which is in sharp contrast to the 21 percent of respondents who had the same opinion about the PSSA sign. Second, respondents who indicated that they slowed down when they saw the sign increased from 23 percent to 50 percent. However, for unexplained reasons, 20 percent fewer respondents indicated the sign means, "There is a signal ahead."

CONCLUSIONS AND RECOMMENDATIONS

The results of this study should be used with caution because the study was conducted on a limited number of sites. The conclusions and recommendations are as follows:

1. An increase of speed at an intersection approach, particularly in the decision zone or its vicinity at a high-speed signalized intersection, is not desirable for safe movements of vehicles through the intersection. Hence, in general, the use of active advance warning signs such as the PTSWF or FSSA signs should be discouraged. The signs were found to encourage high speed under some flasher and signal conditions, particularly when the flasher was inactive and the signal indication was either green or yellow. The signs should be particularly discouraged at high-speed signalized intersections with a tangent approach.
2. At high-speed signalized intersections with a curved approach, the CFSSA sign seems to be preferable to the PTSWF sign for reducing speed. The PTSWF sign at a curved approach seemed to encourage high speed during the yellow interval and also increased the vehicle conflict rate. Fewer motorists related the PTSWF sign to the traffic signal at high-speed signalized intersections.
3. A study to further investigate the effectiveness of the FSSA sign at high-speed signalized intersections is recommended. Although the FSSA sign did not seem to be effective in reducing the vehicle conflict rate at curved approach, it seemed to assist motorists to prepare for stopping at the intersection when the flasher was active and the signal was red. The possibility that the FSSA sign provides a better alternative at locations where the PTSWF sign cannot be effective should be examined.
4. Future study should examine the effects of the active advance warning signs on frequency and severity of accidents at high-speed signalized intersections. Finally, the current practice of locating and timing an active warning sign at high-speed signalized intersections should be reviewed.

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Issues in Flashing Operation for Malfunctioning Traffic Signals

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Current standards and practices have been evaluated for the display to be shown to drivers when control equipment at a signalized intersection malfunctions. Documents setting forth current good engineering practice were reviewed; signalized intersections were inspected in metropolitan Atlanta; and eight traffic engineering agencies were interviewed. The *Manual on Uniform Traffic Control Devices* (MUTCD) describes flashing yellow/red as "normal"; however, two other primary references caution that this may be inappropriate and hazardous if the minor street lacks sight distance or traffic volume on the major street is moderate or heavy. Ten intersections were readily found that failed to meet AASHTO standards for sight distance but were programmed for flashing yellow/red. The agencies stated that flashing red/red during rush hours would produce intolerable congestion during the estimated 10 min needed for a traffic-control police officer to reach the scene. It was recommended that serious consideration be given to changing the MUTCD to acknowledge the potential hazard of flashing yellow/red and to make mention of flashing red/red. Traffic engineering agencies should evaluate all signalized intersections that use flashing yellow/red and identify as "critical" those that would operate with difficulty during rush hours or that have inadequate sight distance for a minor-street approach. Remote monitoring of signal status, selective use of Right Turn Only blank-out signs, and a public information program were recommended. Research is needed on accident rates during malfunction flash.

This research was prompted by the senior author's being consulted about a serious accident that occurred soon after a tripped conflict monitor placed a signalized intersection on flashing yellow for the main street and flashing red for the cross street, just as the *Manual on Uniform Traffic Control Devices* (MUTCD) suggests is normal. A visit to the accident scene made it clear that accidents were quite foreseeable at this location if the intersection were allowed to operate in this flashing mode. If main-street volumes are too heavy for side-street traffic to enter or cross, or if side-street traffic cannot see far enough along the main street for safety, then how should the intersection be operated when a signal malfunction occurs?

Flashing operation at a signalized intersection can be preprogrammed for selected off-peak time periods when traffic is considered too light to require normal green/yellow/red (stop-and-go) operation. Flashing operation will also occur if a stop-and-go signal malfunctions in certain ways, such as by the simultaneous display of green to two conflicting movements. Issues pertaining to off-peak flash are already discussed extensively in the literature (*1*). Therefore, the focus here is on malfunction flash. (However, the section related to sight distance is also pertinent to off-peak flash.)

Agency records reviewed by the authors show that malfunction flash occurs frequently enough to be considered an expected and foreseeable event. Like freeway incidents, malfunction flash is not uncommon but is rather unpredictable as to location. Some causes of malfunction flash are controller failure, load switch failure, and voltage problems such as transients, sags, and brief power outages.

Over the past 20 years of experience with malfunction flash improvements have been made in the electrical design of signal control equipment that have tended to reduce the incidence of malfunction flash. At the same time, however, the industry has expanded the list of faults that can be watched for by the conflict monitor in the cabinet. In the early days of solid-state equipment, the conflict monitor unit was aptly named because it would cause the intersection to go to flash only in the event of green (or yellow) shown simultaneously to two conflicting movements. In 1976 the National Electrical Manufacturers Association (NEMA) expanded the concept of "conflict" to include absence of signal ("dark failure" or "red failure") for a movement (*2*). Currently, at least one manufacturer offers a conflict monitor that can (as an option) cause flashing operation if a load switch fails by turning on two or three colors at once. For example, the intersection will go to flash if one movement is shown a red and a conflicting movement is shown both a green and a yellow. Currently, NEMA is considering requiring a similar load switch monitoring capability (for optional use) for their future design standard TS 2 (*3*). Fault monitoring has expanded over the years to the point that NEMA's draft TS 2 now refers to a conflict monitor as a "malfunction management unit." Plainly these safety-oriented trends in the signal industry could keep malfunction flash at least as frequent an occurrence as it is today.

RESEARCH PROCEDURE

The purpose of this research was to evaluate current standards and practices for the display to be shown to drivers when the control equipment at a signalized intersection malfunctions. The procedure was first to review the documents setting forth current engineering good practice. This step showed that two primary references caution that flashing yellow/red could be inappropriate and hazardous if minor-street drivers waiting on the flashing red cannot see far enough along the main street or are confronted with moderate to heavy volumes on the main street. The latter condition is well known to be

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common during rush hours. The researchers then went to a number of signalized intersections in the Atlanta metropolitan area to determine if sight distance might be a potential problem during flashing yellow/red. Through field observation, 10 intersections lacking adequate sight distance were easily found. A check with the local traffic engineering agencies showed that each intersection was programmed to flash yellow/red. The researchers measured the sight distance available to minor-street drivers who could find themselves waiting on a flashing red. Finally, interviews were conducted with eight traffic engineering agencies to determine opinions and practices with respect to malfunction flash. Conclusions were drawn as to the potential difficulty of ensuring both safety and uncongested flow during periods of malfunction flash. Recommendations for improving safety during malfunction flash and for needed research were developed.

CURRENT ENGINEERING GOOD PRACTICE

The MUTCD, which is published by FHWA, has been adopted by most states as a standard. Section 4B-6(7) states the following:

> When a traffic control signal is put on flashing operation, normally a yellow indication should be used for the major street and a red indication for the other approaches. . . (*4*, p. 4B-6)

FHWA publishes a companion manual, the *Traffic Control Devices Handbook* (TCDH) (*5*), which is "intended to augment the MUTCD by serving an interpretive function. . ." The TCDH states the following:

> Flashing yellow/red may be appropriate at simple, four-legged or three-legged intersections where the minor-street drivers have an unrestricted view of approaching main street traffic, and the traffic volumes are low. (*5*, p. 4-9)

The TCDH in effect cautions that the normal flashing yellow/red operation set forth by the MUTCD could be inappropriate when minor-street drivers cannot see far enough along the main street or cannot find a gap in main-street traffic long enough to avoid collisions.

AASHTO publishes a primary reference titled *A Policy on Geometric Design of Highways and Streets* (*6*). The book is well known to traffic engineers as well as to highway designers and is commonly called the Green Book. This reference includes sight-distance guidelines for intersections under various types (cases) of traffic control. Case III pertains to Stop sign control of traffic on a minor road waiting to enter or cross a major roadway:

> Where traffic on the minor road of an intersection is controlled by stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position. . . (*6*)

The Green Book adds the following:

> This principle applies to signalized intersections as well as those with only stop signs, as there may be a malfunction of the signals, or it may be desirable to place the signals periodically in a flashing operation. The latter is the same as stop sign control. (*6*)

The Green Book also states the following:

> Intersections controlled by traffic signals are considered by many not to require sight distance between intersecting traffic flows because the flows move at separate times. However, due to a variety of operational characteristics associated with intersections, sight distance based on the Case III procedures should be available to the driver. This principle is based on the increased driver workload at intersections and the hazard involved when vehicles turn onto or cross the major highway. The hazard associated with unanticipated vehicle conflicts at signalized intersections, such as violation of the signal, right turns on red, malfunction of the signal, or use of flashing red/yellow mode, further substantiate the need for incorporation of Case III sight distance even at signal-controlled intersections. (*6*)

The Green Book is seen to be very specific about the potential hazards of flashing yellow/red, such as during signal malfunction, where entering sight distance falls short of AASHTO guidelines for Case III intersections.

This review of current engineering good practice showed first that the MUTCD simply states that it is normal for a flashing display to be yellow/red. The TCDH and the AASHTO Green Book caution that flashing yellow/red may be inappropriate and hazardous if sight distance is a problem or traffic volume on the main street is moderate or heavy.

FIELD MEASUREMENT OF ENTERING SIGHT DISTANCE

The next step was to apply the AASHTO Case III sight-distance criteria to a number of signalized intersections in the Atlanta metropolitan area. The purpose was to gather information on specific locations with sight-distance problems.

Field observations quickly turned up 10 intersections that were programmed to flash yellow/red upon malfunction and that appeared possibly to be lacking in sight distance from the minor-street stopped positions (*7*). Posted speed limits on the major street varied from 35 to 45 mph. The characteristics of each intersection are detailed by Walker (*7*).

AASHTO sight-distance criteria assume that the minor-street driver will stop with his front bumper 10 ft back from the near edge of the intersecting major-street pavement. However, the senior author has found that, as a practical matter, a court may impose a higher standard for that driver. If edging the car forward will improve the driver's sight distance, a jury may expect the driver to edge out to the extent that it is safe to do so. Therefore, all sight-distance measurements in this research were performed with the front bumper assumed to be located as close as possible to the edge of the major-street pavement. This procedure improved the sight distance of all 10 intersections measured, thus improving their prospects of meeting AASHTO Case III standards. The waiting driver was taken to be 7 ft back from the front bumper as if stopped in a full-sized vehicle.

In the AASHTO Case III procedures, the driver's eye is 3.5 ft above the minor-street pavement, and he/she is looking for an object 4.25 ft above the major-street pavement (*6*). In this research the same height of object was used, but the waiting driver was taken to have a height of eye of 4.0 ft; the extra 0.5 ft of height results in a sight distance slightly longer than AASHTO standard.

Most of the 10 intersections were four-way, allowing checks of all three AASHTO Case III maneuvers, namely crossing (III A), turning left (III B), and turning right (III C). Four of the intersections had possible sight-distance problems in both directions of view from the minor street. In all, for the 10 intersections there were 35 sight-distance checks.

It was found that each intersection failed all of the Case III tests that applied to it, even with the waiting driver taken to be 10 ft closer to the intersection than called for by AASHTO guidelines for Case III measurements. All of the sight-distance restrictions were caused by horizontal or vertical curvature, or buildings, or a combination of these. Deficiencies in sight distance varied from 75 to 630 ft, averaging 312 ft. Of the 35 cases of tested, almost half failed by more than the length of a football field.

AASHTO Case III sight distances are intended to give a minor-street driver enough time to complete the departure maneuver before a vehicle approaching the intersection from just out of view is close enough to create a conflict. The deficiencies in measured sight distances were converted to seconds of travel time along the major street at the posted speed limit. These varied from 1.1 to 9.5 sec, averaging 5.2 sec.

INTERVIEWS WITH TRAFFIC ENGINEERING AGENCIES

Comprehensive interviews were conducted with eight traffic engineering agencies in the Atlanta metropolitan area (*7*). Each engineer was asked the same series of questions, and extensive notes were taken by the interviewer. The eight agencies were about equally divided among categories of large, medium, and small number of signalized intersections operated. The large and medium-sized agencies were able to monitor in real time the flash status of about 25 percent of their signals, on the average, and the two small agencies could continuously monitor about half their signals.

Off-Peak Flash

All eight engineers reported that they do not use off-peak flash. However, each had extensive opinions on the subject. These were reported in detail by Walker (*7*).

Malfunction Flash

Three of the eight traffic engineers reported using only flashing yellow/red, as described by MUTCD Section 4B-6(7), for malfunctioning traffic signals in their jurisdictions. One of these noted that the MUTCD specifies no flashing mode other than flashing yellow/red.

Of the five engineers who sometimes used flashing red/red, four did so at intersections of two major streets or where there is no clear major street. One of these four noted that MUTCD 4B-6(7), which states that flashing yellow/red is normally used, is applicable only to those intersections where a major and a minor street cross. He went on to say that the logic of 4B-6(7) is that flashing red/red is the only acceptable alternative where two major streets cross.

One engineer specified all-way flashing red for intersections of five or more legs, as implied by the TCDH.

Five of the engineers stated that entering sight distance was considered when selecting a particular mode of flashing operation for signal malfunction. Two of these engineers were confident that no signalized intersections in their jurisdictions had sight restrictions from minor-street stopped positions that required flashing red/red. One of the engineers used flashing red/red at some locations because entering sight distance was restricted. One engineer said that his agency was committed to eliminating sight-distance restrictions rather than resorting to flashing red/red. According to another engineer, all of his agency's intersections had been designed or modified to operate safely under two-way stop control, so that flashing red/red is unnecessary.

The engineers who considered sight distance as related to flashing operation relied on engineering judgment, rather than some computational procedure or the AASHTO Case III sight-distance guidelines, when evaluating entering sight distance.

None of the engineers considered the availability of adequate gap to entering minor-street vehicles when selecting the malfunction-flash mode.

Five of the eight engineers relied to some extent on traffic-control police officers to direct traffic at malfunctioning signals. Two of these engineers routinely requested traffic-control police officers to be sent to major intersections (as determined by the engineer) that went to malfunction flash during heavy traffic periods. Another of the five engineers had identified several critical intersections at which police officers were requested if malfunction occurred during a period of peak traffic volume. Two engineers requested police assistance at such malfunctioning signals if the signal-repair technician was delayed in reaching the intersection or in restoring the signal to proper operation.

Local police departments of five jurisdictions were described as generally cooperative when asked to direct traffic at malfunctioning signals. Officers were reported to be well trained in manual traffic control. One engineer noted that his local police department had no clear directive of its responsibility to assist the traffic engineering agency; officers were only provided if available at the time of signal malfunction.

Two engineers described their police departments as less cooperative, possibly because of manpower constraints, when asked to direct traffic at malfunctioning signals. One of these engineers noted that many of his police officers were unskilled in manual traffic control.

In the jurisdictions investigated, many traffic signals at major intersections were part of closed-loop computer systems, allowing the traffic engineers to monitor signal status continuously. Most of the engineers agreed that, if requested, a traffic-control police officer could be at the scene of a malfunctioning traffic signal within 10 min of agency notification. Even if signal status was monitored continuously, seven of the eight engineers believed that a 10-min travel time to a flashing red/red intersection would produce intolerable congestion during rush hours. Depending on the time of malfunction, several of the engineers believed that a flashing red/red signal along an arterial system could soon cause a bottleneck capable of disrupting traffic operations at adjacent intersections.

The engineers were united in the opinion that the flashing red/red mode is too harmful to main-street-traffic flow to select for signal malfunction just because, at some time of the day, traffic volume at a particular intersection may cause difficulty to minor-street vehicles entering under two-way-stop control. Flashing red/red during rush hours could severely delay a traffic-control police officer (and even emergency vehicles) in arriving at the scene.

SUMMARY

The research produced the following principal findings:

- The MUTCD states that it is normal for a flashing display to be yellow/red. The TCDH and the AASHTO Green Book caution that flashing yellow/red may be inappropriate and hazardous if sight distance is lacking or traffic volume on the major street is moderate or heavy.
- Field observations in metropolitan Atlanta readily turned up 10 signalized intersections that failed to meet AASHTO standards for sight distance and that were programmed for flashing yellow/red.
- Interviews with eight traffic engineering agencies in metropolitan Atlanta showed variation in interpretation of the MUTCD. One engineer noted that the MUTCD specifies no flashing mode other than yellow/red. Another understood the MUTCD to mean that flashing red/red is the only acceptable display where two major streets cross.
- Of the five agencies that sometimes used flashing red/red, only one selected it where sight distance was inadequate. (The other four reported no sight-distance problems.)
- The agencies determined sight distance by engineering judgment rather than by some formal procedure.
- When selecting the flash display, none of the agencies considered the availability of adequate gaps to entering minor-street vehicles.
- Seven of the eight agencies judged that flashing red/red during rush hours would produce intolerable congestion during the estimated 10 min needed for a traffic-control police officer to reach the scene.
- All agencies agreed that flashing red/red should not be selected just because, at some time of day, major-street traffic could cause difficulty to vehicles entering from the minor street.

CONCLUSIONS AND RECOMMENDATIONS

It is concluded that there could be a problem here that needs to be addressed by the traffic engineering profession. If major-street volumes are too heavy for minor-street traffic to enter or cross, or if minor-street traffic cannot see far enough along the major street for safety, there appears to be no acceptable mode of flashing operation.

The literature explains the hazard of flashing yellow/red but offers no traffic-operational solution. Flashing red/red, although possibly increasing the likelihood of rear-end collisions, could reduce the risk of serious right-angle collisions. The engineers interviewed clearly believe that flashing red/red could cause intolerable congestion on the major street during rush hours.

Nonetheless, serious consideration should be given to changing Section 4B-6(7) of the MUTCD to acknowledge the potential hazard of flashing yellow/red. Although it is the most common flashing mode, the MUTCD need not characterize it as "normal." Red/red should be stated to be an acceptable mode of flashing operation but not the only acceptable alternative where two major streets cross.

Traffic engineering agencies should evaluate all their signalized intersections that use flashing yellow/red. For each intersection those times of day should be identified when minor-street traffic likely will have difficulty finding long enough gaps in major-street traffic to avoid collisions. The intersection should be considered "critical" when in flashing mode during these times of day, and solutions such as those described next should be considered.

Traffic engineering agencies should identify their minor-street approaches lacking in AASHTO Case III sight distance. These intersections should be characterized as "critical" when in flashing yellow/red and a reasonably safe solution should be sought for each. For example, the signal status could be monitored continuously by a master located at curbside or at the traffic operations center. When flashing operation is detected, a traffic-control police officer (and the signal-repair crew) could be dispatched to the intersection. Another possible solution could be to use a flashing yellow on the major street; other approaches not having safe sight distance conceivably could have their flashing red indication supplemented by a blank-out sign message such as Right Turn Only. There could be a public information program encouraging motorists facing a flashing red to edge out carefully and then turn right if traffic is heavy or they cannot see very far.

More research by the traffic engineering profession is needed into the issues in flashing operation for malfunctioning traffic signals. Total accident rates and accident rates by category during malfunction flash should be studied. Cost-benefit analysis should be applied to accident data to compare the property and injury costs of flashing yellow/red and flashing red/red. The frequency of signal malfunctions in general and the frequency of signal malfunctions relative to types of controllers, conflict monitors, environmental conditions, and so on, should be investigated.

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Before and After Comparison of Leading Exclusive and Permissive/Exclusive Lagging Left Turn Phasing

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Three different types of left turn phasing were compared: leading exclusive; leading exclusive/permissive; and permissive/exclusive lagging. All three types of phasing were in operation at a single intersection and were changed from one type to another over a period of several months. Cycle length and offsets for progression were kept the same as each type of phasing was implemented to minimize confounding factors. Time-lapse photography was used to collect volume and delay data for 8 hr of operation for each type of phasing. Traffic volume and average delay per vehicle in each hour were statistically analyzed to determine if statistically significant changes occurred between different types of left turn phasing. Most changes in traffic volume were not statistically significant. In general, substantial reductions in delay occurred for both through and left turn movements when the change from leading exclusive to leading exclusive/permissive was made; increases in both through and left turn delay occurred when phasing was changed from leading exclusive/permissive to permissive/exclusive lagging; and delay under permissive/exclusive lagging operation was less than under the original leading exclusive phasing.

Selection of the appropriate type of left turn phasing for an individual intersection is a topic of discussion and debate in the traffic engineering community. Permissive, exclusive, and exclusive/permissive left turn phasing offer trade-offs in through vehicle delay, left turn delay, and safety. Leading and lagging left turn operation each also have advantages and disadvantages. The effects of lagging left turn phasing are of particular interest in Arizona where it has been adopted by three major jurisdictions. The operational effects, in terms of delay, are evaluated in this study.

The types of left turn phasing are as follows:

- Permissive left turn—Vehicles are allowed to make a turn on a circular green indication but must yield to opposing traffic (sometimes called a permitted left turn).
- Exclusive left turn—Vehicles are allowed to make a turn only on a green arrow indication and have the right-of-way while the green arrow is displayed (sometimes called a protected left turn).
- Exclusive/permissive—Vehicles are allowed to make a turn either on a green arrow indication or on the circular green indication, but left turning vehicles must yield to opposing traffic while the circular green indication is displayed.

In addition to the above, other variations are exclusive and exclusive/permissive phasing. These variations are known as leading and lagging left turn phasing and refer to the relationship of the left turn phase to the through phase. In leading left turn phasing, the exclusive left turn phase occurs before the opposing through phase begins. In lagging left turn phasing, the exclusive left turn phase occurs after the termination of the opposing through phase. The variations are called leading exclusive, lagging exclusive, leading exclusive/permissive, and permissive/exclusive lagging. In typical application, the exclusive portion of the phase appears only when there is left turn demand.

In 1988 and 1989 the city of Scottsdale, Arizona, converted most intersection approaches that had leading exclusive left turn phasing to permissive/exclusive lagging operation. Conversion was done in two steps to minimize confusion for motorists. The first modification was to change from leading exclusive left turn phasing to leading exclusive/permissive left turn phasing. Then, after an adjustment period of approximately 8 weeks, the signal was again modified to operate as permissive/exclusive lagging left turn phasing. This staged conversion from leading to lagging turning allowed a unique opportunity to evaluate and compare the operational effects of three different types of left turn phasing. Vehicle delay was the principal operational effect that was evaluated. Data were collected at one intersection for three different conditions: leading exclusive (condition D1); leading exclusive/permissive (condition D2); and permissive/exclusive lagging (condition D3). The intersection selected for study was Hayden Road and Indian School Road.

STUDY OBJECTIVE

The general objective of this study was to evaluate the changes in the delay characteristics on the north and south approaches of the intersection of Hayden Road and Indian School Road under each of the three phasing conditions (D1, D2, and D3). Specific issues considered were as follows:

1. Did the traffic volume for through and left turning vehicles change significantly under each of the three conditions?
2. Did the delay for the through and left turning vehicles change significantly under each of the three conditions?
3. Did the left turn volume, as a percent of total volume, change under each of the three conditions?
4. What was the effect of the change in delay in terms of the economic cost of delay?

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STUDY PROCEDURE

A before and after study was used to meet the study objectives. Data were collected at the same intersection for three different conditions of traffic signal phasing (D1, D2, and D3). Cycle length and offsets (for signal progression at the study intersection and adjacent intersections) remained the same during all three conditions to reduce confounding factors.

Intersection Description

At the intersection of Hayden Road and Indian School Road (Figure 1) in Scottsdale, Hayden Road is a north/south major arterial with three lanes in each direction plus a two-way left turn lane or raised median with left turn pockets. Hayden Road has a large range in traffic volumes during the day. Indian School Road is an east/west collector roadway at this location with two lanes in each direction plus a left turn lane.

The existing improvements immediately adjacent to the intersection include a raised median on Hayden Road north of Indian School Road which continues to the north and provides periodic left turn pockets for turning vehicles. There is also a short segment of raised median on Hayden Road south of Indian School Road. This median terminates approximately 50 ft south of the intersection and a continuous two-way left turn lane extends further to the south. Approximately 250 ft of left turn storage is available for southbound left turning traffic. The two-way left turn lane provides almost unlimited left turn storage for northbound left turns.

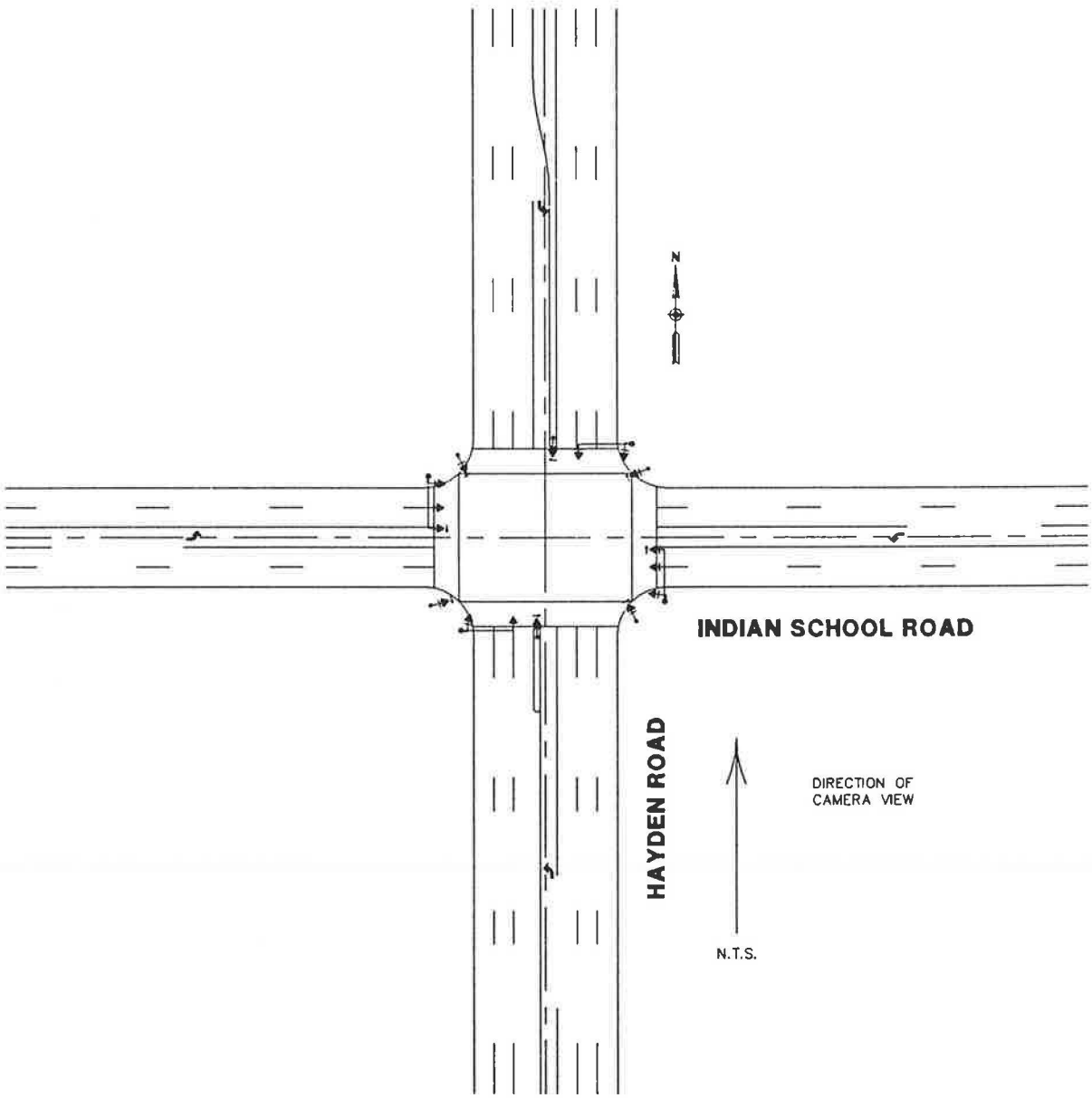

FIGURE 1 Intersection diagram.

For condition D1 the left turn signals consisted of a three-arrow signal face (red arrow, yellow arrow, green arrow). On November 18, 1988, five-section signal heads (red ball, yellow ball, green ball, yellow arrow, green arrow) were installed for condition D2. For condition D3, about 8 weeks later, additional signs were added with the legend Green Arrow After (symbolic green ball) indicating the green arrow would appear after the green ball indication.

Method of Data Collection

Time-lapse photography was identified as a data collection method that could accurately record volume and delay information and provide a permanent record. The actual camera location for filming was on Hayden Road approximately 500 ft south of Indian School Road on the east side of the street. The camera was mounted on a street light pole at a height of approximately 30 ft. This provided the most advantageous possible view of both northbound and southbound traffic. The limited field of view of the camera precluded observation of eastbound and westbound traffic.

Table 1 lists the dates and times of filming for conditions D1, D2, and D3. Each roll of film contained 3,600 frames recorded at one frame per second. Reloading the film required a small time interval between each roll. The same day of the week (Thursday) was used for all three conditions to minimize the possible variations in traffic volume.

Method of Data Reduction

Data reduction for each set of film was accomplished using Traffic Data Input Program (TDIP), version 2.0. Using this program, a time was recorded for each vehicle as it entered the field of view. The time was again recorded as the vehicle crossed the stop bar. The amount of time required for the vehicle to travel between these two points, as well as the total number of vehicles, was calculated. Because the camera was approximately 500 ft south of the intersection, only the northbound and southbound approaches could be observed.

Once all the data were transcribed to determine the total time each vehicle was within the field of view, the normal travel time was determined and subtracted from the observed total time for each vehicle. The result was the actual total vehicle delay experienced because of the signalized control. Accordingly, vehicles that entered the field of view and traveled at approximately the posted speed to cross the stop bar experienced no delay. The delay for vehicles that were required to stop at the traffic signal or were slowed by queues at the intersection was included in the total delay and the calculation for average delay per vehicle for the intersection.

The normal travel time was calculated to be approximately 5 sec using the posted speed of 45 mph (66 ft/sec) and the distance from the stop bar to the beginning of the field of view of 330 ft. A sample of the data reduction is included elsewhere (*1*).

TABLE 1 Filming Times*

Hour	D1	D2	D3
	11/2/88	1/12/90	2/22/90
1	9:09 a.m.	8:44 a.m.	8:44 a.m.
2	10:11 a.m.	9:47 a.m.	9:47 a.m.
3	11:14 a.m.	10:49 a.m.	10:49 a.m.
4	12:16 p.m.	11:51 a.m.	11:51 a.m.
5	13:18 p.m.	12:53 p.m.	12:53 p.m.
6	14:20 p.m.	13:55 p.m.	13:55 p.m.
7	15:22 p.m.	14:57 p.m.	14:57 p.m.
8	16:24 p.m.	15:59 p.m.	15:59 p.m.

*The starting time for each roll of film is shown

SIGNAL TIMING

Traffic signal control was provided by an eight-phase controller, which is intertied to the city's central traffic signal control system for progression control. The intersection operated as a semiactuated signal with detectors on Indian School Road for through movements and on all approaches for left turn movements. The connection to the city's central control computer allows for implementation of sophisticated variations to the signal timing plan to react to changes in traffic volumes by time of day. The timing plan provided three different cycle lengths and progression offsets during the observation period at predetermined times of the day. The cycle lengths and offsets, as well as the times of implementations, remained the same for each of the sets of data. Progression offsets for each phasing plan were measured from the start of the north/south through movement. The cycle lengths and offset for each of the timing plans are shown in Table 2. Maximum green times for the left turn phase generally varied from 15 to 20 sec with minimum green times, if there was a call for a left turn phase, of approximately 6 sec (including the clearance interval). Unused green time from the left turn phases and the east/west through phase was allocated to the north/south through phase.

TABLE 2 Signal Timing*

Time of Day	8:30 - 10:30	10:30 - 15:30	15:30 - 18:00
Cycle Length	94 s	110 s	112 s
Offset	2s	16s	100s
NB Left	17s (max)	17s (max)	18s (max)
SB Left	17s (max)	15s (max)	16s (max)
NB Through	32s	45s	40s
SB Through	32s	47s	42s

*Timing is the same for D1, D2 and D3

INTERSECTION OPERATION

Phasing diagrams for each condition are shown in Figure 2. The condition D1 signal phasing was leading exclusive left turn phasing. Under this phasing plan, the protected leading arrow was actuated when a vehicle or vehicles were detected in the left turn lane. The left turns would operate independently with either the north, or south, or both left turn arrows occurring before the north/south through movements. Similarly, the left turn indications would end individually when the detectors sensed that no more vehicles were waiting to turn or when the traffic signal controller detected that the maximum green time for the left turn arrows had expired. Once the left turn phase was completed, the red arrow was

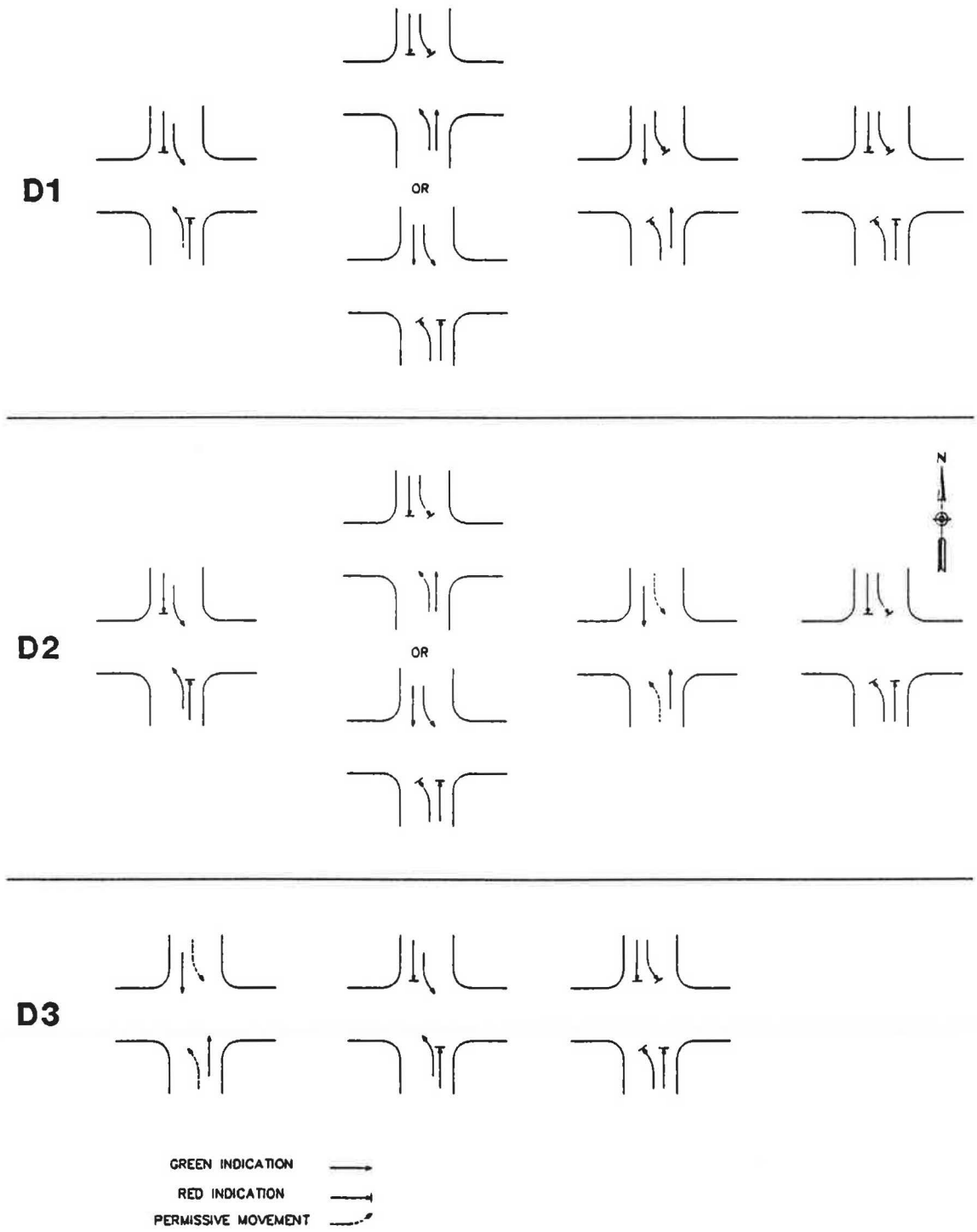

FIGURE 2 Phasing diagram.

displayed and any other arriving left turn vehicles were required to wait until the next cycle. This phasing allowed for utilization of minimum green time for the left turns because, once the queues were dissipated in either left turn direction, the corresponding arrow would terminate allowing the opposing through movement to begin. This phasing plan caused left turning vehicles arriving with the platoon from the adjacent intersection to wait until the next cycle for the left turn arrow. It also caused variations in the start of the green phase because of the variations in left turn demand. One disadvantage of the leading exclusive left turn phase was that a long queue in the through lane adjacent to the left turn lane could block access to the left turn lane (where there is a raised median). Thus, some arriving left turn vehicles would have to wait until the next cycle to make their turn.

Condition D2 was leading exclusive/permissive. It provided for exclusive leading left turns followed by permissive left turns. Under this phasing pattern the left turn arrows operated the same as in D1 except that after termination of the exclusive left turn phase (the green arrow), no red arrow was displayed for left turning vehicles. This allowed any left turning vehicles remaining in the queue or arriving after the termination of the exclusive phase to utilize gaps in traffic and the yellow phase to execute left turning movements.

Condition D3 was permissive/exclusive lagging left turn phasing. In this phasing, the start of the green time for through movement was consistent for both directions because no arrows appeared at the beginning of the phase. Drivers of left turning vehicles saw a green ball at the beginning of the through phase, allowing them to use any available gaps in the traffic and the clearance interval to execute left turn movements. Following the termination of the through phase, a green arrow appeared for left turns allowing remaining left turning vehicles to execute their maneuvers. This type of phasing does not cause blockage of the left turn lane by the adjacent through lane because the arrow occurs after the queue that may have accumulated in the through lane has dissipated. However, this phasing does require that both through movements terminate at the same time even if there is a demand for left turns only on one approach to the intersection.

STUDY FINDINGS

Volume

Tables 3 and 4 show the hourly volume and an 8-hr total for each condition (D1, D2, and D3) and for each movement (north, south, through, and left). Traffic volume did change for each of the measured traffic flows between D1, D2, and D3. For comparison, Tables 3 and 4 also show the percentage change in traffic volume for D2 compared to D1, D3 compared to D2, and D3 compared to D1. Examination of the tables will show the changes. The percent of traffic volume turning left is shown in Table 5. These data indicate that the turn percentage for the northbound traffic was 12.6, 11.0, and 12.7 percent in conditions D1, D2, and D3. For southbound traffic the left turn percent was 7.9, 7.1, and 7.3 percent in D1, D2, and D3. Although traffic volumes did change during the various periods of data collection, the percentages for the left turning vehicles remained fairly stable.

TABLE 3 Northbound Volume Data

Northbound Through Volume (vehicles per hour)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	1190	1312	1373	10.3%	4.6%	15.4%
2	1109	1182	1217	6.6%	3.0%	9.7%
3	1168	1299	1293	11.2%	-.5%	10.7%
4	1199	1383	1477	15.3%	6.8%	23.2%
5	1334	1432	1466	7.3%	2.4%	9.9%
6	1378	1602	1526	16.3%	-4.7%	10.7%
7	1697	1742	1813	2.7%	4.1%	6.8%
8	2047	2022	2228	-1.2%	10.2%	8.8%
Total	11122	11974	12393	7.7%	3.5%	11.4%

Northbound Left Turn Volume (vehicles per hour)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	216	173	187	-19.9%	8.1%	-13.4%
2	200	172	235	-14.0%	36.6%	17.5%
3	218	151	233	-30.7%	54.3%	6.9%
4	195	188	232	-3.6%	23.4%	19.0%
5	201	227	249	12.9%	9.7%	23.9%
6	187	208	219	11.2%	5.3%	17.1%
7	164	174	235	6.1%	35.1%	43.3%
8	228	191	220	-16.2%	15.2%	-3.5%
Total	1609	1484	1810	-7.8%	22.0%	12.5%

TABLE 4 Southbound Volume Data

Southbound Through Volume (vehicles per hour)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	1135	1099	1526	-3.2%	38.9%	34.4%
2	1034	1065	1293	3.0%	21.4%	25.0%
3	1018	1143	1234	12.3%	8.0%	21.2%
4	1029	1121	1311	8.9%	16.9%	27.4%
5	1129	1121	1440	-.7%	28.5%	27.5%
6	1344	1280	1422	-4.8%	11.1%	5.8%
7	1512	1478	1650	-2.2%	11.6%	9.1%
8	1612	1702	1775	5.6%	4.3%	10.1%
Total	9813	10009	11651	2.0%	16.4%	18.7%

Southbound Left Turn Volume (vehicles per hour)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	74	84	68	13.5%	-19.0%	-8.1%
2	69	47	81	-31.9%	72.3%	17.4%
3	124	75	110	-39.5%	46.7%	-11.3%
4	87	96	133	10.3%	38.5%	52.9%
5	100	105	149	5.0%	41.9%	49.0%
6	116	102	122	-12.1%	19.6%	5.2%
7	116	121	105	4.3%	-13.2%	-9.5%
8	150	131	144	-12.7%	9.9%	-4.0%
Total	836	761	912	-9.0%	19.8%	9.1%

TABLE 5 Left Turn Volume Percentage

	Data 1		Data 2		Data 3	
	NB	SB	NB	SB	NB	SB
Volume Summary (total number of vehicles)						
Thru	11122	11974	12393	9813	10009	11651
Left	1609	1484	1810	836	761	912
Total	12731	13458	14203	10649	10770	12563
Left Trn%	12.6%	11.0%	12.7%	7.9%	7.1%	7.3%

TABLE 6 Northbound Delay Data

Northbound Through Delay (seconds per vehicle)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	9.9	8.5	10.7	-14.1%	25.9%	8.1%
2	14.1	9.9	13	-29.8%	31.3%	-7.8%
3	18.1	14.4	15.3	-20.4%	6.3%	-15.5%
4	17.8	16.4	15.8	-7.9%	-3.7%	-11.2%
5	17.4	15.9	13.9	-8.6%	-12.6%	-20.1%
6	19.9	19.7	17.2	-1.0%	-12.7%	-13.6%
7	20.2	18.4	19	-8.9%	3.3%	-5.9%
8	31.4	22.2	28.9	-29.3%	30.2%	-8.0%
Avg	18.6	15.7	16.7	-15.7%	6.7%	-10.1%

Northbound Left Turn Delay (seconds per vehicle)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	54.4	21.1	29.9	-61.2%	41.7%	-45.0%
2	72.3	22.7	29.2	-68.6%	28.6%	-59.6%
3	76.9	19.3	41.9	-74.9%	117.1%	-45.5%
4	56.1	22	38.8	-60.8%	76.4%	-30.8%
5	59.9	18.8	46.5	-68.6%	147.3%	-22.4%
6	47.3	23.9	41.3	-49.5%	72.8%	-12.7%
7	55.2	28.9	47.4	-47.6%	64.0%	-14.1%
8	55.1	70.3	63.5	27.6%	-9.7%	15.2%
Avg	59.7	28.4	42.3	-52.4%	49.1%	-29.1%

TABLE 7 Southbound Delay Data

Southbound Through Delay (seconds per vehicle)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	20.4	21.9	20.9	7.4%	-4.6%	2.5%
2	20.3	18.8	16.5	-7.4%	-12.2%	-18.7%
3	19.2	18.7	16.6	-2.6%	-11.2%	-13.5%
4	16.6	17.2	16.1	3.6%	-6.4%	-3.0%
5	15.3	15.5	13.9	1.3%	-10.3%	-9.2%
6	17.9	15.9	16.3	-11.2%	2.5%	-8.9%
7	17.7	18.8	15.3	6.2%	-18.6%	-13.6%
8	22.7	67.3	23.3	196.5%	-65.4%	2.6%
Avg	18.8	24.3	17.4	29.3%	-28.4%	-7.5%

Southbound Left Delay (seconds per vehicle)

Hour	D1	D2	D3	1vs.2	2vs.3	1vs.3
1	33.7	17.4	27.6	-48.4%	58.6%	-18.1%
2	47.2	17.8	29.2	-62.3%	64.0%	-38.1%
3	46.7	18.1	36.5	-61.2%	101.7%	-21.8%
4	44.2	18.7	40.8	-57.7%	118.2%	-7.7%
5	44.1	19.5	31	-55.8%	59.0%	-29.7%
6	40.8	17.1	33.5	-58.1%	95.9%	-17.9%
7	34.8	22.4	32.3	-35.6%	44.2%	-7.2%
8	40.1	27.8	47.7	-30.7%	71.6%	19.0%
Avg	41.5	19.9	34.8	-52.1%	75.4%	-16.0%

Delay

Because a principal measure of performance for at-grade signalized intersections is delay, changes in delay give one direct measure of the effects of changing the type of left turn phasing. Tables 6 and 7 show delay for each condition and for each movement. The percentage change between conditions is also shown.

VALUE OF TRAVEL TIME

The value of travel time savings is an important component of user benefits that can be used to quantify the effects of the change from leading to lagging left turn operation in terms of economic costs. *A Manual on User Benefit Analysis of Highway and Bus Transit Improvements* (2) is a widely used reference on the value of travel time savings. For "medium time savings" and "average" trip type the manual gives a travel time value of $1.80/hr per person (in 1975 dollars).

This value must be adjusted for both vehicle occupancy and inflation (the $1.80 value is based upon 1975 data). The value for median time savings was selected to approximate the systemwide improvement that would be expected for a typical trip through several signalized intersections with lagging left turn phasing. Adjusting for an assumed average occupancy of 1.56 adults per vehicle as recommended in the manual (2) and inflation results in a 1990 value of travel time of $6.00 per vehicle hour.

The total observed delay over the 8 hr of filming was determined by multiplying the average delay per vehicle for each hour of observation by the number of vehicles observed during each respective hour. The delay was totaled for north and south through and left turn movements for each condition. Using the total observed delay and the $6.00 per vehicle hour value of travel time, the daily value of passenger time saved under D2 (compared to D1) was calculated to be $56.40 per day or $20,586 per year. The daily value of travel time saved under D3 (compared to D1) was calculated to be $6.60 per day or $2,409 per year. It is emphasized that these savings are based upon an 8-hr day and do not reflect additional savings acrrued during the other 16 hr.

STATISTICAL ANALYSIS

Both delay and volume (8-hr averages) varied among conditions D1, D2, and D3. To determine if the variations were statistically significant, the two-sample *t*-test was selected as the test. This test assumed that each population is normal and that the values of the two population variances are equal.

Before applying the *t*-test, the variances of each set of data were compared. The comparisons showed that the variances were roughly similar in magnitude, except for the southbound through delay. The *t*-test was, therefore, judged to be appropriate for each data set except southbound through delay. A level of significance of 0.05 was selected.

Volume and Delay

Tables 8 and 9 present statistical summaries and note which changes were statistically significant based upon the two-sample *t*-test. The only changes in volume that were statistically significant were as follows:

- Southbound through volume— +18.7 percent beteeen D1 and D3,
- Northbound left turn volume— +22.0 percent between D2 and D3, and
- Northbound left turn volume— +12.8 percent between D1 and D3.

The changes in delay that were statistically significant were as follows:

- Northbound left turn delay— −52.4 percent between D1 and D2,
- Northbound left turn delay— −29.1 percent between D1 and D3,
- Southbound left turn delay— −52.1 percent between D1 and D2,
- Southbound left turn delay— +75.4 percent between D2 and D3, and
- Southbound left turn delay— −16.0 percent between D1 and D3.

The statistical analysis of the data for the southbound through volume showed that the variance for D2 is 30 to 50 times higher than the variances for D1 and D3. Because one of the assumptions for the use of the *t*-test is that the variances are equal, the *t*-test cannot be used for this data. No other tests have been shown to be appropriate to evaluate data with these characteristics and no statistical conclusion can be drawn.

TABLE 8 Statistical Summary—Volume (Vehicles per Hour)

	Northbound Through Volume			Southbound Through Volume		
	D1	D2	D3	D1	D2	D3
Avg	1390	1497	1549	1227	1251	1456
Std	302.9	259.0	306.2	218.3	214.8	174.3
Var	91,737.9	67,068.7	93,759.4	47,667.6	44,866.9	30,323.2
% Chng	1vs2	2vs3	1vs3	1vs2	2vs3	1vs3
	7.7%	3.5%	11.4%	2.0%	16.4%	18.7%*

	Northbound Left Turn Volume			Southbound Left Turn Volume		
	D1	D2	D3	D1	D2	D3
Avg	201	186	226	105	95	114
Std	18.9	22.2	17.3	25.6	24.9	27.0
Var	355.6	490.8	297.7	654.0	628.4	731.5
%Chng	1vs2	2vs3	1vs3	1vs2	2vs3	1vs3
	-7.8%	22.0%*	12.8%*	-9.0%	19.8%	9.1%

*Denotes change in average value which is statistically significant at the .05 level of significance.

TABLE 9 Statistical Summary—Delay (Seconds per Vehicle)

	Northbound Through Delay			Southbound Through Delay		
	D1	D2	D3	D1	D2	D3
Avg	18.6	15.7	16.7	18.8	24.3	17.4
Std	5.8	4.4	5.2	2.2	16.4	2.9
Var	33.3	19.2	26.8	4.9	268.1	8.5
	1vs2	2vs3	1vs3	1vs2	2vs3	1vs3
%Change	-15.7%	6.7%	-10.1%	29.3%**	-28.4%**	-7.5%**

	Northbound Left Turn Delay			Southbound Left Turn Delay		
	D1	D2	D3	D1	D2	D3
Avg	59.7	28.4	42.3	41.5	19.9	34.8
Std	9.3	16.1	10.2	4.8	3.4	6.2
Var	86.4	259.7	104.0	22.7	11.5	38.9
	1vs2	2vs3	1vs3	1vs2	2vs3	1vs3
%Change	-52.4%*	49.1%	-29.1%*	-52.1%*	75.4%*	-16.0%*

* Denotes change in average value which is statistically significant at the .05 level of significance.

** No appropriate statistical test for evaluation of changes.

Relationship of Changes in Volume and Delay

Table 10 summarizes changes in volume and delay. A review shows that although the northbound through volume and the northbound through delay changed between all the data sets, no statistically significant difference occurred. However, although the volumes increased between each of the sets of data, the delay decreased between D1 and D2, and actually increased between D2 and D3 while still maintaining an overall decrease in delay between D1 and D3.

Similarly, the northbound left turn volumes, although experiencing statistically significant increases in volume between D1 and D3, also showed a significantly lower left turn delay between D1 and D3. Like the northbound through delay, the northbound left delay increased between D2 and D3 (it was not statistically significant). The total change between D1 and D3 was significantly lower by 29.1 percent or 17.4 sec per vehicle.

The relative changes in delay, compared to the changes in volume, suggest that changes in volume were a less important contributor to changes in delay than was the change in the type of left turn phasing. A stronger statement can be made in comparing conditions D1 and D3. Whereas volume increased 12.8 percent, delay decreased 29.1 percent.

The southbound left turn volume and delay also exhibited changes that paralleled those observed in the northbound left turn volume and delay. Delay decreased 52.1 percent between D1 and D2, increased 75.4 percent between D2 and D3, and decreased 16.0 percent between D1 and D3. These changes were all statistically significant and do not appear to be compounded by any changes in the left turn volumes because no statistically significant variation occurred in southbound left turn volume.

The changes observed in the southbound through delay did not correspond to those observed for the northbound through

TABLE 10 Changes in Volume and Delay

	Percent Change		
	1 vs. 2	2 vs. 3	1 vs. 3
Northbound Through Volume	7.7 %	3.5 %	11.4 %
Northbound Through Delay	-15.7 %	6.7 %	-10.1 %
Southbound Through Volume	2.0 %	16.4 %	18.7 % *
Southbound Through Delay	29.3 %**	-28.4 %**	-7.5 %**
Northbound Left Turn Volume	-7.8 %	22.0 % *	12.8 % *
Northbound Left Turn Delay	-52.4 % *	49.1 %	-29.1 % *
Southbound Left Turn Volume	-9.0 %	19.8 %	9.1 %
Southbound Left Turn Delay	-52.1 % *	75.4 % *	-16.0 % *

* Denotes change in average value which is statistically significant at the .05 level of significance.

** No appropriate statistical test for evaluation of changes.

TABLE 11 Hourly Comparisons

Through Volume and Delay	D1	D2	D3
Northbound			
Hour	five	one	one
Hourly Volume (vehicles)	1334	1312	1373
Delay per vehicle (sec/veh)	17.4	8.5	10.7
Southbound			
Hour	six	six	four
Hourly volume	1344	1280	1311
Delay per vehicle	17.9	15.9	16.1

Left Turn Volume and Delay	D1	D2	D3
Northbound			
Hour	one	six	six
Hourly Volume (vehicles)	216	208	219
Delay per vehicle (sec/veh)	54.4	23.9	41.3
Southbound			
Hour	six	seven	six
Hourly volume	116	121	122
Delay per vehicle	40.8	22.4	33.5

and the northbound and southbound left turns. It appears that the observations for D2 and specifically for Hour 8 in the data were dramatically different than any of the observations for any other hour observed. The difficulty in observing southbound traffic and precisely measuring delay caused by the distance from the camera and limited resolution of the time-lapse photography appears to have compounded the effects of the change in phasing on the observed delay. Therefore, these data do not appear to be valid when compared with the other data.

Additional efforts were made to identify trends in the data. By selecting specific hours for each condition with similar traffic volumes, a comparison could be made of corresponding delay (see Table 11). For example, for the northbound through volume, Hour 5 in D1 was compared with Hour 1 in D2 and Hour 1 in D3. For each of the sets with similar volumes, the corresponding delay data showed a decrease in delay between D1 and D2, an increase in delay between D2 to D3, and an overall decrease in delay between D1 to D3. These comparisons seem to confirm the general trends observed in the delay under each condition and indicate that the volume changes did not appreciably affect the results.

CONCLUSIONS

The three conditions observed in the study were as follows:

- Condition D1—Leading exclusive left turn phasing,
- Condition D2—Leading exclusive/permissive left turn phasing, and
- Condition D3—Permissive/exclusive lagging left turn phasing.

1. Northbound and southbound through volumes increased between each of the sets of data. These increases are generally consistent with the historical trend in traffic volumes associated with the continued growth in the urban area between observation times. The only change in through volume found to be statistically significant was the change in the southbound traffic volume between D1 and D3.
2. The left turn volumes decreased between D1 and D2 (these changes were not statistically significant). Left turn volumes increased between D2 and D3. This change was statistically significant for northbound but not for southbound traffic. The overall left turn volumes between D1 and D3 increased; this increase was statistically significant for northbound but not for southbound traffic.
3. The delay for the northbound through volumes did not change significantly between any of the three conditions. The left turn delays—both northbound and southbound—decreased by a statistically significant 52 percent for each direction of traffic between D1 and D2. From D2 to D3, the left turn delay increased a statistically significant 75.4 percent for the southbound lefts and 49.1 percent for the northbound lefts (not statistically significant). Between D1 and D3, delay for left turns—both northbound and southbound—did decrease by a statistically significant amount, 16 percent for southbound left turns and 29.1 percent for northbound left turns. The trend shows a decrease in delay from D1 to D2 and an increase in delay from D2 to D3. The data suggest that changes in volume were a less important contributor to changes in delay than were changes in the type of left turn phasing.
4. The left turn percentages for each of the three data sets were generally consistent and did not appear to significantly affect the delay data that were observed.
5. The change from leading exclusive (D1) to permissive/exclusive lagging (D3) left turns in a staged procedure provided a unique opportunity to observe changes in vehicle delay under varying conditions. The addition of the permissive left turn at the intersection clearly provided a benefit to the left turning vehicles by reducing the delay experienced on both approaches. The change from leading exclusive/permissive (D2) to permissive/exclusive lagging (D3) caused the delay for left turning traffic to increase a statistically significant amount.

ACKNOWLEDGMENT

The authors gratefully acknowledge the assistance and cooperation provided by the city of Scottsdale in conducting this study. More detailed documentation on this research may be found elsewhere (*1*).

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Innovative Passive Device Studies and Demonstrations Currently Being Conducted in the United States and Canada

EUGENE R. RUSSELL

In the late 1960s a flurry of new warning devices were proposed to improve conspicuity, or driver awareness, or understanding, or all three at railroad-highway crossings. A period of little or no activity in regard to these devices followed, probably because the emphasis in the 1970s and 1980s was on upgrading grade crossings to active warning devices on a priority basis. Most high-volume grade crossings have now been upgraded; however, thousands of passive grade crossings on low-volume roads exist where expensive active devices cannot be justified on a cost-effective basis. Thus, low-cost, innovative devices that are more effective have drawn renewed interest that will likely continue throughout the 1990s. Most of the devices presented are too new to be reported in the published literature. Most are in the early development stage and some have yet to be proven or studied adequately. However, the argument is presented that if a successful effort is to be made in the area of low-cost, innovative devices for low-volume grade crossings, a coordination of effort is needed, starting with an awareness of these as-yet fragmented studies. Brief backgrounds of innovative devices and discussion of recent efforts include the Conrail device (Ohio and Kansas); new retroreflective materials (Arizona, Minnesota, Vermont, and Nebraska); retroreflective trackside objects (Arizona); a proposed 3M/BN passive warning sign; a proposed adaptation of the variable aspect signs to be used at grade crossings; a Texas study to enhance the effectiveness of the current, standard crossbuck; a human factors study being conducted at the FHWA Turner Fairbank research facility; a Canadian study of new sign systems at passive crossings using intermediate signs; and a before and after study of the effects of an Operation Lifesaver media blitz on driver behavior at crossings.

In the 1960s considerable activity focused on finding innovative devices for passive railroad-highway grade crossings or the approaches thereto. Some devices were intended to increase conspicuity but most were intended to give the driver more information (e.g., differentiating between active and passive crossings, indicating the angle of the crossing, or providing a more meaningful symbol to aid driver understanding). Then a lapse of several years in these efforts followed. This was probably because of the emphasis on upgrading hazardous crossings on a priority basis through a massive, federally funded program that provided categorical grant money for this purpose.

Now that this highly successful federal program has resulted in upgrading the high-priority grade crossings, attention is once again being focused on improving low-volume crossings where low highway or train traffic does not justify the cost of active warning devices. Thus, the emphasis is once again on finding effective, low-cost, innovative devices to improve safety at low-volume grade crossings.

Many of these devices are too new to be reported in the literature. Most are in the early development phase, and some have yet to be proven or studied. If a successful effort is to be made in this area, coordination of the overall effort is needed. As a first step to a more coordinated effort, a wider audience of researchers, developers, manufacturers, public officials, and the general public needs to be aware of several isolated studies and demonstrations currently underway in the United States.

GENERAL REVIEW OF INNOVATIVE DEVICES FOR PASSIVE GRADE CROSSINGS

The most recent comprehensive summary of innovative devices for passive grade crossings that have been proposed or studied is contained in a COMSIS Corporation report on driver behavior at rail-highway crossings (*1*). Some of the material in this first section is paraphrased from the COMSIS report with references as cited in the report. This material provides a very brief background as a setting for presenting current studies.

Advanced Warning Sign

Railroad advance warning signs are generally considered to be well-understood indications of a grade crossing ahead (*2–5*). However, considerable evidence exists that beyond good, general recognition, traffic-control devices at grade crossings are not well understood (i.e., drivers often have poor comprehension of their exact meaning and of the proper action required) (*3,6–8*). Ruden, Wasser et al. believed that because the railroad advance warning sign was round, whereas most others are diamond-shaped, it could cause confusion (*9*). His study team developed three new signs, all presenting some version of an "RxR" symbol on a yellow, diamond-shaped field.

As early as 1968, NCHRP Report 50, one of the most comprehensive reviews of the grade-crossing safety factors,

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recognized a need for signs that could help drivers more clearly understand the action expected at a particular grade crossing (*10*). The report suggested a variety of advance warning signs, all using diamond-shaped sign fields that met three important criteria:

1. The sign better met drivers' sterotypes regarding warnings,
2. The sign had more meaningful content, and
3. The sign provided improved information content by discriminating between active and passive crossing sites.

This author believes that a need for signs meeting the third criterion still exists. That is, the proper action expected of drivers approaching a grade crossing and the consequences of taking inappropriate action differ at passive and active crossings. A driver approaching a passive crossing has a heavier responsibility than at an active crossing. At a passive crossing the driver must determine if a train is at or near the crossing and whether it presents a danger. In rural areas it is sometimes difficult (and almost impossible at night) to determine, at a safe distance from the grade crossing, whether it is an active crossing because the only difference is an unlit pair of flashers.

Schoppert and Hoyt proposed passive signs that diagrammatically showed the crossing angle (*10*). At a passive grade crossing, the angle of crossing is important. If the grade crossing is skewed, there is one quadrant in which a driver must execute a more definitive (over the shoulder) head movement to see if there is danger. In these cases, with the current, standard sign at night, it is not at all clear where to look. For example, at a sharply skewed grade crossing, a driver could conscientiously look to the right (at a 90° angle) and see no evidence of a train (headlight), whereas one could be approaching from over the driver's right shoulder (at about 135°) and not be seen.

Crossbuck Sign

Drivers generally recognize the standard crossbuck and know that it marks a grade crossing. Because no comparable sign is encountered in normal highway driving, not all drivers fully understand the crossbuck's meaning or what action is required to ensure their safety. In terms of driver action it really means "yield to a train when present," but this message may not get across to the general driving public.

The COMSIS report reviewed a number of studies conducted to improve the crossbuck sign (*3,4,9*). Signs with and without wording, with black letters on white background, blank signs with a red border, and blank yellow signs with a black border were tested (*4*).

Another study compared a blank white version with either a 2-in. red or 2-in. black border (*5*). Subject recognition of the worded crossbuck was 100 percent. In recognition studies of blank versions, the percentage correct (recognition) ranged from 70 to 83 percent. The COMSIS report concluded that, "Despite these problems [some biases were noted in the studies] it appears that the crossbuck itself has meaning to many subjects, even without the wording" (*1*).

Schoppert and Hoyt suggested that the crossbuck be incorporated into a larger sign field and that the field be an inverted triangular field (*10*). The larger field provides better target value, and the inverted triangle is similar to the standard yield sign, which would presumably convey the message to yield.

This author believes that the standard crossbuck shape has sufficient conspicuity and recognition to mark a crossing. Promoting proper action would be better accomplished through increased education (e.g., drivers' education in high school or Operation Lifesaver programs). The crossbuck's effectiveness could also be enhanced through additional information from the advanced warning sign, such as differentiating active and passive grade crossings, indicating the crossing angle, or providing additional information on or near the crossbuck, such as with a standard yield sign or an added message. Additional word messages or a unique sign or symbol on the crossbuck post itself may also be helpful.

The COMSIS report reviewed many studies that mostly relate to increased conspicuity of advance signs, crossing signs, or combinations. The use of red and white within signs to improve conspicuity was suggested by Ruden (*2,4,11*). The use of the unique standard shapes of grade crossings, round advance warning and crossbuck, is sometimes considered an advantage for drawing attention, although at other times it is questioned because it violates driver expectancy about warning sign characteristics. One study found wide differences in driver recall of visually similar signs, and its author argued that the degree of risk or demand presented by the sign message was the major factor (*12*).

This author suggests that an analysis of the literature indicates most innovative devices should either improve understanding or be directed toward specific hazards or demanding situations.

Illumination

Although not necessarily new and innovative, illumination of a grade crossing is a low-cost improvement and could effect significant reductions in night accidents where vehicles run into trains occupying the crossing. Russell and Konz studied grade-crossing illumination extensively and found (pre-1980 data) that the incidence of vehicles striking trains rose from 23 percent of all vehicle-train collisions during the day to 46 percent at night (*12–16*). Accident data from 1986 to 1987 show an increase from 22 percent during the day to 33 percent at night (*1*). The data from both periods reflect a greater percentage of night accidents in which vehicles run into trains, indicating both the drivers' difficulty of seeing trains when approaching unilluminated crossings at night and the difficulty of becoming alerted to the crossing itself and determining whether it is active or passive (i.e., being able to see signals in the inactivated mode). It is this author's experience from extensive night driving on midwestern low-volume roads that recognizing inactive flashers at night is very difficult.

Russell and Konz recommended a minimum of two lights, one on each approach (*12*). They also recommended an average of 2 ft-c of light on the side of the train (*12*). Russell, Konz, and Mather have tried to promote the concept that lighting standards should relate to a minimum amount of light on the side of trains.

Mather has promoted the use of illumination in Oregon and has conducted several demonstrations at Oregon grade

crossings. He essentially followed the recommendations for an average of 1 ft-c on the side of the train. Mather has also developed some suggested guidelines for light type, type of luminaire cutoff, height, and location.

DEVICES CURRENTLY BEING USED, PROMOTED, OR DEVELOPED

A range of devices are currently being used, promoted, developed, or demonstrated as low-cost devices at low-volume grade crossings. Most are based on the concept of more effective retroreflective materials that provide greater conspicuity at night and give drivers more awareness of the crossing itself, signs, or devices at the crossing or the presence of a train. Devices that the author has investigated through personal contact are discussed.

Conrail Device

The Consolidated Rail Corporation (Conrail) has formed a joint labor-management group to investigate safety at railway-highway grade crossings. One effort of the group has been development of a modified crossbuck. The Conrail crossbuck consists of a three-panel, retroreflective and reflecting device to be installed on the post below the crossbuck sign (Conrail shield) and used in conjunction with various new crossbuck designs consisting of standardized, 90° blades with standard wording, but of different color combinations. The latest design consists of red letters on a silver background, all of high-intensity retroreflective materials with a 2-in. strip of silver material on the back of each blade.

The Conrail shield has a triangular, three-dimensional configuration. It has alternating stripes of red and silver with 2-in. mirror strips between the red and silver (Figure 1). The retroreflective material is "activated" by automobile headlights on the approach or by the headlight of an approaching locomotive. The mirror strips also reflect light from the train towards the approaching driver. Because of these latter two attributes, its developers refer to it as a self-triggering device (M. Joyce, unpublished data).

The Conrail crossbuck is being tested in Ohio and Kansas (K. Hinshaw and J. M. Molitoris, unpublished data). Conrail's goal is to amend the standard crossbuck nationwide (at passive grade crossings). This will require its acceptance in the *Manual of Uniform Traffic Control Devices* (MUTCD), a process that would start with a successful product evaluation and demonstration project.

Ohio Studies

Areas of concern that will be studied or demonstrated in Ohio include driver response to the Conrail crossbuck and behavior testing, data evaluation, and cost-benefit methodology. The Ohio Department of Transportation (ODOT) is evaluating the Conrail crossbuck, along with other low-cost, innovative devices at passive crossings. They intend to promote a corridor project across northern Ohio. The objectives of the study are as follows:

1. To verify that the existing crossbuck may or may not be operating at its intended level of effectiveness as a passive crossing control device in both daytime and nighttime conditions,
2. To verify that the Conrail crossbuck has the visual impact (e.g., visibility and reflectivity) it is intended to have relative to the existing crossbuck,
3. To analyze and document familiar and unfamiliar driver behavior as it is affected by the existing crossbuck compared to the Conrail crossbuck,

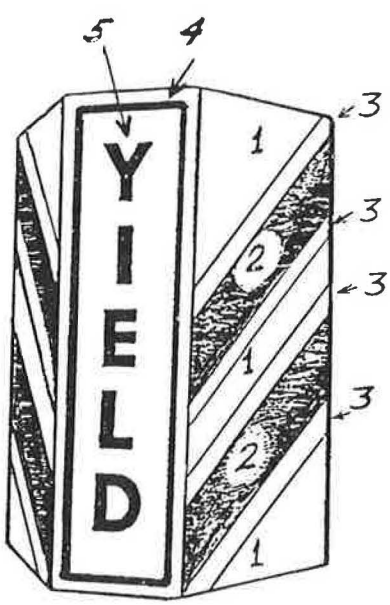

FIGURE 1 Conrail shield.

Dimensions:

Front Panel, 9" x 38"
Side Panels, 12" x 38"
Yield Panel, 8" x 37"
Red Strips, 4 1/4" x 16 1/2"
Mirror Strips, 1 1/2" x 16 1/2"

Key:

1. Silver
2. Red
3. Mirror strips (total 15% of surface)
4. Red border (1/2")
5. Red letters on silver background
   letter height = 6"

4. To evaluate and document familiar and unfamiliar driver behavior relative to both the standard and Conrail crossbuck under both daytime and nighttime conditions,
5. To compare and document visibility of the standard crossbuck versus the Conrail crossbuck under daytime and nighttime conditions with a train approaching and traveling through the crossing,
6. To evaluate and document the process, costs, and benefits of amending the MUTCD and implementing the Conrail crossbuck, and
7. To consider and comment on alternative modifications to the crossbuck or other passive crossing control signs or devices and compare effectiveness, costs, and benefits.

The anticipated benefits of the Ohio study include the following:

1. An improved safety environment at railroad-highway intersections,
2. A nationwide reduction in crashes, injuries, and fatalities at railroad-highway intersections, and
3. A relatively low-cost, high-benefit grade crossing improvement.

Kansas Demonstration Project

The FRA worked with two universities (Kansas State University and the University of Kansas), three major railroads (the Atchison, Topeka, and Santa Fe Railway Company, the Burlington Northern Railroad, and the Union Pacific Railroad), and the Kansas Department of Transportation in developing support, a funding package, and a contract for demonstrations at six grade crossings, two on each of the three railroads.

Six rural grade crossings were selected to demonstrate four specific low-cost warning devices plus two combinations of the four. The six systems were matched up with the six grade crossings. The six systems are listed in Table 1. Figure 2 shows a schematic sketch of the installation of the Conrail device with roadside delineators. The research and evaluation plan described here will be used for each grade crossing system.

The variables to be recorded and analyzed in the field are as follows:

1. Speed of approaching vehicles will be taken at three points on the approach—one in the approach and two near the beginning and end of the nonrecovery zone on each approach. A standard radar gun will be used by a hidden or inconspicuous observer.
2. Deceleration rates will be calculated from the speeds and time to traverse the zone.
3. The point of brake light applications will be observed on each approach.
4. Head movements made by drivers in the approach zone will be observed and recorded during daylight hours.
5. Traffic counts will be recorded by hand during the observation periods.

Three observation periods will be conducted: one before and two after periods. The first after study will be about 2 months after installation of the experimental devices and the second after study will be approximately 6 months later. Each observation period will last approximately 1 week. Data will be collected on 3 weekdays. Each day will be divided into a day (light) period and night (dark) period. The day period will be from 2:30 p.m. to 5:30 p.m. (or when the setting sun causes a problem, if prior to 5:30 p.m.). The night period will be from 9:00 p.m. to midnight. The 2:30–5:30 p.m. period corresponds to rural, peak traffic flow. There is practically no rural traffic after midnight.

After the study has been completed, data will be analyzed for any statistically significant changes. Pictorial history (i.e., slides, video) and subjective analysis of the demonstration will also be included in the analysis.

The final step will be to restore the innovative devices and restore the grade crossing to standard condition in conformance with the MUTCD.

TABLE 1 Devices To Be Used in the Six Studies, as Proposed

Study No.	Description
1	Modified Canadian Crossbuck (not approved; an additional study No. 3 was substituted)
2	YIELD sign (with YIELD AHEAD)
3	Roadside delineators on approaches and delineators or retroreflecting material on crossbuck post
4	Conrail Shield
5	Combination of studies Nos. 3 & 4 Conrail Shield and roadside delineators on approaches and delineators or retroreflecting materials on crossbuck post
6	Combination of Studies Nos. 2, 3 & 4 Conrail Shield, YIELD sign (with YIELD AHEAD), roadside delineators or retroreflecting material on approach and retroreflecting material on crossbuck post

Retroreflective Trackside Objects

The Arizona Corporation Commission expressed interest in alternatives to costly lighting systems at crossings in suburban areas.

The primary concept of the devices used in Arizona was that night visibility can be enhanced by a high-intensity, retroreflective material. In addition, the reflective material provides flicker as a train passes between drivers' headlights and the high-intensity, retroreflective devices on the far side of the track. The Arizona Department of Transportation (ADOT) received FHWA permission to install these devices at 15 grade crossings paid for with regular grade crossing funds (M. Mariscal, unpublished data).

ADOT conducted tests of retroreflective material on all four quadrants of three grade crossings. Poles were 4 in. × 48 in. with round tops and retroreflective panels on each side. Retroreflective tape was put on the crossbucks. Subjective analysis of this test (i.e., field observation and video) indicated the devices would be somewhat effective (M. Mariscal, unpublished data).

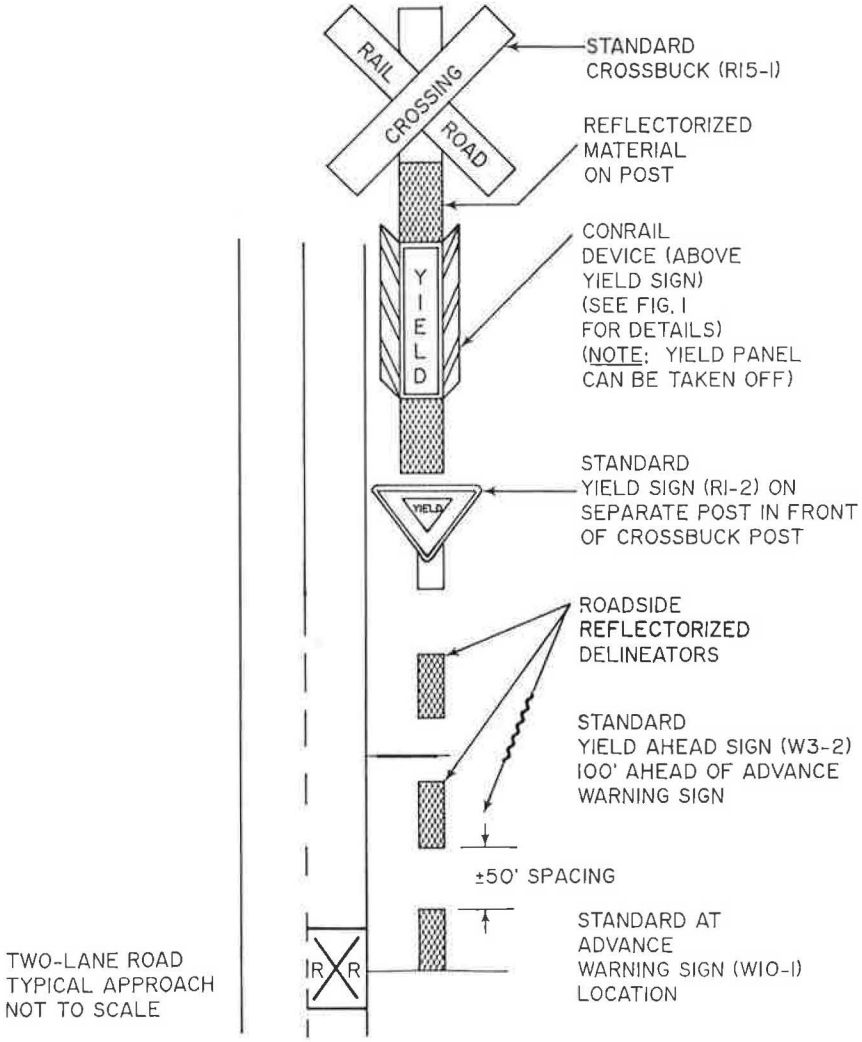

FIGURE 2 Schematic of Conrail shield, delineators on approach, and reflectorized crossbuck post (from Study 5).

A second test was conducted in April 1989 at Fort Rock Road, west of Seligman. Poles were 8 by 48 in. with two 8-in. and 48-in. retroreflectorized panels. Additional 8-by-48-in. panels were installed on the backs of the crossbucks. Subjective analysis by field observation and video indicated this device was very effective (M. Mariscal, unpublished data).

In spring 1991, FHWA approved the use of these retroreflective devices at 16 locations on mainline crossings. The railroads will install the devices at their cost, estimated at $280 per crossing, with FHWA paying 90 percent (M. Mariscal, unpublished data).

During these latest tests, ADOT personnel will spend one day stopping drivers to see what they think of the crossing devices.

Diamond-Grade Retroreflective Material

3M Corporation is promoting the use of diamond-grade, retroreflective material on standard crossbucks and posts. This diamond-grade material is being used in Nebraska at selected grade crossings and will be used at grade crossings in the state of Minnesota (E. Tompkins, unpublished data). Diamond-grade material has about three times the reflective rating of engineering-grade material and about two times the reflective rating of high-intensity, retroreflective material. It should increase the visibility of crossbucks at night. When used on the far side of the tracks, it should enhance the flicker effect when compared to engineering-grade retroreflective material. The state of Texas has recently made a decision to use diamond-grade retroreflective material at grade crossings (H. Richards, unpublished data).

3M/BN Passive Warning Sign

Burlington Northern (BN) Railroad has been involved in a research and development project with 3M to develop what they call a "passive warning sign." The warning sign appears to light up when a train approaches a crossing because the light from the locomotive headlight is captured by the sign and then redirected 90° outward toward the oncoming vehic-

ular traffic (Figure 3). The patented optical system designed by 3M uses no mirrors, but rather thin-film Fresnel lenses to control the light (J. LeVere, unpublished data).

The face plate of the sign that the motoring public sees may contain any message or color. A three-quarter size model with a red "X" front panel was demonstrated at the Kansas DOT Annual Railroad Conference (J. LeVere, unpublished data). The proposed size of the sign would be 60 in. high by 12 in. wide by 12 in. deep. It would be mounted on the crossbuck post.

The passive warning sign would be most effective at night but less effective at dawn and dusk. It would have little or no special effectiveness during the day. However, it could be designed with an effective, nonilluminated, daytime warning message.

Solar-Powered Illumination

Recent advances in technology allow illumination by photovoltaic (solar) panels and batteries, and activation by telescopic switches that can sense the headlight of an approaching train (J. LeVere, unpublished data). The illumination is train-activated; that is, the telescopic switches activate light, 50-watt, quartz-halogen flood lamps, four mounted on wooden poles on each side of the track. The lights remain on for 10 min, operated by battery power, then shut off until activated again by the next train.

The experimental system BN is testing (at a grade crossing on Vermilion Road near Longmont, Colorado, since November 5, 1990) consists of the following main subsystems (J. LeVere, unpublished data):

- Photovoltaic modules and charge controllers,
- Maintenance-free lead acid batteries,
- Eight 50-watt quartz-halogen flood lamps mounted on wooden poles, and
- Telescopic switches and relay control system.

The solar panels charge a 24-volt bank of batteries on each pole. The system will provide about 2 hr of continuous operation. If trains take longer than 10 min to clear the crossing, the time will have to be adjusted.

Other Low-Cost Retroreflective Devices

An FHWA memorandum summarized research that involved the use of existing, low-cost retroreflective devices at a nonsignalized crossing to focus the driver's attention when a train is present (*17*). A number of devices had been installed at rural grade crossings and evaluated at night with a train trav-

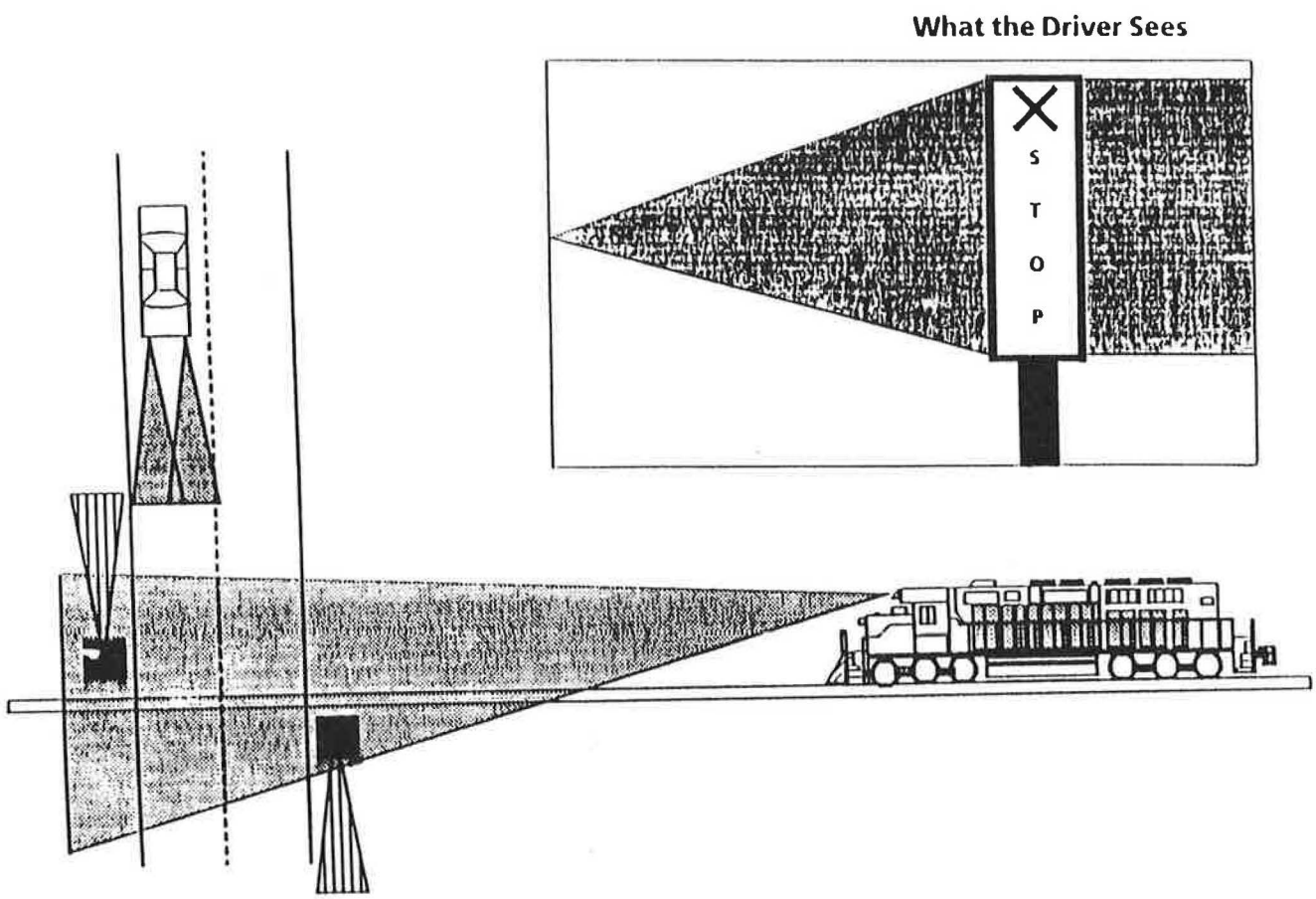

FIGURE 3 Schematic of the BN/3M passive warning sign (Source: BN Systems).

eling through the crossing. A list of these devices is shown in Table 2. Based on a subjective evaluation by the contractor, two of the devices were the most promising: multiple retroreflectorized panels installed in all four quadrants of the grade crossing and retroreflective material installed on the backs of crossbucks (*17*).

The test devices were installed at three Vermont Railroad Company crossings in Vermont on both the near side and far side of the crossings. The near side devices were intended to help draw attention to the crossbuck and the crossing; the far side devices were supposed to create flicker as the rail cars and wheels passed in front of them and thus to accent train movement.

The subjective evaluation of effectiveness was done by noting the following:

- Degree of greater visibility,
- Distance from the crossing at which the devices were visible,
- Durability,
- Vandal-resistance, and
- Installation cost.

Both types of devices tested were visible at 200 to 300 ft in advance of the crossing. Experience with the off-the-shelf, retroreflective panels indicates that they are both durable and relatively inexpensive (*17*).

The FHWA memorandum includes some guidelines on the use of these retroreflective devices:

> When improving the visibility of the railroad crossing, high efficiency retroreflective material should be used on the backs of crossbucks and sign supports and on both side of panels. Panels on two-way roads should use white/silver retroreflective material, while panels on one-way roads should have yellow retroreflective material on the left side and white/silver on the right side. Whenever white is specified as a sign color, it includes silver-colored retroreflective coatings or elements that retroreflect white light. (*17*)

Variable-Aspect Signs

The principle of a variable-aspect sign (VAS) is that, although it is a stationary, passive sign, when used in pairs of panels, they appear to move or alternate between dark and light in opposite phases as an approaching driver's angle of view changes (C. J. T. Young, unpublished data). Possible changes include changes in message, changes in colors, changes in shape (such as an arrow that alternates between two lengths and appears to "jab" in one direction). They can be designed as "enhancers" to increase the effectiveness of existing signs by adding panels that appear to flash as the driver approaches. The design of the VAS that gives the moving or flashing effect is described below:

> The dynamic effect of the VAS is produced by placing a special image surface behind an assembly of parallel, cylindrical lenses. The VAS is constructed in two parts, with a front facing lens system and a backplate which contains the image surface of the sign. An intervening air space separates the lens structure from the image surface. The lens system is made up of clear plastic strips molded and polished to allow high display resolution and quick reversal of two images. The image surface is constructed to allow easy design and substitution of word and symbol messages in any combination of figure-ground colors. VAS can use any colors that can be used in other signs, and they can include retro-reflective surfaces. Its construction is related to an arrangement that has been used for viewing stereoscopic pictures, but it has been further developed and much extended by Outlook Engineering Corporation to be applicable to highway signs. (C. J. T. Young, unpublished data)

One use of VAS that has been suggested is at a rail-highway grade crossing. Possible displays include a pair of "blinking" panels added to the highway advance warning sign and a VAS

TABLE 2 Devices Installed and Evaluated (*17*)

The devices installed and evaluated included the following:

1. Three-inch retroreflector buttons attached to highway delineator posts.
2. Retroreflective material installed on the back of crossbucks and their support posts. The retroreflective material consisted of 4-inch wide, encapsulated lens sheeting tape.
3. A 3-inch by 9-inch aluminum plate covered with an encapsulated lens sheeting attached to a highway delineator post. Multiple 4-inch by 6-inch plates mounted on delineator posts were also evaluated.
4. [a] A multiple retroreflectorized panel installed in all four quadrants of the railroad crossing. The posts were covered with encapsulated lens sheeting on both sides.
5. Combinations of retroreflector buttons and rectangular aluminum retroreflectorized plates.
6. Standard pavement markers.
7. Retroreflector buttons attached to the cross ties.

[a]System number 4 consisted of impactable, curved delineator panels, 3 inches wide x 4 feet high, placed to optimize visibility between boxcars and train wheels. The concave side faced the near side of the approach; the convex side faced the far side of the approach. No fewer than three of these panels were used at a grade crossing.

crossbuck with blades that alternate between black and white (C. J. T. Young, unpublished data).

ADDITIONAL STUDIES IN PROGRESS

Texas

The objective of a Texas study is to enhance the effectiveness of current MUTCD standard crossbuck and advance warning signs (passive devices) at railroad-highway grade crossings (D. Fambro, unpublished data). The improvements to the standard passive warning devices are intended to improve safety and effectiveness while being relatively low-cost (i.e., less expensive than active warning devices) (*18*).

To accomplish the objectives the Texas researchers will

- Conduct a state-of-the-art review of current and past studies and practices,
- Conduct a workshop with Texas grade-crossing experts to select candidate warning devices or improvements to test, and
- Test the candidate warning devices or improvements in both controlled and real-world environments (*18*).

The candidate devices initially identified are as follows:

- Stop and Yield signs,
- Crossbucks with double blades and backblades,
- The Canadian crossbuck,
- The Conrail device,
- The BN/3M device,
- Various reflective devices,
- Illumination, and
- Flashing lights (other than standard, active flashers).

The candidate devices were selected by a panel of experts from the State Department of Highways and Public Transportation (SDHPT), Texas Transportation Institute (TTI), and FRA. Delphi techniques were used to identify the candidate devices as well as to select attributes of controlled, convenient sites and to identify parameters to be studied in a human factors evaluation.

Human factors experts will use focus groups to develop a questionnaire about scenes showing the new devices. This part of the study will narrow the candidate list. Those devices that the focus groups determine to be the most effective will be installed in the field at several locations throughout Texas. It is expected that up to 200 crossings may have the new or improved systems installed.

Two systems will receive priority in the Texas studies: the standard crossbuck with system improvements and the modified Canadian crossbuck (*18*).

Canada

In Canada, a study will look at the effectiveness of having an intermediate warning sign located between the advance warning sign and the crossbuck. The intermediate sign would give a driver additional information about the crossing that should contribute to taking the proper action.

The Canadian Ministry of Transportation is conducting the study of passive grade crossings to examine the following (E. Duran, unpublished data):

- The differences between daytime and nighttime driver behavior,
- The effect at grade crossings of restricted sightlines that result from sight restrictions in any quadrant, and
- The effectiveness of a new signing system for passive grade crossings.

The new passive signing system will include a series of intermediate warning signs between the advance warning sign and the crossing. For example, an approach might have a series of three intermediate signs: Restricted Visibility, Be Prepared to Stop, and Stop Before Crossing (E. Duran, unpublished data).

The sites will be low-volume roads with at least one quadrant with restricted sight distance, good sight distance down the tracks in both directions for a stopped driver, and conditions that minimize the chance of creating a hazard to drivers (E. Duran, unpublished data).

The data collected will be vehicle approach speed, brake light application, and a video recording of driver action. These data will be collected before and after improvements are made.

The results of the study are expected to identify driver behavior at night, demonstrate the importance of clearing sightlines at grade crossings, and determine the behavioral effectiveness of new intermediate advisory or warning signs (E. Duran, unpublished data).

FHWA Turner Fairbank

A human factors-type lab study is being conducted at the FHWA Turner Fairbank research facility. Subjects are being shown slides or photographs of various crossbuck modifications in traditional research tests used by research psychologists. These tests are designed to test the impact that a particular sign has on a subject, particularly the sign's conspicuity or ability to get the subject's attention and the subject's understanding of the sign (B. Alicandri, unpublished data).

Determining the conspicuity and driver's understanding of the modified Canadian crossbuck are two aspects of the study. All signs and modifications being studied are generated by a special computer graphics package from which a slide or a photograph is produced.

Minnesota

The governor of Minnesota recently signed into law an act relating to the transportation needs of the state that included a provision to conduct a study of railroad-highway grade crossing safety improvements in the state. The study will include the following:

1. A method of determining the relative benefits of grade-crossing warning and improvements to the railroad, to the road authority, and to the public, and cost guidelines;
2. Funding sources for grade-crossing warning and improvements;

3. Grade-crossing safety research needs;
4. Recommendations for statutory changes to improve grade-crossing safety;
5. The adequacy of existing and proposed methods of grade-crossing safety, including train visibility, signal and warning device design, a public reporting system for malfunctioning warning devices, improved systems of crossing warnings, and recommendations for additional funds for rail crossing safety education; and
6. Methods for establishing statewide priorities for grade-crossing safety and for implementing these priorities (*19*).

Iowa

A study of the effect of an Operation Lifesaver media blitz is being conducted in Iowa (K. Brewer, unpublished data). Although this is not a study of an innovative device, the importance of education should not be overlooked when looking for low-cost improvements in grade-crossing safety.

The objective of the research is as follows:

> . . . to observe and record driver behavior at a selected sample of railroad highway grade crossings before an Operation Lifesaver campaign, and to estimate the significance of any change in driver behavior that may be attributed to the conduct of the Operation Lifesaver campaign. (*19*)

The study will concentrate on a set of grade crossings that includes a different level of traffic warning devices such as gates, signals and crossbucks, and advance warning signs. The locations chosen represent a mix of land use types and different railroad environments.

During the "pre-blitz" (before) study, driver action at and approaching grade crossings was observed for four 4-hr periods. The following variables were recorded:

- Approach speed,
- Crossing speed,
- Driver attention ("looking behavior"),
- Obedience or disobedience to any traffic warnings, and
- Time elapsed between the automobile crossing the track and the train's arrival (if a train was on the approach during the driver's approach) (*19*).

The Operation Lifesaver media blitz included a grade-crossing investigation course for the police department, more than eight different school presentations, and several business/industry presentations. Posters were hung at business locations and information was put in employee pay envelopes.

Community awareness media activities included a radio station's newsmaker show, public service announcements in the local newspaper, 16 billboard locations, a display at a police chiefs convention, a public education display at the local mall, and a contest at the mall.

After data will be taken at the crossings immediately following the media blitz. It is anticipated that if Operation Lifesaver is an effective public information campaign, one or more of the variables measured will show positive changes from the precampaign results to the postcampaign results. If improvements are recorded in driver behavior, another follow-up field study will be performed 4 to 5 months later to determine if any long-term effects occurred in driver improvements.

CONCLUSIONS

This author offers the following personal conclusions. Two major problems exist today at passive crossings. First is identification of the crossing both in regard to its existence and recognition that it is a passive crossing, particularly at night. Second is full understanding of the crossing's significance (i.e., that full responsibility for safe passage over a passive crossing rests with the driver). To deal with these problems, this author believes that greater conspicuity is needed, particularly at night, but that greater conspicuity itself needs to be associated with something unique that will come to be associated with passive crossings. Greater emphasis is also needed on education of drivers about their responsibilities at passive crossings. Finally, additional information is needed to assist drivers to carry out their responsibilities safely.

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Methodology To Analyze Driver Decision Environment During Signal Change Intervals: Application of Fuzzy Set Theory

SHINYA KIKUCHI AND JEFFREY R. RIEGNER

During a signal change interval, most drivers must decide whether to stop or go based on uncertain information on speed, the remaining yellow time, and distance from the intersection. The decision process under fuzzy information, such as this case, is suited for analysis by fuzzy set theory. Fuzzy set theory and its logic has been used to analyze the driver's decision between "stop" and "clear" during the signal change interval. Fuzzy sets of "not safe stopping" and "not safe clearing" are defined along the approach roadway. Depending on the way the two sets intersect, the dilemma, indecision, or option zones can be represented. Possibility and necessity measures of "safe stopping" and "safe clearing" were defined, and they were assumed to represent the decision criteria of aggressive and conservative drivers, respectively. Based on these measures, a new approach to determine signal change intervals is suggested. It states that an interval should be such that, at any point along the approach, the possibility of "safe stopping" or "safe clearing" are always one, and the necessity measure of the two sets should exceed a certain minimum value.

The signal change interval has been a topic of traffic engineering studies since its formulation by Gazis et al. (*1*) in the early 1960s. Traditionally, this interval is computed to eliminate the dilemma zone. The dilemma zone is known to exist if the signal change interval is such that the driver cannot stop or go without violating the traffic rules. If a driver is in the dilemma zone at the onset of the yellow phase and decides to proceed, the vehicle will still be in the intersection when the all-red period ends. Conversely, if the driver decides to stop, the vehicle will not be able to stop without entering the intersection. In the traditional formula, signal change intervals are computed based on the idea that, at any point along the approach, at least one of the actions (stop or clear) is possible.

If all the characteristics of the approaching vehicles and drivers are the same as the assumed input values, and if the driver knows his exact location at the time of signal change, the signal change intervals set by the traditional formula should be adequate to allow time for the driver to complete one of the actions. Hence, at the end of a signal change interval, the intersection should be cleared for crossing traffic.

However, because the information available to a driver concerning location, the amount of yellow time remaining, approach speed, and other parameters is fuzzy, the result is driver indecision when the yellow indication appears. Furthermore, because not all drivers are approaching at the same speed and their decision criteria are different, some drivers always experience a certain degree of indecision of dilemma. Thus, the traditional formula is not applicable for the analysis of the decision process of individual drivers. Rather, it is suited for diagnostic analysis of the signal timing as stated by Mahalel and Prashker (*2*).

In this paper, we recognize that some of the information available to the driver is fuzzy; in other words, the values known to the driver are only approximate. We then attempt to analyze the decision-making environment using the logic of fuzzy set theory. Two basic fuzzy sets are defined: one representing the minimum distance from which stopping is possible and the other the maximum distance from which the "clearing" maneuver is possible. Both distances are measured from the intersection stop line. Knowing these fuzzy distances, we determine the possibility and necessity measures of completing the stopping and clearing maneuvers safely at any point along the approach roadway. The possibility and necessity measures are believed to represent the decisions of two types of drivers: aggressive (or risk-taking) and conservative (or risk-averting), respectively. We then introduce a fuzzy set that represents the approximate location as perceived by a driver and evaluate the driver's likelihood of completing each of the actions using the possibility and necessity measures. The level of the driver's anxiety and the zones where the dilemma and indecision occur can also be illustrated using the intersection of the membership functions of these fuzzy sets.

Analyses similar to the authors' are found in prior papers dealing with the stopping probability function. Among them are Olson and Rothery (*3*) and Sheffi and Mahmassani (*4*). The probability function represents the observed frequency of stopping decisions along the approach. It shows the final action of the drivers in an aggregated form, but it does not differentiate the circumstances in which individual drivers made the choice. At a given location, the same value of stopping probability may be obtained when the driver is in the dilemma zone or in the option zone. The existence of the probability function itself, however, shows that every driver's judgment and decision pattern is different and that each reacts differently to the given information. The previous work has helped develop our idea for using fuzzy sets to analyze the effect of fuzziness of information on individual drivers' decisions of stopping and clearing. Another work related to this study is one by May (*5*), in which extensive field measurements of driver behavior and risks were compiled and analyzed. What we propose as possibility and necessity measures in

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this paper are perhaps related to May's observation of risk measurements.

The authors suggest two criteria in determining signal change intervals: one, the possibilities of both the stopping or going actions at any point along the approach should be one; two, the necessity measures of stopping and clearing should be greater than a given minimum level at any point along the approach roadway. Further, it is suggested that the use of possibility and necessity measures provides the theoretical basis for explaining the regions of dilemma, indecision, and option.

TERMINOLOGY AND CONVENTION OF SYMBOLS

The following terminology is used to indicate the condition of the driver at the onset of the yellow indication: dilemma, indecision, option, or imperative.

Dilemma is the situation in which the driver can complete neither the stopping nor clearing action safely. Two types of dilemma can exist. Type 1 dilemma is the situation in which the driver can perform neither action at all. Type 2 dilemma is the situation in which the driver can perform one or both of the actions but with difficulty. The dilemma zone is the area in which the driver faces a dilemma of either type.

Indecision is the situation in which the driver can complete one of the actions safely and the other action with some level of difficulty.

Option is the situation in which the driver can complete both actions safely.

Imperative is the condition in which the driver must choose either stopping or clearing; if the driver is far from the intersection, he or she must stop, and if the driver is very close to the intersection, he or she must clear it.

A bold letter indicates a fuzzy number set. The following are symbols of sets and parameters:

1. **SD:** fuzzy stopping distance (distance from the intersection) when the approach speed is fuzzy;
2. **CD:** fuzzy clearing distance (distance from the intersection) when approach speed (V) and the remaining signal change interval (t) are fuzzy;
3. **V:** fuzzy set of approximate speed;
4. **S:** fuzzy set of safe stopping distances;
5. **C:** fuzzy set of safe clearing distances;
6. **NS:** fuzzy set of distances from which the vehicle is not able to stop safely;
7. **NC:** fuzzy set of distances from which the vehicle is not able to clear safely;
8. **L:** fuzzy set of the location of the driver at the onset of the yellow indication;
9. **t:** fuzzy signal interval perceived by the driver; and
10. **E:** universal set.

All distances are measured from the intersection stop line.

TRADITIONAL MODEL OF SIGNAL CHANGE INTERVALS

The traditional model of signal change intervals is based on equating two distances that are both measured from the intersection stop line: one to stop (stopping distance, D_s), and the other to clear the intersection during the signal change interval (clearing distance D_g). Finding the signal change interval by equating the two distances means that, at any point along the approach roadway, either the stopping or the clearing action can be completed within the signal change interval.

The stopping distance and the clearing distance are, respectively,

$$D_s = V^2/2b + Vd \tag{1}$$

and

$$D_g = Vt - (w + \ell) + a(t - d)^2/2 \tag{2}$$

where

d = driver perception and reaction time,
V = approach speed,
b = deceleration rate,
w = intersection width,
ℓ = vehicle length, and
a = acceleration rate.

If it is assumed that the vehicle will not accelerate upon seeing the yellow light,

$$t = d + (V/2b) + (w + \ell)/V \tag{3}$$

If it is assumed that the vehicle will accelerate upon seeing the yellow light,

$$t = d + [-V + \sqrt{V^2 + 2a(V^2/2b + w + \ell)}]/a \tag{4}$$

Equation 3 is the most commonly quoted expression, and it is recommended by ITE (*6*). [A recently recommended alternative from the ITE Technical Council includes the impact of grade (*7*).]

If t in Equation 3 is set at $D_g > D_s$, both the stopping and going actions are possible at a particular speed. If, on the other hand, t is set at $D_s > D_g$, a dilemma zone is said to exist, because for section $(D_s - D_g)$ neither the stopping nor clearing action is possible.

ANALYSIS USING FUZZY SET THEORY

The use of Equation 3 (or 4) assumes that all the parameters are known to the driver as "crisp" values and that the driver makes the correct decision depending on location. In reality, the driver's understanding of the situation is not clear. He can only approximate the following:

- The remaining amber and all-red time,
- The location of the vehicle relative to the boundary between the stop zone and clearing zone (relative to D_s and D_g),
- The speed of the vehicle,
- The vehicle's acceleration and deceleration capabilities, and
- The width of the intersection (if the driver is not familiar with it).

The length of the driver's perception/reaction time depends on how precise the information is to him or her. An interesting analysis of driver perception/reaction time using a computer simulation model is presented by Chan and Liao (*8*). The model allows the analyst to test driver reaction while watching a vehicle approaching the intersection on the screen.

The quantities of the parameters above are normally judged by the driver based on his or her experience, intuition, and familiarity with the intersection and signal. Furthermore, the driver can control the values of some of the parameters (for example, the braking and acceleration force to be applied). Thus, the values of the variables are not completely random; rather, they are subjectively judged numbers, given a myriad of factors such as the driver's personality, physical condition, vehicle characteristics, the environment and geometric design of the intersection approach, relationship to other vehicles in the traffic stream, and so forth. Yet, in many cases, after stopping or clearing, the driver still wonders if he or she has made the best decision. Detailed discussions of how some of the parameters influence the driver's decision are presented in the work of Cheng et al. (*9*). In this paper, we treat these parameters as fuzzy, and introduce fuzzy set theory to analyze the decision-making process.

Analysis Procedure

The parameters considered fuzzy here are those perceived by drivers. Among them, we assume the following three parameters as fuzzy quantities: the approach speed, the remaining signal change interval, and the driver's location at the time of signal change. These three parameters are selected only to simplify the presentation; fuzziness of other parameters can be incorporated without compromising the generality.

When the driver's knowledge of speed (**V**) and signal change interval (**t**) is approximate, the stopping distance and the clearing distance can be defined as fuzzy quantities with fuzzy sets called "stopping distance" (**SD**) and "clearing distance" (**CD**), respectively.

Next, we determine a set that represents the distances greater than **SD** and call it a set of safe stopping distances. Similarly, we determine a set that represents the distances smaller than **CD** and call it a set of safe clearing distances. If the vehicle is within the safe stopping distance, it can stop safely; if it is within the safe clearing distance, it can clear the intersection safely.

Determining whether a driver can stop or clear safely from a particular distance involves comparing the distance with the fuzzy sets of safe stopping distance and safe clearing distance. A comparison of a crisp number with a fuzzy number requires the introduction of possibility and necessity measures, because the term greater or smaller (than a fuzzy number) can only be stated by a degree. In this case, the possibility measure represents the optimistic judgment and the necessity measure represents the pessimistic judgment when comparing two numbers.

First we develop possibility distributions of "safe stopping" and "safe clearing" with respect to the distance from the intersection. These define the approach area where a stopping maneuver is possible and the area where a clearing maneuver is possible, respectively. Second, we define the necessity measures of the two sets again in terms of the distance from the intersection. Thus, these possibility and necessity measures can represent the judgment of aggressive and conservative drivers, respectively, because of their criteria for comparing two numbers. It must be noted that the types of drivers represent a state of mind; thus, the same driver can be in an aggressive or conservative state depending on the circumstances.

Because the driver's knowledge of a location is also fuzzy, we compute how much the driver's approximate location belongs to each of the sets ("safe stopping" and "safe clearing"). As a result, we should be able to measure the possibility and the necessity of completing each action based on the approximate information.

Stopping Maneuver

Fuzzy Set of Stopping Distance

Given an approximate approach speed (**V**) in Equation 1, the approximate stopping distance, **SD**, is computed, where **SD** is a fuzzy set. The membership function **SD** is denoted as $h_{SD}(x)$. A possible shape of $h_{SD}(x)$ is shown in Figure 1a. This function shows the degree that a given distance x belongs to the set "stopping distance." In other words, it shows the possibility distribution of stopping distance when the approach speed is approximately V.

Possibility of Safe Stopping

The possibility of stopping safely from a distance x is determined by comparing x with **SD**. If x is greater than **SD**, the vehicle can stop safely. The set of numbers which is "possibly" greater than **SD** is called "safe stopping distance," and it is denoted by **S**. Because **SD** is a fuzzy number, the distance greater than **SD** must also form a fuzzy set. The membership function of set **S** is

$$h_S(x) = \max_{x \geq z} h_{SD}(z) \tag{5}$$

or

$$h_S(x) = \begin{cases} h_{SD}(x) & x_1 \leq x \\ 1 & x > x_1 \end{cases}$$

where x_1 is the value of distance where $h_{SD}(x)$ becomes 1.

This membership indicates the possibility that the vehicle can stop safely from distance x. Thus, it is denoted

$$\text{Poss}\ (x \in \mathbf{S}) = h_S(x) \tag{6}$$

The shape of $h_S(x)$ is similar to a cumulative distribution of $h_{SD}(x)$ as seen by the solid line in Figure 1b. It shows the degree (in numbers between 0 and 1) that the vehicle can stop safely along the approach. For a distance close to the intersection, the possibility is 0, whereas for a distance much farther from the intersection, it is 1. This possibility measure should represent the judgment of an aggressive, risk-taking, or optimistic driver because it accounts for any evidence that may indicate that his location is greater than **SD**. For the

definition of possibility measure, refer to Klir and Folger (*10*) and Zimmermann (*11*).

Necessity of Safe Stopping

The necessity measure of safe stopping distance (a degree of "necessarily greater" than **SD**) is derived from

$$\text{Nec}(x \in \mathbf{S}) = 1 - \text{Poss}(x \in \mathbf{NS}) \tag{7}$$

where **NS** is the complement of **S**; thus, Nec($x \in$ **S**) is 1 minus the possibility of "not being able to stop safely" (or a set of "unsafe stopping" distances) from distance x. Because Poss($x \in$ **NS**) can be derived from the membership function that represents a number possibly less than **SD** (as shown by the dashed line in Figure 1b), Nec($x \in$ **S**) is derived by Equation 7 and is shown in Figure 1c.

In contrast to Poss($x \in$ **S**), it can represent the judgment of a conservative or risk-averting driver because it takes only the sure evidence to justify that x is less than **SD**. For an explanation of necessity measures, refer to Klir and Folger (*10*).

From location p on the approach, for example, a risk-averting driver may not feel it is safe to stop, whereas a risk-taking driver may feel it is safe to stop, as seen by the comparison of values of the possibility and necessity measures of safe stopping in Figure 1c. The fact that the necessity distribution of safe stopping is located to the right of the possibility distribution in Figure 1c indicates that a risk-taking driver would feel the need to stop before the risk-averting driver as each approaches the intersection. The difference between the values of possibility and necessity measures originates from the lack of accurate information available to the driver. If sufficient information is available, and if the driver is normative, the possibility and necessity measures should be equal and the decision becomes crisp as would result from the traditional equation.

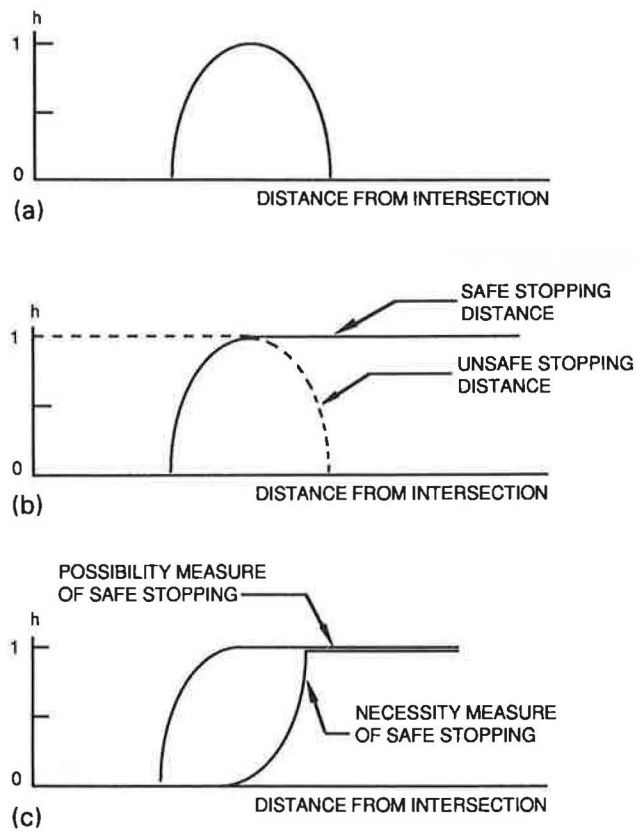


FIGURE 1 Stopping maneuvers considered: (*a*) fuzzy set representing stopping distance, (*b*) possibility distribution of safe stopping distance and unsafe stopping distance, and (*c*) possibility and necessity distributions of safe stopping distance.

Clearing Maneuver

Fuzzy Set of Clearing Distance

Given the approximate values of approach speed and signal change interval, the fuzzy clearing distance is derived from Equation 2 and is denoted **CD**. The membership function of the set is denoted $h_{\mathbf{CD}}(x)$ and its hypothetical shape is shown in Figure 2a.

Possibility of Safe Clearing

The possibility of clearing the intersection from distance x is determined by examining whether x is smaller than **CD**. The membership function of the set of numbers that is possibly smaller than **CD** is defined by

$$h_{\mathbf{C}}(x) = \max_{x \le z} h_{\mathbf{CD}}(z)$$

or

$$h_{\mathbf{C}}(x) = \begin{cases} 1 & x \le x_1 \\ h_{\mathbf{CD}}(x) & x > x_1 \end{cases} \tag{8}$$

where x_1 is the value of x where $h_{\mathbf{CD}}(x)$ takes the maximum value (which is 1), and **C** denotes the set of clearing distances.

The possibility of clearing safely from distance x is now presented by the possibility measure and shown by the dashed line in Figure 2b:

$$\text{Poss}(x \in \mathbf{C}) = h_{\mathbf{C}}(x) \tag{9}$$

Similar to the case of the stopping maneuver, this function is believed to represent the judgment of a risk-taking or optimistic driver.

Necessity of Safe Clearing

The corresponding necessity is derived from its definition and shown in Figure 2c:

$$\text{Nec}(x \in \mathbf{C}) = 1 - \text{Poss}(x \in \mathbf{NC}) \tag{10}$$

where Poss($x \in$ **NC**) represents the possibility of "not able to clear" from distance x, in other words, the possibility that

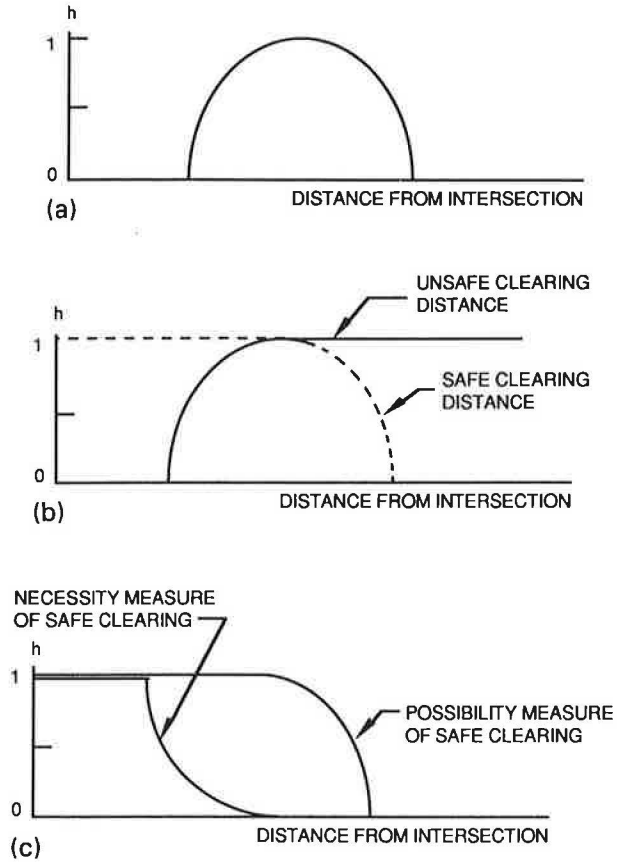


FIGURE 2 Clearing maneuvers considered: (*a*) fuzzy set representing clearing distance, (*b*) possibility distribution of safe clearing distance and unsafe clearing distance, and (*c*) possibility and necessity distributions of safe clearing distance.

x is greater than **CD**, which is shown by the solid line in Figure 2b.

Again, the necessity measure is believed to represent the judgment of a conservative or risk-averting driver. The fact that the possibility distribution of safe clearing extends to the right of the necessity distribution in Figure 2c indicates that a risk-taking driver perceives that it is safe to clear before the risk-averting driver does as each approaches the intersection.

Dilemma Zone, Indecision Zone, and Option Zone

The dilemma, indecision, and option zones can be illustrated by the way in which **S** and **C** intersect. The zones are defined based on the intersection of Poss($x \in$ **S**) and Poss($x \in$ **C**), and also on Nec($x \in$ **S**) and Nec($x \in$ **C**), separately. Respectively, they represent the decision-making environment for risk-taking and risk-averting drivers.

Based on Possibility Measure (for Risk-Taking Drivers)

When the possibility measures of safe stopping and clearing are superimposed, the possible patterns of overlaps are shown in Figures 3a, 3b, and 3c. In Figure 3a, there is a section (D_1) where neither safe stopping nor safe clearing is possible. This section is the Type 1 dilemma zone. In Figures 3a and 3b, there are zones (D_2) where the possibilities of both safe stopping and safe clearing are less than 1. These zones correspond to the Type 2 dilemma. Also in Figure 3b, at I, one of the actions is possible but the other is not completely possible. This zone corresponds to the indecision zone. In Figure 3c, both the safe stopping and clearing actions are possible in Section O. This is the option zone. The option zone, however, is located between indecision zones.

Based on Necessity Measure (for Risk-Averting Drivers)

Similarly, the intersection of two necessity measures of safe stopping and clearing are shown in Figures 3d, 3e, and 3f. Similar to the cases of Figures 3a, 3b, and 3c, the dilemma, indecision, and option zones of risk-averting drivers can be identified using the necessity measures.

It is clearly seen by comparing Figures 3a and 3d that the total area of dilemma is greater when the necessity measures are used to describe it. This indicates that risk-averting drivers would experience a greater level of uncertainty than the risk-taking drivers. When the information is assumed to be crisp, as in the traditional signal change interval formula, the value of the measure changes from 0 to 1 abruptly. Thus, the zones of indecision and Type 2 dilemma could not be identified for the two types of drivers.

Degrees of Dilemma

In the Type 2 dilemma zone, the degree of dilemma the driver experiences can be expressed by the intersections of the sets "cannot safely stop" (**NS**) and "cannot safely clear" (**NC**). **NS** and **NC** are the complements of **S** and **C**. The degrees of dilemma based on possibility and necessity measures, respectively, may be described by the height of the intersection of Poss($x \in$ **NS**) and Poss($x \in$ **NC**), or Nec($x \in$ **NS**) and Nec($x \in$ **NC**).

Fuzziness of Vehicle Location and Its Impact on Dilemma, Indecision, and Option

Next, we incorporate the fact that the driver's knowledge of his location is usually fuzzy at the onset of the yellow indication. The fuzzy set of this location is denoted by a membership function $h_L(x)$. This function represents a fuzzy set that states "the driver's location is approximately L feet from the intersection." Given $h_L(x)$, the state of the driver's decision process can be examined from the intersections of Poss($x \in$ **S**) and Poss($x \in$ **C**) with $h_L(x)$. The intersection indicates the degree that the approximate distance L belongs to the safe stopping set or the safe clearing set.

The possibility and necessity measures of safe stopping and safe clearing from approximate distance L are computed as

$$\text{Poss}(L \in \mathbf{S}) = \max\{\min(h_L(x), \text{Poss}(x \in \mathbf{S}))\} \quad \text{for all } x \qquad (11)$$

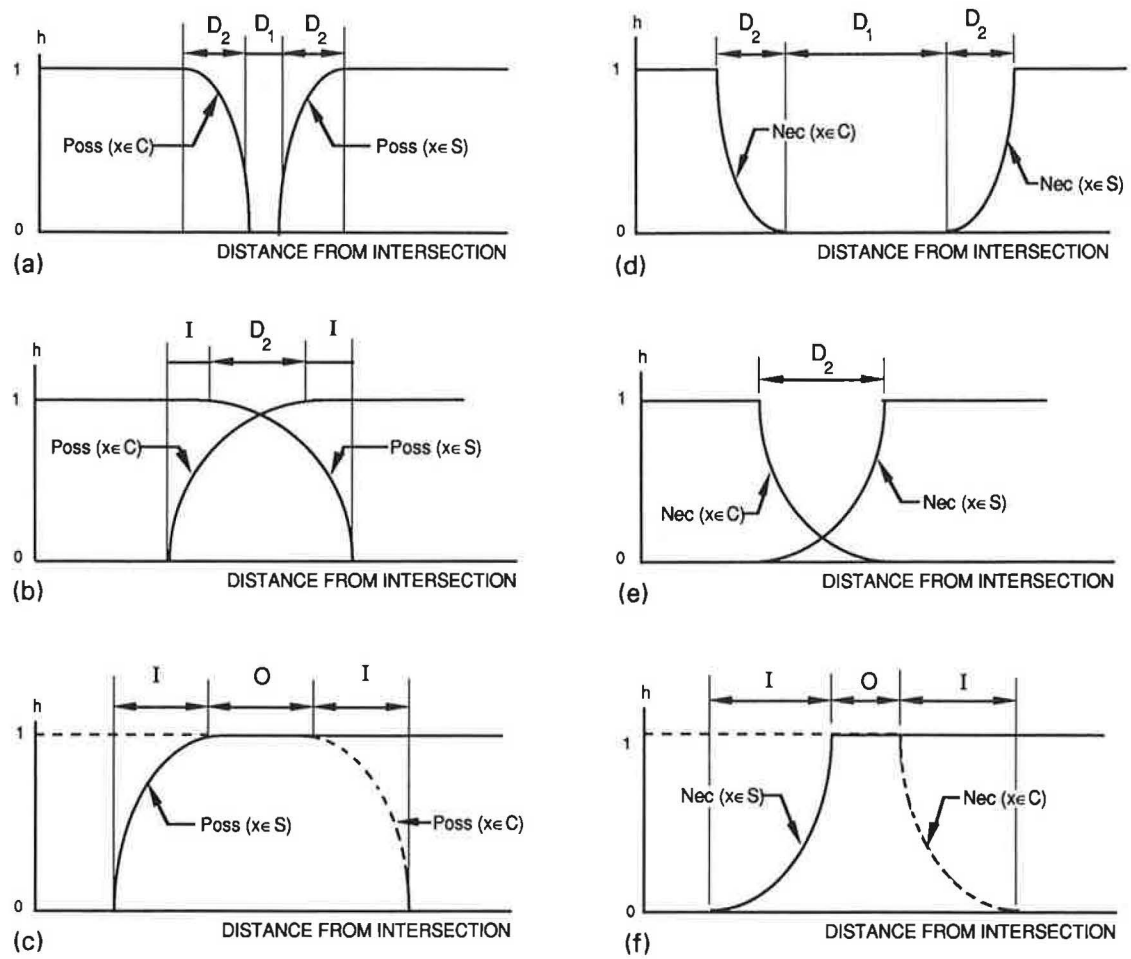


FIGURE 3 Dilemma, option, and indecision zones considered: (*a*) Type 1 and 2 dilemma zones based on possibility distribution, (*b*) Type 2 dilemma zone based on possibility distribution, (*c*) option and indecision zones based on necessity distribution, (*d*) Type 1 and 2 dilemma zones based on necessity distribution, (*e*) Type 2 dilemma zone based on necessity distribution, and (*f*) option and indecision zones based on necessity distributions.

$$\text{Nec}(L \in \mathbf{S}) = 1 - \text{Poss}(L \in \mathbf{NS}) \tag{12}$$

$$\text{Poss}(L \in \mathbf{C}) = \max\{\min(h_L(x), \text{Poss}\ (x \in \mathbf{C}))\} \quad \text{for all } x \tag{13}$$

$$\text{Nec}(L \in \mathbf{C}) = 1 - \text{Poss}(L \in \mathbf{NC}) \tag{14}$$

For an approximate distance L, the possibilities of safe stopping and safe clearing according to Equations 11 and 13 are illustrated in Figures 4a and 4b. They are given by the heights of a and b, respectively, in the figures.

Criteria for Determining Signal Change Intervals

Determination of signal change intervals should account for the fuzziness of perceived values of the parameters and the difference in decision criteria of different types of drivers. The following criteria may be suggested to determine the signal change intervals:

1. The possibility of taking at least one action safely must be guaranteed at any point x along the approach:

$$\text{Min}\ (\text{Poss}(x \in \mathbf{S}), \text{Poss}(x \in \mathbf{C})) = 1 \tag{15}$$

This criterion accounts for safe completion of actions by aggressive drivers.

2. The necessity measures of taking one of the two actions must be greater than a given level α at any point along the approach:

$$\text{Min}\ (\text{Nec}(x \in \mathbf{S}), \text{Nec}(x \in \mathbf{C})) \geq \alpha \tag{16}$$

This criterion guarantees the minimum level of safe completion of action. It is an attempt to offer a certainty level that a risk-averting driver can take at least one of the actions safely.

The signal change interval derived from the traditional formula satisfies the first criterion so that at least one of the actions is possible along the approach. It does not allow for the maximum certainty of safe stopping or safe clearing when measured on the basis of necessity.

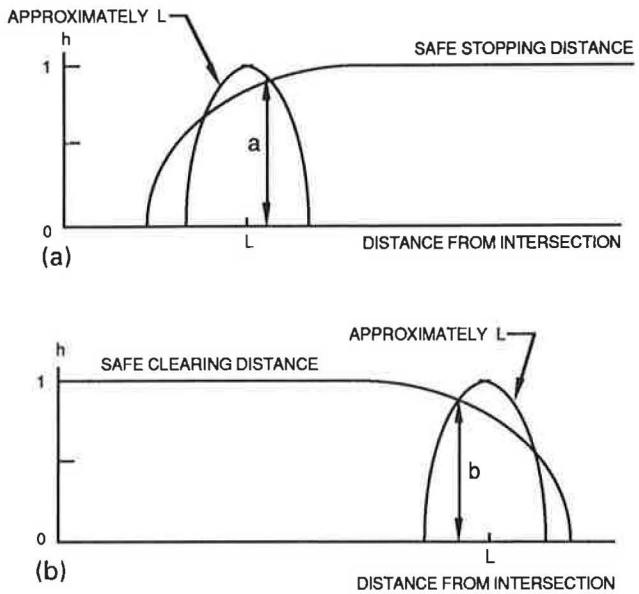


FIGURE 4 Possibility measures for safe stopping and clearing from a distance of approximately *L*: (*a*) possibility of safe stopping and (*b*) possibility of safe clearing.

EXAMPLE ANALYSIS

In this section, using a set of example values, we present two procedures: one to identify the dilemma and indecision zones and a second to develop signal change intervals based on the method just presented.

The following values are used for the examples: approach speed (V) = 40 mph, deceleration rate (b) = 9 ft/sec^2, intersection width (w) = 50 ft, vehicle length (ℓ) = 20 ft, and perception/reaction time (d) = 1.5 sec. The signal change interval computed with the traditional formula Equation 3 is 5.3 sec.

Dilemma and Indecision Zones

We now analyze the dilemma and indecision zones for the same intersection, assuming that the perceived approach speed and signal change interval are approximately 40 mph and approximately 5 sec, respectively. The fuzzy sets for the approach speed (V) and perceived signal change interval (t) are defined using a triangular fuzzy number (TFN) of the form (f1, f2, f3), which respectively represent minimum, most likely, and maximum values for the approximate number. "Approximately 40 mph" can be represented by (30, 40, 50) mph and the "approximately 5 sec" by (4, 5, 6) sec. This assumption of a TFN for approximately 40 mph is reasonable when compared to the observed distribution of approach speed presented by Olson and Rothery (*3*). Introducing these TFNs for **V** and **t**, we compute the fuzzy sets for "stopping distance" and "clearing distance" as **SD** = (174, 282, 406) ft and **CD** = (138, 282, 447) ft using Equations 1 and 3. These fuzzy sets are shown in Figures 5a and 5b. For the arithmetic operations of the fuzzy numbers, refer to Kaufmann and Gupta (*12*). The corresponding possibility and necessity measures of "safe stopping" and "safe clearing" are computed in Equations 5, 7, 8, and 10 and shown in Figures 5c and 5d.

The indecision and dilemma zones are presented in Figures 5e and 5f. Figure 5e shows that the possibility measure of at least one of the actions is 1 along the approach; thus, at least one action is possible. This is expected, because the values of f2 for **SD** and **CD** are the same as the original crisp values. The necessity measures of the two sets, illustrated in Figure 5f, show Type 2 dilemma zones. The value of the necessity measure is less than 1. These two figures indicate that, for a 5-sec signal change interval, no dilemma exists for risk-taking drivers, but a Type 2 dilemma exists for risk-averting drivers.

The length and the location of the indecision zone or Type 2 dilemma zone in Figures 5e and 5f are compared with the observed and derived stopping probabilities presented by Olson and Rothery (*3*), Sheffi and Mahmassani (*4*), and Zegeer and Deen (*13*). Our example shows that the Type 2 dilemma zone lies between 138 and 447 ft from the intersection as seen in Figure 5f. The observations reported by Olson and Rothery (*3*) show the range in which stopping probability is between 0 and 1 as 200 to 380 ft at approach speed 50 mph, and 80 to 200 ft at 30 mph speed; Zegeer and Deen (*12*) show 100 to 300 ft at approach speed 40 mph; Sheffi and Mahmassani's (*4*) derived probability shows approximately 60 to 350 ft at 40 mph.

Figure 6 compares their stopping probability functions with our possibility and necessity measures of "safe stopping" and "safe clearing." The shaded area is bounded by the possibility of "safe stopping" and the necessity of "not safe to clear," the former representing the aggressive driver's stopping criterion and the latter the conservative driver's clearing criterion. Lines 1 and 2 represent the stopping probability curves shown by Sheffi and Mahmassani (*4*) and Zegeer and Deen (*13*), respectively. Lines 3 and 4 and Lines 5 and 6 are the observed stopping probability frequencies for approach speed of approximately 30 mph and 50 mph, respectively, reported by Olson and Rothery (*3*). The shaded area is very close to Lines 1 and 2, and it also lies between line pairs of 30 mph approach speed and 50 mph. Line 7 shows the necessity measure of safe stopping—a risk-averting driver's stopping criterion. The characteristics of the intersections presented in the previous papers are probably not identical. However, the lines derived by the fuzzy measure closely match with the ones that were surveyed or mathematically derived previously. This suggests that the fuzzy measures can be an alternative method to identify the zones of dilemma and indecision, and to examine the adequacy of the signal change interval.

Signal Change Interval

The authors next used the criteria presented earlier to suggest a signal change interval that accounts for the driver's fuzzy perception of V and t. Because the first criterion is satisfied by the 5.3 sec, the interval computed at the beginning of this section, the signal change interval that satisfies the second criterion is computed. This is accomplished by determining t so that the slope of the decreasing section of Nec($x \in$ **C**) intersects with Nec($x \in$ **S**) in Figure 7 at a height greater than α. In other words, the following condition must be satisfied at the intersection of the Nec($x \in$ **C**) and Nec($x \in$ **S**) lines:

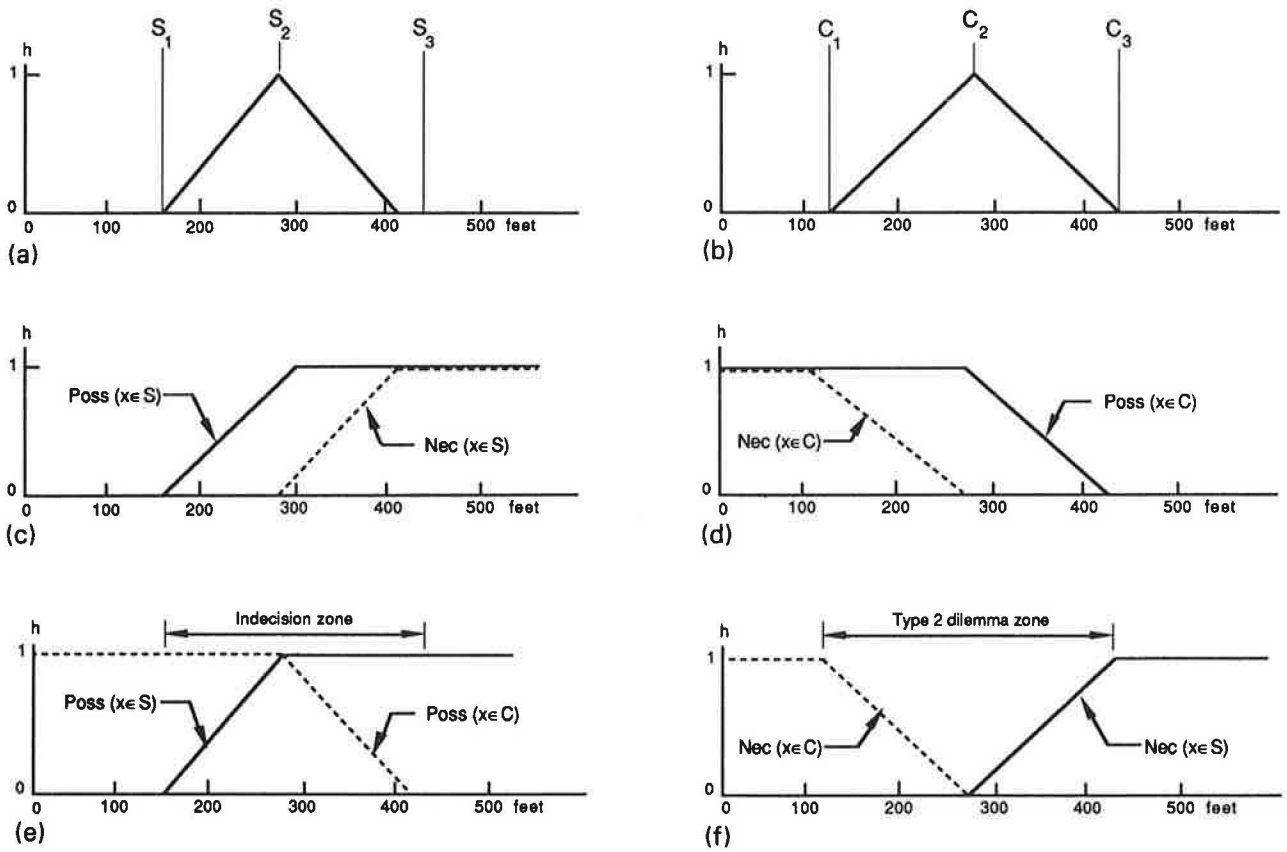
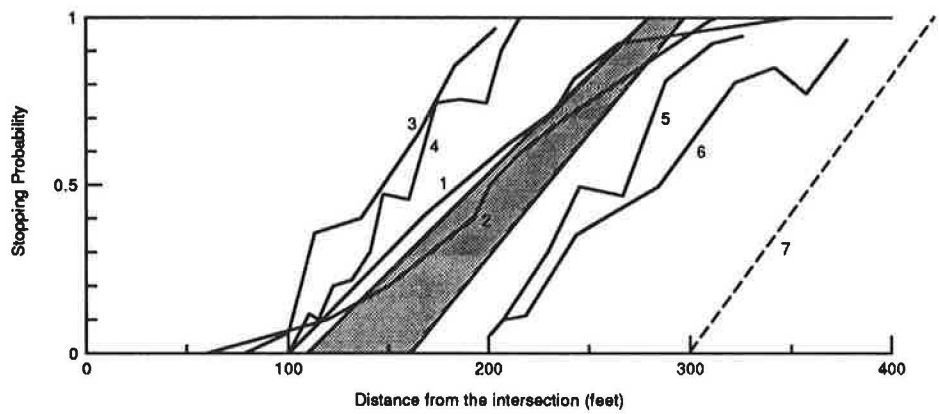


FIGURE 5 Fuzzy sets and possibility and necessity measures for example problem: (*a*) fuzzy stopping distance (SD), (*b*) fuzzy clearing distance (CD), (*c*) safe stopping distance (S), (*d*) safe clearing distance (C), (*e*) indecision zone, and (*f*) Type 2 dilemma zone.



Notes — Shaded area: likelihood of stopping derived from the fuzzy measures Nec (x∈NC) and Poss (x∈S) at approach speed = 40 mph;

Line 1: Stopping probability (approach speed = 40 mph) by Zegeer and Deen (13);

Line 2: Stopping probability (approach speed = 40 mph) by Sheffi and Mahmassani (4);

Lines 3 and 4: Stopping probability (approach speed = 30 mph) by Olson and Rothery (3);

Lines 5 and 6: Stopping probability (approach speed = 50 mph) by Olson and Rothery (3);

Line 7: Necessity measure of stopping (Nec (x∈S) at approach speed = 40 mph.

FIGURE 6 Comparison of stopping probabilities and likelihood of stopping derived from fuzzy measures.

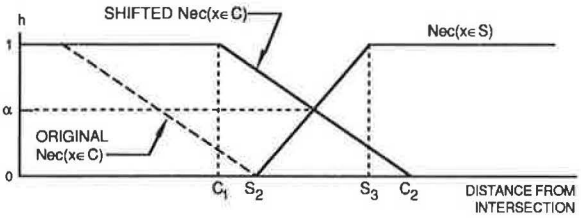


FIGURE 7 Determination of C_1 and C_2 for given α.

$$(1 - \alpha)C_2 + \alpha C_1 = (1 - \alpha)S_2 + \alpha S_3 \quad (17)$$

where C_1, C_2, S_2, and S_3 are as shown in Figure 7. Because C_1 and C_2 are functions of t, once they are determined by Equation 17, t can be obtained.

For an α value of 0.5, for example, t becomes 7.8 sec. This is 2.5 sec longer than the value computed by Equation 3. However, the 7.8 sec of yellow and all-red time for the given condition is still within the range of maximum signal change interval demand surveyed by Lin and Vijaykumar (*14*) for similar conditions.

CONCLUSIONS

Recognizing the fact that drivers must decide whether to stop or clear based on fuzzy information, the authors propose the use of fuzzy set theory for the analysis of the driver's decision-making environment at signalized intersections.

Using fuzzy measures of the safe stopping and safe clearing sets, the dilemma, indecision, and option zones are defined along the approach. The possibility and necessity measures of the two sets identify these zones for risk-taking and risk-averting drivers, respectively. The intersection of the complements of the two sets identifies the level of dilemma for the two types of drivers. Criteria for setting signal change intervals are also suggested when the information available to the driver on speed, location, and the remaining time of the signal change interval are vague.

The difference between the possibility and necessity measures narrows as more accurate information becomes available to the driver. Eventually, if the information is totally crisp to the driver, the two measures coincide. Under this environment, the traditional equation for the signal change interval is justified.

Providing accurate information to the driver reduces dilemma and indecision and, therefore, helps to reduce the signal change interval. Any measures resulting in more accurate driver-perceived information is essential to reduce driver indecision and shorten the signal change interval.

The stopping probability functions studied by many in the past represent only the consequences of decisions made. The possibility and necessity measures that we propose, on the other hand, can indicate the availability of the choices and the degree of safety for completing them for two extreme types of drivers (risk-taking and risk-averting). The normative driver's behavior perhaps lies between the two extreme types of drivers; thus the proposed method can identify the ranges of the dilemma and indecision zones. The approach presented here could be extended to the analysis of other risk-measurement problems in traffic engineering.

ACKNOWLEDGMENT

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Development of a Headlight Measurement System

M. RYS, E. RUSSELL, D. HARWOOD, AND A. RYS

The development, validation, and testing of a headlight measurement system (HMS) are described. The HMS hardware and the software developed for the system are described. The field study consisted of collecting data at six key visual points in the driving task. A complete, detailed manual developed on the setup and use of the HMS is discussed. The equipment setup and operating instructions, the calibration procedure, and recommendations for future system modification to improve the HMS are discussed. It was concluded that although more expensive sensors, in the $6,000 to $10,000 range, could be custom made and incorporated into a more sophisticated (but costly) system, the system developed proved to be adequate for the main purpose of determining distribution of light output of the current fleet of highway vehicles at any set of up to seven desired points quickly, easily, and with reasonable accuracy with relatively inexpensive, off-the-shelf hardware. Two companion manuals were also developed during the study: *Light Measurement System Manual* and *Installation and Operation Manual*.

A headlight measurement system (HMS) has been developed that can be used in field studies to sample illumination levels simultaneously from approaching vehicle headlights at a maximum of seven locations and at any number of points on the approach, or continuously, dependent upon the capacity of the portable computer incorporated into the system. The original study objectives included obtaining an extensive data base of headlight illumination levels of the fleet of vehicles currently in use on U.S. highways. Development of equipment needs, design of equipment hardware and software, testing and debugging in both lab and field situations were much more complex and time consuming than anticipated. The main thrust of the project evolved into the development and testing of the best system possible with off-the-shelf components that could be purchased and assembled at relatively modest cost.

Government regulation of vehicle headlight design has historically been based on measurements of headlight output under laboratory conditions. Bench measurement of headlight photometrics can be quite accurate but fails to account for many of the factors that influence the light output in the real world. Among the factors not always accounted for are variations in headlight mounting height, improper aiming, voltage variations in the vehicle electrical system, dirty or corroded headlight lenses and reflectors, vehicle pitch angle because of the bouncing of the vehicle body on its suspension system, and vehicle yaw angle as determined by driver steering control. All of these factors can influence headlight performance and real-world light levels and distribution as affected by these factors can only be determined in the field. These factors are dynamic variables that can only be sampled at various points in time. Thus, field measurements of existing, real-world headlight levels are needed to determine light levels and distributions of the vehicle fleet at any point in time. Also, good field values can be used as the basis for realistic performance standards for headlights. However, no system was readily available that would measure and record accurate headlight output throughout the range of values expected.

The HMS was developed in this study to provide NHTSA with a tool to set up a program to measure light levels from vehicle headlights in the field over their full range of expected values. The details of the development and operation of such a system should be of interest to anyone contemplating collecting data of this nature. Developing such a system at reasonable cost with no guidelines to begin with was not an easy task. However, the knowledge gained in developing the system, along with the equipment and operating manuals and the details in the final report, should be valuable information for anyone contemplating building such a system.

DESCRIPTION OF HEADLIGHT MEASUREMENT SYSTEM

System Overview

The HMS is an integrated set of light meters and computer equipment that measures the light output of automobile headlights at preselected positions. The HMS is designed to measure light levels from headlights of moving vehicles on the highway with no disruption to traffic flow. The equipment can also be used for light-level measurements of stationary vehicles or sources under field, garage, or laboratory conditions.

Light levels can be measured at any location where the light meters can be placed. An infrared triggering device can be used to record light meter readings when a vehicle is in position at specific distances from the sensors. Light levels measured by the sensors are recorded in a computer data base on a portable microcomputer. The HMS is portable and can be moved from site to site with a required setup time of approximately 2 hours. Operation of the system requires a trained crew of two or three persons.

The following section describes the major components of the HMS. A complete list of all components and supplies needed for field use of the system is presented in Table 1.

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TABLE 1 Required Equipment and Supplies for Field Studies with the HMS

Microcomputer - Roadrunner by Micro Express, LCD screen, portable
Backup diskettes containing DOS version 3.3, Quiklite program
Blank diskettes to store copies of files containing field data
Junction box - black box (approx. 5-in x 8-in x 2-in) with 11 connection ports on top
Cable to connect junction box to microcomputer - gray insulation, 6-ft long with multipin connectors at each end; computer end has a second wire with single BNC coaxial connector
Power supply inverter for microcomputer - Power inverter Model PV-400, Triplite brand from Triple Mfg.; Backup is Triplite Model SB-200a
Power supply batteries for microcomputer - two deep-cycle marine batteries
Battery recharger - Sportsmaster 10-amp charger for deep-cycle batteries
Cables to connect the batteries to transformer and microcomputer
Infrared sensors - Banner Multi-Beam by Eng. Corp. Model SbEX transmitter, Model SBRX1 receiver
Adjustable brackets to attach infrared sensors to mounting posts
Batteries for infrared transmitter - Eagle-Picher Carefree Magnum 12-volt, 4.6 ampere-hours
Posts for mounting infrared sensors - 4 ft. gray, U-shaped steel posts with regularly spaced holes
Cables to connect infrared receivers to microcomputer junction box - gray 3-wire insulated cables, 250 ft long (2)
Light meters - Minolta Illuminance Meter T-1 (2 meters currently available)
Translator boxes to convert and amplify analog signal from light meters - Wilderson Instrument Co. Inc., Model TW8100 Two-wire TX
Wires to connect data translators and Minolta meters - 3 gray 4-ft wires with red and blue connectors at one end, single connector at other end
Cables to connect translators to microcomputer junction box - black, RG-59 coaxial wire, 250-ft long (6 cables currently available)
9-volt batteries for light meters
Cap for light meter to use during calibration - black lens cover supplied with light meter
Mounting bracket for light meters - black sheet metal cover for attaching light meter to mounting pole with band clamps
Cable to connect sensor and electronics portions of light meters - 15 ft. grey cable; useful in obtaining measurements at pavement edge locations; available from Minolta but not purchased as part of this study; a cable of this type was borrowed for testing purposes
Mounting post for roadside sign sensor - two-part, gray round steel extension pole plus 3-ft. gray U-shaped steel channel to be driven into the ground, and band clamps to connect these two parts
Mounting hardware for overhead sign location - two-part assembly, consisting of (1) the base that clamps onto the bridge rail of the overpass; this base is made up of two flat metal plates connected by pipes with adjustable clamp screws at one end and (2) a two-part gray round extension pole that lowers the light meter below the bridge rail of the overpass
Support light meter for measuring glare of oncoming vehicles - A two-legged folding sign stand (such as is used to mount a temporary "MEN WORKING" sign) was used to mount this sensor. (This stand was borrowed and is not part of the equipment currently available with the HMS system.)
Sand bags - Sand bags were used to mount and protect the pavementedge sensors at a height of 6-in. above the pavement and to anchor the sign stand used for the oncoming vehicle location
Voltmeter - to confirm proper voltage levels for all batteries
Miscellaneous tools and supplies, including:
hand tools such as: screw driver, adjustable wrench, allen wrench, pliers, sledge hammer, tape measure - 100 ft. cloth, distance measuring wheel, spray paint or marking crayon, and duct tape

System Components

The HMS consists of five major components: the light meters, the infrared vehicle sensors, the mounting hardware, the portable microcomputer, and the microcomputer software. A block diagram of the HMS is shown in Figure 1. The function and operation of each component is described below.

Light Meters

Light levels are measured using commercial off-the-shelf battery-powered light meters, the Minolta Illuminance Meter, Model T-1. These meters may be mounted at any location where light levels are to be measured. The light meters are connected to the computer by cables. In the current system,

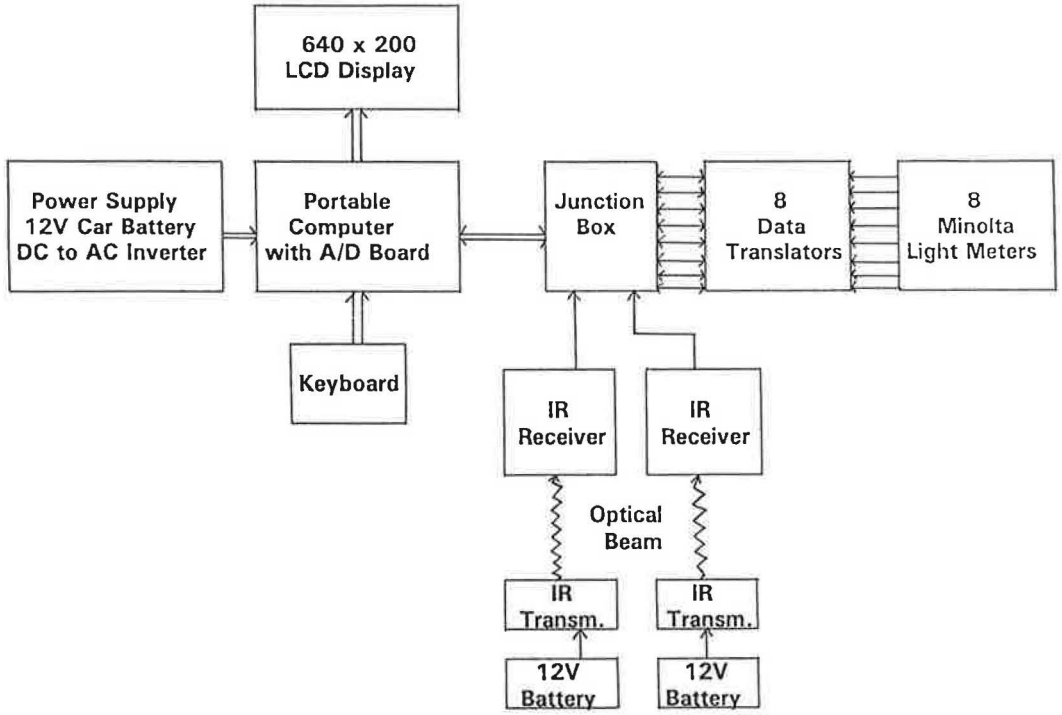

FIGURE 1 Block diagram of the headlight measurement system.

the maximum cable length that has been tested from light meter to computer is 250 ft (76.3 m).

The system was originally designed with less-expensive custom-made light sensors than the Minolta meters currently used. However, it was found that these less-expensive sensors were not capable of reading levels as low as 0.001 fcd and that their range was very limited when set to read their lowest light level, which was about 0.05 fcd. The possibility of using more sophisticated, custom-designed sensors was investigated. It was determined that they could be developed but their cost would be several thousand dollars each. It was concluded that the Minolta meter would be sufficient in terms of minimum reading, accuracy, and reliability and would thus be most cost-effective.

The Minolta meters selected for use with the system are capable of reading light levels in any of five ranges that can be manually selected by the user:

- 0.001 to 0.999 fc,
- 1.00 to 9.99 fc,
- 10.0 to 99.9 fc,
- 100.0 to 999.9 fc, and
- 1000.0 to 9999 fc.

Only the two lower ranges are normally needed for headlight field studies on highways at typical sign locations, because light levels above 10 fc are not likely to be measured on roadside or overhead locations.

The lowest light level the system can measure is 0.001 fc with ±0.001 fc accuracy (±100 percent) at the *lowest* level. All of the lab measurements indicate that the midpoint of each range has an accuracy better than ±2 percent *or* the least significant digit, whichever is greater. Except when pushing the system to record the lowest possible values, the range should be selected so that expected readings will be midrange. Accuracy is lowest at the extremes of each range. If a set of readings overlaps two ranges, or it is desired to read values spanning two or more ranges, two or more meters can be used, each set on a different range. Automatic ranging equipment was investigated, but it was concluded that the benefits did not justify the cost. Again, estimates obtained were for several thousand dollars.

The Minolta meters have a digital readout display on the meter itself, as well as an analog output port at which the reading in units of 0.001 fcd is represented by an analog voltage in millivolts. To avoid voltage drop, the analog voltage output stage of the Minolta meter has to be converted to, and transmitted as, a variable converter to the computer located some distance away. Therefore, an amplifier or translator device was installed in the line to convert the analog voltage in millivolts to an analog current in milliamps. This analog current is detected by the portable microcomputer and converted into a reading in foot-candles when a light level reading is taken. The translator device must be calibrated for the specific meter being used and the specific range (in foot-candles) in which it is being used. Thus, it is most convenient to have available in the field several translator devices that have been previously calibrated for each meter and each range in which each meter is likely to be used.

Each Minolta meter consists of two parts: a sensor and an electronics package. In normal operation, these two parts of the meter are plugged together into a single unit. However, the manufacturer can provide a cable approximately 15 ft (4.6 m) long so that the sensor and the electronics package can be placed in different locations. This was done when the sensor was placed near the pavement edge to avoid the loss of

the electronics package if the pavement edge sensor were run over by a vehicle. Laboratory tests found that the Minolta meters gave identical readings with and without the use of this cable connecting the two parts of the meter.

The HMS is designed to use a maximum of eight light meters at any one time. Each meter can be placed at a different location and each can operate within a specific user-selected range. Two meters were purchased with the system and are available for further use. An additional six meters should be purchased to take full advantage of the system capabilities.

Infrared Sensors

Two pairs of infrared sensors (each pair consisting of a transmitter and a receiver unit) are used to detect the presence of a vehicle and trigger the system to record light level readings. The corresponding transmitter and receiver units are mounted on posts on opposite sides of the road and aligned so that a beam is continuously being transmitted between them. When a vehicle breaks this beam, a signal is sent to the computer system. This signal consists of transistor-transistor (TTL) compatible voltages, which implies that a signal of about 5 volts might correspond to the uninterrupted light beam, and a zero volt signal would be received when the light beam is blocked.

The receiver unit is mounted on the same side of the road as the portable microcomputer and is powered through the cable that connects the receiver to the microcomputer. The transmitter unit is located on the opposite side of the road from the computer and is battery powered.

Because two pairs of infrared sensors are available, the system can be triggered to begin or end readings at two specific locations. The specific relationship between the infrared sensor triggers and the beginning or ending of data collection is determined by the computer software.

The infrared sensors are shown in Figure 2 and Figure 3.

Mounting Hardware

The HMS includes hardware for mounting the light meters and infrared sensors in appropriate positions. Mounting hardware for the light meters includes a pole for mounting a meter in a roadside sign location and a bracket and rod to mount a meter on a highway overpass in the position that would be occupied by an overhead sign. Hardware was also developed for mounting a light meter on a highway shoulder near the edge of the traveled way, 0.5 ft (0.15 m) off of the pavement surface. However, it was found that it was more satisfactory to tape it to a sandbag placed at the appropriate position. Sandbags provide additional protection to the light meter and reduce the possibility of it being struck by an errant vehicle and crushed. A photograph of mounting hardware for the right side sign location is shown in Figure 4 and a photograph of mounting hardware for the overhead sign location is shown in Figure 5.

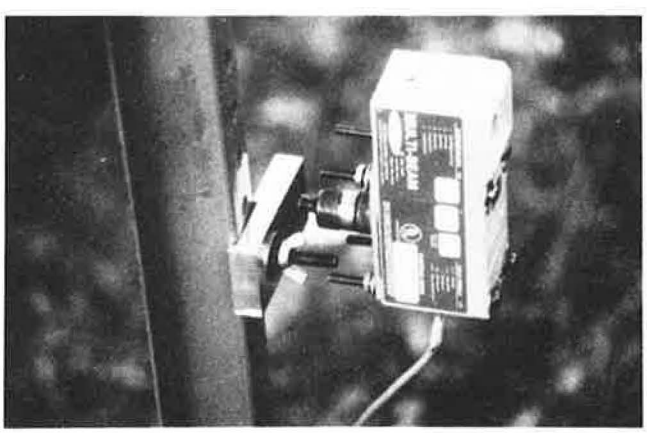

FIGURE 2 Close-up of one infrared transmitter unit.

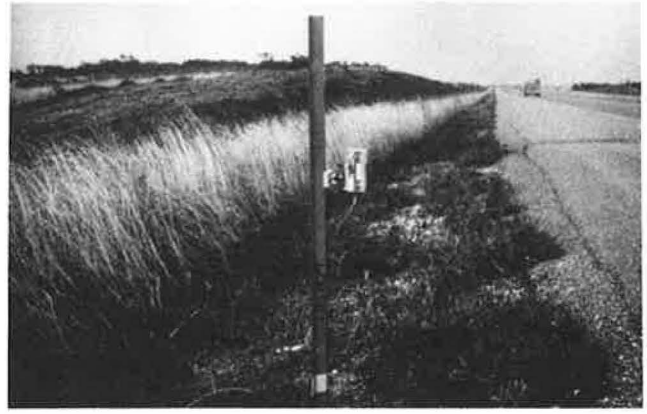

FIGURE 3 Infrared sensor in place at the freeway site.

Portable Microcomputer

The HMS is operated by a portable microcomputer. This computer is IBM-PC-compatible with 1 Mb of random access memory (RAM), a 20 Mb hard disk drive, and an 80286 processor. Although this computer was at the state of the art when purchased in 1988, faster processors such as the 80386 are now available at reasonable cost.

Up to eight light meters and two infrared vehicle sensors can be connected to the computer through a junction box. The computer contains an analog-digital conversion board that translates the analog signals received from the light meters into digital data that can be stored in a data base.

The HMS is controlled by a software program known as Quiklite that was written in the QuickBasic language specifically for use with this system. This program, whose operation is described in more detail in the next section, detects when the infrared sensors are triggered by a vehicle to begin or end a data collection interval, reads the light meters at specified times during the data collection interval, and stores the light meter readings in a data base. The HMS uses the PC-File+ data base format, and that data base software can be used to analyze the data after they are collected.

The complete list of hardware in Table 1 shows the nature of the system (i.e., composed of relatively simple, relatively inexpensive off-the-shelf components). Two manuals were developed during the study that detail the hardware and software

FIGURE 4 Mounting hardware for the right sign location.

FIGURE 5 Mounting hardware for the overhead sign location.

components and system operation: *Light Measurement System Manual* and *Installation and Operation Manual*.

The software and HMS operation are described in the next section.

Computer Software

Central to the HMS is the Quiklite program. This program, written in QuickBasic and compiled in an executable module stored under the file name QUIKLITE.EXE, can be run by entering the command QUIKLITE from the MS DOS prompt.

The program is controlled by the user from a main menu with a selection of seven options:

- Setup system,
- Display only,
- Record test,
- Review test records,
- 1702A information services,
- PC-File+, and
- Return to DOS.

Each of these options is explained below.

Option 1—Setup System Five fundamental parameters that control the operation of the Quiklite program must be set before headlight data can be recorded:

- Sample rate,
- Average base,
- Distances between Switches 1 and 2,
- Time between second switch and third set of readings, and
- Sensor ranges.

All five parameters can be changed at any time depending upon the data collection objectives, site geometry, equipment configuration, and so forth.

The sample rate is the number of readings per second made from each light meter while data are being collected. The software is capable of making from 1 to 4,000 readings per sec. However, it was found that the processor speed sets a practical upper limit on the combination of sample rate, average base, and distance between infrared sensors. Trials established that the most satisfactory value for the processor used was 500 readings per sec, which was the setting used for all data collected as part of this study. Newer portable computers could greatly increase the processing speed and number of readings per second and greatly enhance the system, as discussed in later sections.

The average base is the number of consecutive readings that are averaged to create each value stored in the data file. The average base used in this study was 50 readings (i.e., one average stored for each 50 readings made). Thus, the combination of a sampling rate of 500 readings per sec and an average base of 50 readings implies that each average light level stored in the data base is the average of 50 readings made over an interval of 0.1 sec.

The user must enter the distance (in feet) between the infrared sensors used to detect vehicle passages (called Switches 1 and 2 in the program).

The data collection interval for each vehicle passage selected for measurement of headlight levels begins when the first infrared sensor is triggered by the vehicle and ends at a specified time interval after the second sensor is triggered. The significance of this time interval is addressed further in the discussion of Option 3—record test.

The range selected for each of the eight light meters must be set in the Quiklite program and these ranges may be changed

at any time. These ranges must also be manually set on the light meter itself.

Option 2—Display Only The display mode allows continuous visual monitoring of the light levels being read on each sensor. The light levels are presented on the computer screen in both a digital and graphic format. This mode is useful for verifying that the infrared sensors and light meters are operating, determining whether the light meters are set to the appropriate ranges, and watching the measured light levels change in real time. No data can be stored in the data base in this mode.

Option 3—Record Test The record test mode is the option used to record light levels and store them in the data base. When a selected vehicle approaches and Option 3 is chosen, the system is "armed" so that readings will be "triggered" when the chosen vehicle passes through the infrared sensors. Thus, Option 3 should not be selected until the previous vehicle has passed the sensors.

Measurement of light levels begins as the vehicle trips the first sensor and continues until a specified time interval has passed after the vehicle trips the second sensor. When the vehicle has completely passed, the light level readings and the vehicle speed are displayed on the screen. The user then has the option to either save or delete the data in the data base.

The original concept for the HMS was that measurement and recording of light levels would be continuous from the time the vehicle tripped the first sensor until a specified time interval after the vehicle tripped the second sensor. Using the values of the initial setup parameters described earlier, it was intended that light levels from each meter would be recorded continuously every 0.1 sec during the data collection interval. Furthermore, it was intended that each recorded value would include the mean, variance, maximum, and minimum of specified number of readings (e.g., 50 readings). However, after the system was built, it was found during testing that the processing speed of the computer was not fast enough to permit this much data to be obtained and stored. Better computer technology available today (e.g., the 80386 processor) should allow the originally intended data collection procedures to be easily implemented.

An alternative scheme that would collect meaningful data within computer constraints was developed, and the available version of the Quiklite program uses this alternative scheme. Mean values of the reading from each light meter are recorded at three times during the data collection interval. These are the times when the vehicle trips the first infrared sensor (labeled in the data base as time T_1); the time when the vehicle trips the second infrared sensor (time T_2); and at the specified time interval (e.g., 1.1 sec) after the second infrared sensor is tripped (time T_3). The position of the vehicle at time T_3 can be estimated because the vehicle's speed between times T_1 and T_2 is calculated by the system.

Option 4—Review Test Records The review test records option allows the user to review data already recorded. Individual records or sequences of records can be recalled by their assigned record numbers, and the user can step forward or backward through records one at a time.

Option 5—1702A Information Services The 1702A information services option allows the user to scroll through an information file that contains a description of the HMS. This file is equivalent to a manual for use of the system.

Option 6—PC-File+ Data obtained with the PC-Plus system are stored in data files that can be accessed and manipulated with PC-File+. Selecting Option 6 accesses the PC-File+ data base manager and allows the user to sort and summarize data and generate reports.

Option 7—Return to DOS The microcomputer system uses version 3.3 of MS DOS as its operating system. Option 7 allows the user to leave the Quiklite program and return to DOS. From DOS, the user can reset the internal date and time functions, which are recorded in each record of the data base. Before the system is transported, the user should run

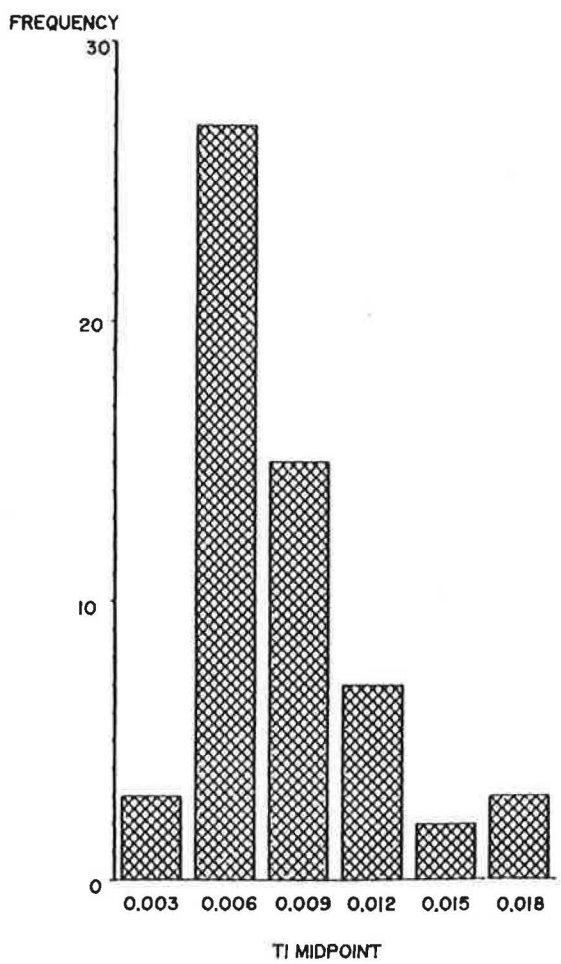

FIGURE 6 Histogram of illuminance for passenger cars at right overhead sensor location at T_1 distance.

the HDPARK command from DOS to park the hard disk drive. The procedure avoids damage to the hard disk during transportation.

RESULTS

A sample of the results obtained from data collected by the HMS is presented in Figure 6. These values and distributions are for passenger cars at right overhead sign location at T_1 distance.

These values are presented here primarily to illustrate the type of results possible with the system. The values obtained are considered to be reliable to the extent that small sample results are reliable. After the HMS was built and debugged, remaining funds were sufficient for only limited field testing. Thus the sample was very small relative to the size of the vehicle fleet in the United States. A set of results and discussion of a pilot field study at two highway locations using this HMS are described elsewhere in this Record by Rys et al.

CONCLUSIONS

An HMS for field studies of illumination levels from approaching vehicle headlights was developed. The system allows for simultaneous sampling of light levels at seven locations and at any number of points on the approach under control of a portable computer. The main thrust of the project was the design and testing of the best system possible with off-the-shelf components that could be purchased and assembled at relatively modest cost. Development of equipment needs, design of equipment hardware and software, testing and debugging in both lab and field situations were much more complex and time consuming than anticipated. The system described in this paper performed well during the pilot field studies. Several improvements in the system's flexibility, portability, sensitivity, and speed of data acquisition could be made by modifying the software and upgrading the computer hardware and light sensors at a modest increase in cost.

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Field Studies with a Headlight Measurement System

M. RYS, E. RUSSELL, D. HARWOOD, AND A. RYS

A pilot study was used to test equipment, a headlight measurement system, developed for the measurement of light levels from vehicle headlights traveling on highways. Discussed are the locations where a preliminary study concluded that data on actual light levels are needed by drivers for the night driving task, the characteristics of highway sites to be used for data collection, the identification and classification of vehicle and headlight characteristics used for data collection purposes, the preliminary field studies that were performed, and a summary of the field data results from those field studies. It is emphasized that the field study results, though believed reasonable and accurate, are from a very small sample.

Government regulation of vehicle headlight design has historically been based on measurements of headlight output under laboratory conditions. Bench measurement of headlight photometrics can be quite accurate but fails to account for many of the factors that influence the light output in the real world. Among the factors not always accounted for are variations in headlight mounting height, improper aiming, voltage variations in the vehicle electrical system, dirty or corroded headlight lenses and reflectors, vehicle pitch angle as a result of the bouncing of the vehicle body on its suspension system, and vehicle yaw angle as determined by driver steering control. All of these factors can influence headlight performance and real-world light levels but can only be determined in the field. Thus, field measurements of headlight levels are needed as the basis for realistic performance standards for headlights.

A Kansas State University (KSU) team designed and developed a headlight measurement system (HMS) to provide NHTSA with a tool to measure light levels from vehicle headlights in the field. Following completion of the HMS calibration and sensitivity tests and preliminary testing of field procedures at KSU, Midwest Research Institute (MRI) conducted a pilot test of the procedures for a field study of moving vehicles. This study was performed on in-service vehicles on actual highway sections. The pilot study was followed by obtaining a sample of field data to the extent that remaining project resources would allow. The following discussion presents the rationale on how this field study was planned and conducted, and results of the field data that were collected.

A final project report and two manuals detailing the equipment design and operation were written (*1*).

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LIGHT METER LOCATIONS

The HMS can measure light levels at any locations where the light meters, used as sensors for the system, are placed. For development of performance standards, measurements are needed at those locations that are critical to performance of the driving task. The critical issues in headlight performance include the availability of enough light for the driver to see the pavement edges or lane lines and to read signs without creating excessive glare to oncoming drivers. Thus, the intensity and beam pattern of headlights should supply sufficient light at some locations, but not supply too much at others.

Six locations at which light levels are of critical importance to drivers, which should be considered in determining headlight performance, have been identified for this study:

1. Right pavement edge at pavement level,
2. Left pavement edge at pavement level,
3. Typical roadside sign location on the right side of the roadway,
4. Overhead sign location in the right lane,
5. Overhead sign location in the left lane, and
6. Eye location of an oncoming driver in the opposing lane.

Procedures for measuring light levels at each of these locations were developed in this study. Figure 1 shows the location of each of these light meter locations. The cross-sectional location for each of these light meters is discussed in the next sections, followed by a discussion of the distance between these light meters and the approaching vehicles at the time when readings are taken.

Right Pavement Edge at Pavement Level

Performance of the vehicle guidance task at night requires the driver to be continuously aware of the roadway alignment and position of the vehicle relative to the pavement edge. When traveling at highway speeds, drivers must see the pavement edge at some distance ahead to position their vehicles properly.

A light meter to measure the light level from vehicle headlights was placed near the pavement edge. It would be desirable to take light measurements at pavement level at the edge of the traveled way; however, this is not practical. A light meter on the pavement edge would be exposed and could be struck by passing vehicles. Furthermore, unless a hole or depression is made in the pavement, it is physically impossible to place a light meter exactly at the elevation of the pavement

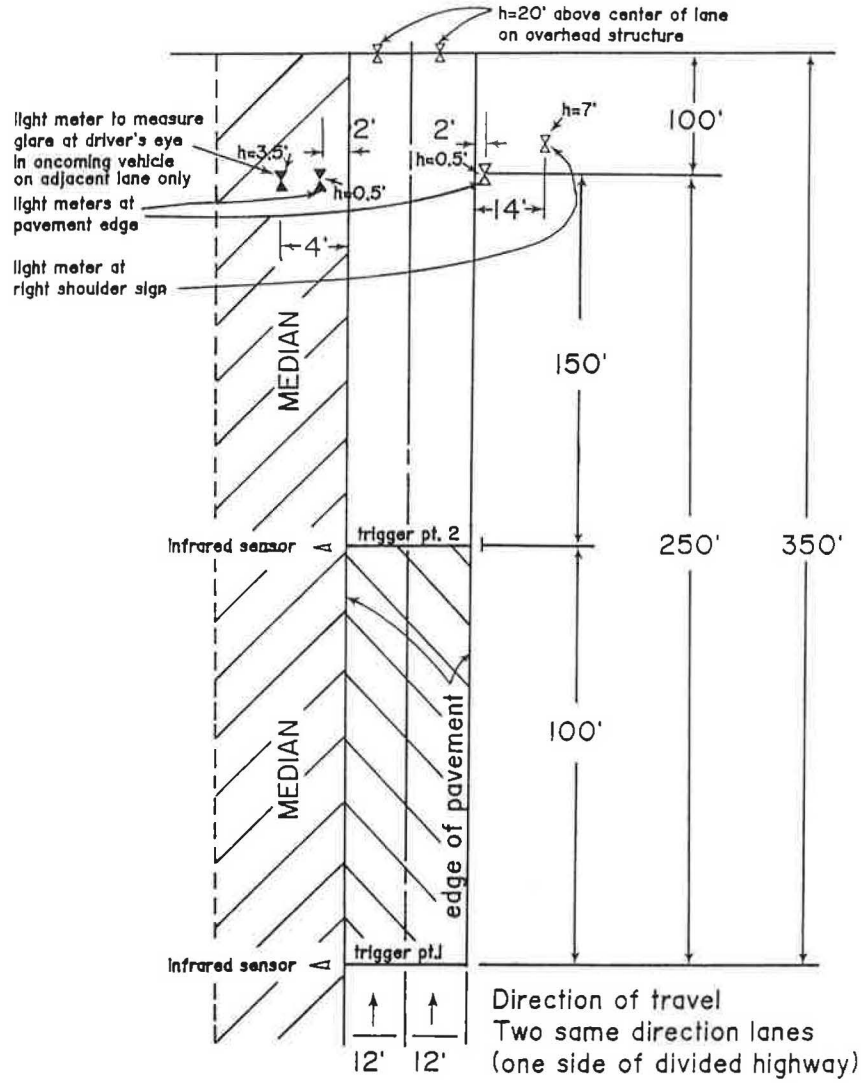

FIGURE 1 Six key locations selected for this study.

edge. Thus, a compromise had to made. The specific location selected for placing a light meter near the pavement edge was on the roadway shoulder, 2 ft (0.61 m) outside the edge of the traveled way and 6 in. (14 cm) above the elevation of the outside edge of the pavement.

Left Pavement Edge at Pavement Level

The placement of the light meter at the left pavement edge is entirely analogous to the placement at the right pavement edge. The light meter at the left pavement edge was placed on the left shoulder of the road, 2 ft (0.61 m) outside the edge of the traveled way and 6 in. (14 cm) above the elevation of the outside edge of the pavement.

Roadside Sign Location on Right Side of Roadway

The illumination level at a typical roadside sign was measured by a light meter located on the right side of the road at a 14 ft (4.27 m) offset from the edge of the traveled way and at an elevation of 7 ft (2.14 m) above the edge of the traveled way. This offset and mounting height place the sensor where the center of a warning sign would be placed on a conventional (nonfreeway) highway according to the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (*2*). The MUTCD specifies that the inside of the sign should be 12 ft (3.66 m) from the edge of the roadway and the bottom of the sign should be 5 ft (1.53 m) above the pavement edge. (Large guide signs used on freeways are mounted further from the road and at higher mounting heights. Light levels on such signs could be studied with the HMS, but for the present study data collection was limited to the right roadside sign location generally used on conventional highways.)

Overhead Sign Location Above the Right Lanes

Light meters at overhead signs were placed at 20 ft (6.1 m) above the center of the right lane, which corresponds to the center of a typical overhead sign location. The MUTCD sets

a minimum vertical clearance of 17 ft (5.19 m) for overhead signs (*2*). The 20-ft (6.1-m) mounting height allows for the minimum clearance plus half of the height of a 6-ft (1.83-m) sign. The location requirements for overhead sign measurements limit data collection for all practical purposes to sites with existing overpass structures that cross over the highway on which light meters could be mounted unless supporting framework is built.

Eye Location of Oncoming Drivers in the Opposing Lane

The final critical location for measurement of light levels is at the driver's eye position of an oncoming vehicle. It is assumed that the oncoming driver's eye is located 4 ft (1.22 m) outside the highway centerline and 3.5 ft (1.07 m) above the pavement. This is the position at which the oncoming driver's eye would be on a two-lane, two-way highway. Light levels cannot be measured at this location on a two-lane, two-way highway because the sensor would be within the lane used by opposing traffic, but light levels can be measured at the corresponding position on the left shoulder of a one-way or divided highway. At such locations, measurements must be limited to vehicles traveling in the far-left lane. On a one-way roadway with only one travel lane, measurements could be made for all vehicles.

SYSTEM SETUP AND KEY POINTS

Longitudinal Placement of Light Meters

No limitations, other than the limitations on the lengths of the connecting cables, exist on the distance between the infrared sensors that are tripped by the passing vehicles and the light meters where the illumination levels from their headlights are measured. Thus, the distance between the vehicles and the light meters can be varied to suit the needs of a particular data collection effort. For all data collected in this study, measurements were made with the two infrared sensors located 250 and 350 ft (76.3 and 106.8 m) from the overhead sign meters and 150 and 250 ft (45.8 and 76.3 m) from all of the other meters. The light meters for the overhead sign location were farther from the infrared sensors than the other light meters because, as a result of the vertical angle, light levels on an overhead sign would be expected to be higher at the greater distance. In other words, the distances used for the other light meter locations would be too close for normal reading of an overhead sign.

Control Parameters and Trigger Points

Central to the HMS is the Quiklite program. This program, written in QuikBasic and compiled in an executable module stored under the file name QUIKLITE.EXE, can be run by entering the command QUIKLITE from the MS DOS prompt. Five fundamental parameters that control the operation of the Quiklite program must be set before headlight data can be recorded:

- Sample rate,
- Average base,
- Distances between Switches 1 and 2,
- Time between second switch and third set of readings, and
- Sensor ranges.

All five parameters can be changed at any time depending upon the data collection objectives, site geometry, equipment configuration, and so forth.

The sample rate is the number of readings per second made from each light meter while data are being collected. The software is capable of making from 1 to 4,000 readings per sec. However, it was found that the processor speed sets a practical upper limit on the combination of sample rate, average base, and distance between infrared sensors. Trials established that the most satisfactory value was 500 readings per sec, which was the setting used for all data collected as part of this study.

The average base is the number of consecutive readings that are averaged to create each value stored in the data file. The average base used in this study was 50 readings (i.e., one average stored for each 50 readings made). Thus, the combination of a sampling rate of 500 readings per sec and an average base of 50 readings implies that each average light level stored in the data base is the average of 50 readings made over an interval of 0.1 sec.

The user must enter the distance (in feet) between the infrared sensors used to detect vehicle passages (called Switches 1 and 2 in the program). In all cases in this study, the distance between sensors was 100 ft (30.5 m).

The data collection interval for each vehicle passage selected for measurement of headlight levels begins when the first infrared sensor is triggered by the vehicle (T_1), continues when the second infrared sensor is triggered (T_2), and ends at a specified time interval after the second sensor is triggered (T_3). In this study, this time interval between the second switch and the third set of readings was set as 1.1 sec.

The range selected for each of the eight light meters must be set in the Quiklite program, and these ranges may be changed at any time. These ranges must also be manually set on the light meter itself. The allowable ranges that can be selected are 0.001 to 0.999, 1.00 to 9.99, 10.0 to 99.9, 100.0 to 999.9, and 1,000.0 to 9,999.

SITE SELECTION

Sites for field data collection were selected to meet the following criteria:

- Level roadway grade,
- Tangent roadway alignment,
- Well removed from intersecting roadways or ramps,
- Moderate nighttime traffic volumes,
- Limited background ambient light levels, and
- Suitable mounting locations for all light meters.

The requirement for a level roadway grade is intended to make interpretation of the measured light levels easier. On a steep upgrade or downgrade, the effect of the grade on

headlight aim would need to be accounted for. Although no roadway grades are truly level because of drainage considerations, many sites with grades under 1 percent are available.

Data collection sites should be located on tangent roadway alignments, not on horizontal curves. On a tangent site, vehicle yaw angles should not vary markedly from a path parallel to the roadway centerline. On horizontal curves, driver steering inputs may vary more widely, leading to larger variations in vehicle yaw angle at any given point on the roadway.

Data collection sites should be well removed from intersecting roadways and ramps both to minimize traffic operation interferences with the vehicle being measured and to avoid the presence of turning or merging vehicles. These vehicles should not be sampled because they are not traveling parallel to the roadway centerline.

Sites with high nighttime traffic volumes should be avoided. The difficulty is finding vehicles that are isolated enough in traffic so that the light levels from their headlights can be measured without interference from adjacent vehicles. Vehicles tend to travel in platoons, even on multilane roadways, so lower traffic volumes are necessary to obtain a reasonable number of isolated vehicles. In the preliminary field studies, the definition used for an isolated vehicle was a vehicle that had no following vehicle traveling in the same direction within ±500 ft (152.5 m). A suggested maximum nighttime traffic volume is 200 vehicles/hr (an average of 3.3 vehicles/min).

It is desirable for the background ambient light level at a data collection site to be as low as possible. Because the light levels from headlights are determined as the difference between a reading in the presence of a vehicle and a background reading, and because the light levels from headlights are relatively low at some locations (e.g., overhead signs), it is desirable for the background reading to be as small a percentage of the headlight reading as possible. Sites with roadway lighting should be avoided and ambient light sources should be checked under nighttime conditions to determine whether a site is suitable. Also, ambient readings should be fairly constant.

Finally, a site should have suitable mounting positions for all, or as many as possible, of the light meter locations discussed above. As a practical matter, it was found that most sites do not provide suitable locations for all of the light meters, so two different types of sites were selected. A freeway site with an overpass is needed for measurement of overhead sign locations. However, a divided or one-way roadway with only one travel lane is most suitable for determining glare from oncoming vehicles. Most sites suitable for overhead sign or glare measurement are also suitable for pavement edge and roadside sign measurement.

Two sites were used in the preliminary field studies, one with an overpass and one with a one-lane, one-way roadway. Both sites were located in a relatively rural portion of a metropolitan area with a population of more than 1 million.

Site 1 was located on Interstate 435 and North Agnes Avenue in Kansas City, Missouri. This site was located on a four-lane divided freeway with a tangent, relatively level roadway. The site had a freeway overpass where a minor street crossed the freeway, but there were no ramp connections between the freeway and the street on the overpass. (As discussed, it was desirable to avoid sites with ramps or turning roadways.)

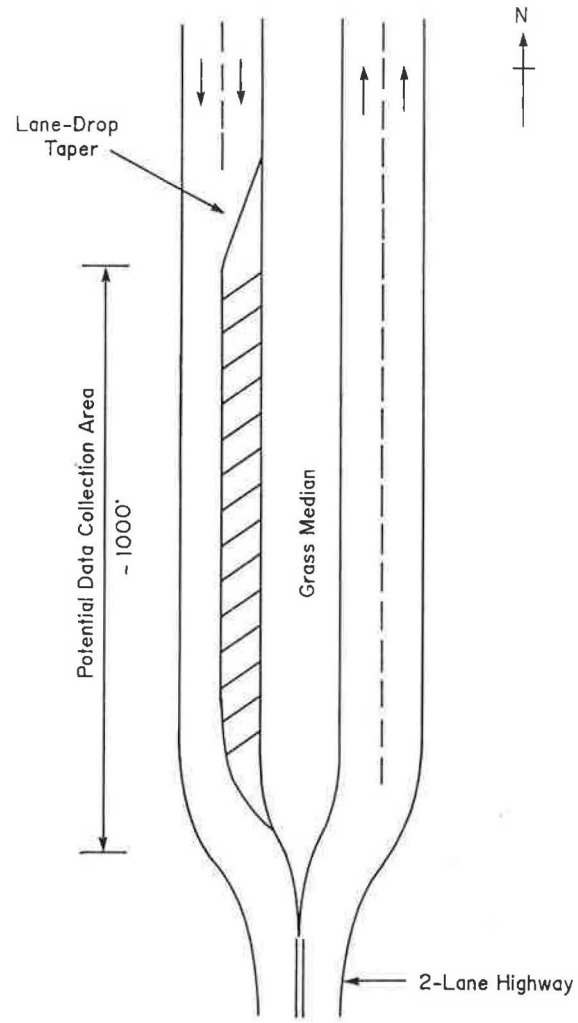

FIGURE 2 Nonfreeway locations (Site 2).

Temporary installation of overhead sign sensors on this overpass structure proved feasible.

Site 2 was located on US-169 between Cookingham Drive and Northwest 108th Street in Kansas City, Missouri. This site was located at the transition from a four-lane divided freeway to a two-lane undivided highway. In making this transition, one of the southbound lanes was dropped approximately 1,000 ft before the median divider ended and the two-lane undivided highway began. The portion of the roadway with a median divider but only one southbound roadway had a tangent, relatively level alignment and provided an ideal site for a light meter on the left shoulder to measure glare from oncoming vehicle headlights. Figure 2 shows a schematic sketch of this location.

IDENTIFICATION OF VEHICLE AND HEADLIGHT CHARACTERISTICS

In addition to measuring the illumination levels from vehicle headlights, it would be desirable to know as much as possible about those vehicles and their headlight systems. This re-

quired either that data about each vehicle be collected in the field or that the manufacturer, model, and year be accurately identified.

Substantial effort went into trying to read vehicle license plate numbers from the roadside at night. If the license plate number could have been obtained, vehicle registration records could be used to determine the vehicle identification number (VIN). From the VIN, available NHTSA records would allow accurate identification of the manufacturer, model, and year of the vehicle and, in most cases, the characteristics of the headlight system.

Vehicle license numbers can be read quite easily from the roadside of high-speed highways during the day, but this proved to be quite difficult at night. Various systems involving viewing with the naked eye, with binoculars, and with low-light video were tried, but none proved satisfactory. The best performance achieved was determining the license plate numbers of approximately one vehicle out of every six and the sample of vehicles obtained in this way was probably biased (e.g., limited to well-lighted license plates of newer cars). It would be possible to obtain license plate numbers from a following vehicle after the vehicle had passed through the measurement site, but this would drastically slow the rate at which data were collected. Likewise, if vehicles were stopped and interviewed downstream, this could cause delay and safety problems. There are sophisticated systems that could have been designed or adapted from state-of-the-art video technology, but such systems were beyond the resources available to this project. Therefore, the attempts to record license numbers were abandoned.

Instead, vehicle and headlight types were noted and recorded for each vehicle for which light levels were measured in the preliminary field studies. These characteristics were as follows:

- Vehicle type—passenger car, pickup truck, van, single-unit truck, tractor-trailer with cab-over-engine tractor, and tractor-trailer with cab-behind-engine tractor.
- Headlight type—round sealed beam, rectangular sealed beam, and aerodynamic (replaceable bulb).

This list of characteristics is about the limit that an average person can differentiate and record as vehicles approach at night. Similar information could be recorded in greater detail in future studies or, if sufficient resources were available, sampled cars could be stopped and interviewed (though the dangers must be considered) or a system capable of reading license plates at night could be used. Notes on headlight condition were also recorded.

PRELIMINARY FIELD DATA COLLECTION

Preliminary field studies were conducted during July through September 1990. Field studies were performed on a total of nine nights during that period—five nights at Site 1 and four nights at Site 2.

Because only two light meters were available, they were used at alternative locations on different nights. (The equipment can record seven light meters simultaneously; the decision to purchase only two meters for the equipment tests was made by the contract monitor.) Light meters at the right pavement edge location, the roadside sign location, and the overhead sign location over the right lane were tested at Site 1. All measurements at Site 1 involved vehicles traveling in the right lane of the freeway with headlights on low beam. The overhead sign location over the left lane was not tested, but data collection would be entirely analogous to the location over the right lane. The right and left pavement edge location, the roadside sign location, and the oncoming vehicle glare location were tested at Site 2.

The purpose of the initial studies during July was to test the equipment under field conditions, develop data collection procedures, test the ability to read vehicle license plates, and debug the computer software. Following resolution of these problems, a small sample of usable data was obtained with the system during August and September 1990. Data collection during this final period was quite successful. However, because at that point project resources were exhausted, sampled size was limited.

RESULTS OF THE PRELIMINARY FIELD STUDIES

Because of limited resources, only a pilot study to test the equipment and obtain a small sample was possible. This fact is stressed because, given the very small sample, one should be very careful in interpreting the results. The results appear to be valid, and the research team is pleased with them, but conclusions made from this or any small sample could be misleading if extrapolated to broad conclusions.

The Statistical Analysis System (SAS) on the KSU mainframe was used to analyze the data. The data consisted of measurements of all vehicles traveling in the right lane with headlights on low beam only. The data that had been stored in the field computer data base Klite.Dta were copied with the help of the file transfer program KERMIT to the mainframe KSUVM. The following SAS procedures were used: Proc Sort, Proc Means, and Proc Chart. The first procedure was used to sort the data by vehicle type and sensor location to check for and eliminate possible data entry errors and values obviously in error [e.g., values considered impossible, such as a calculated speed of 140 mph (225 km/hr) or similar unbelievable values]. (There were very few such values.) Next the background readings were subtracted from all the recordings, and the resulting "true" light intensity levels were used in further analysis of the data.

The mean values of illumination were calculated for each vehicle type and sensor position and for each headlight type and sensor position. The mean values of illumination for different sensor locations can be found in Tables 1, 2, 3, and 4. Tables 1 and 3 contain the mean values gathered during the data collection on Site 1 (Interstate 435 and North Agnes Avenue). Tables 2 and 4 contain data from Site 2 (US-169 between Cookingham Drive and Northwest 108th Street). The mean values of illumination for different headlight types can be found in Tables 5 and 9.

From Table 1 it can be seen that for the passenger cars the readings of illumination monotonically increased as the car

TABLE 1 Mean Values of Illumination at Right Shoulder Sign Location

Vehicle Type	Number of Vehicles Measured	Sensor[a]	Illumination (fcd) Position T_1 Mean	Position T_1 Std. Dev.	Position T_2 Mean	Position T_2 Std. Dev.	Position T_3 Mean	Position T_3 Std. Dev.	Mean Vehicle Speed (mph)
Passenger car	44	rs	.0409	.0229	.0597	.0499	.0731	.0341	66.4
Van or pickup	23	rs	.0692	.0601	.0832	.0644	.0744	.0269	63.5

[a]rs = right shoulder sign location.

TABLE 2 Mean Values of Illumination at Glare Sign Location and Left Pavement Edge

Vehicle Type	Number of Vehicles Measured	Sensor[a]	Illumination (fcd) Position T_1 Mean	Position T_1 Std. Dev.	Position T_2 Mean	Position T_2 Std. Dev.	Position T_3 Mean	Position T_3 Std. Dev.	Mean Vehicle Speed (mph)
Passenger car	53	gs	.0475	.0452	.1068	.1552	.1933	.1519	57.6
		lp	.0899	.0691	.1956	.1488	1.2845	.6966	57.6
Van or pickup	23	gs	.0548	.0291	.0945	.0523	.2511	.1459	57.3
		lp	.1121	.0628	.2508	.1210	1.5278	.8655	57.3

[a]gs = glare sign location
lp = left pavement edge location

TABLE 3 Mean Values of Illumination at Right Overhead Sign Location and Right Shoulder Sign Location

Vehicle Type	Number of Vehicles Measured	Sensor[a]	Illumination (fcd) Position T_1 Mean	Position T_1 Std. Dev.	Position T_2 Mean	Position T_2 Std. Dev.	Position T_3 Mean	Position T_3 Std. Dev.	Mean Vehicle Speed (mph)
Passenger car	57	ro	.0082	.0036	.0118	.0040	.0240	.0064	63.6
		rs	.0350	.0182	.0437	.0147	.0607	.0179	63.6
Semi CBE	25	ro	.0087	.0068	.0129	.0062	.0245	.0101	64.5
		rs	.0428	.0428	.0645	.0621	.0704	.0453	64.5
Semi COE	11	ro	.0064	.0029	.0098	.0041	.0191	.0069	66.3
		rs	.0327	.0159	.0487	.0212	.0616	.0235	66.3
Straight truck	4	ro	.0066	.0018	.0098	.0039	.0224	.0065	64.4
		rs	.0274	.0110	.0388	.0103	.0587	.0130	64.4
Van or pickup	39	ro	.0119	.0151	.0279	.0872	.0384	.0754	63.0
		rs	.0517	.0435	.0604	.0459	.0678	.0441	63.0

[a]ro = right overhead sign location
rs = right shoulder sign location

TABLE 4 Mean Values of Illumination at Right Pavement Edge Sign Location

Vehicle Type	Number of Vehicles Measured	Sensor[a]	Illumination (fcd) Position T_1 Mean	Position T_1 Std. Dev.	Position T_2 Mean	Position T_2 Std. Dev.	Position T_3 Mean	Position T_3 Std. Dev.	Mean Vehicle Speed (mph)
Passenger car	57	rp	.2743	.1720	.7639	.4025	3.1154	1.6256	58.1
Semi COE	2	rp	.2700	.0314	.8847	.0206	1.2668	.4156	60.6
Van or pickup	45	rp	.3659	.2434	1.0940	.6747	3.4301	1.7836	57.0

[a]rp = right pavement edge location

approached the right shoulder sensor. This might be a characteristic of a wide-angle beam, which would cause the readings to increase as the car gets closer to the sensor. This is consistent with the observations taken on different days at the same location (Table 3). A similar trend was observed for all other vehicles measured during all days except for the data in Table 1. For the vans and pickups (see Table 1) the readings increased for location T_2, but then dropped at location T_3. This might be an indication of a narrow-angle beam (i.e., the readings are further out in the beam pattern).

In comparing the data between vehicles, it can be concluded that vans and pickups in general gave higher intensity readings at all sensor positions except the T_3 reading of the right shoulder sensor and tractor-trailer trucks with cab-over-engine

TABLE 5 Illuminance of the Right Overhead Sign Location by Headlight Type

Position	Number of Vehicles Measured	Headlight Type[a]	Illumination (fcd) Mean	Std.Dev.	Min. Value	Max. Value
T_1	16	A	.0157	.0219	.0054	.0969
T_2	16	A	.0485	.1357	.0073	.5570
T_3	16	A	.0581	.1165	.0183	.494
T_1	7	RND	.0052	.0024	.0023	.0089
T_2	7	RND	.0087	.0026	.0059	.0126
T_3	7	RND	.0199	.0042	.0143	.0242
T_1	52	RCT	.0090	.0051	.0025	.0368
T_2	52	RCT	.0131	.0064	.0054	.0476
T_3	52	RCT	.0249	.0079	.0122	.0593
T_1	61	U	.0080	.0052	.0025	.0377
T_2	61	U	.0117	.0050	.0047	.0342
T_3	61	U	.0231	.0080	.0109	.0592

[a]A - Aerodynamic
RND - Sealed beam, round
RCT - Sealed beam, rectangular
U - Unknown

TABLE 6 Illuminance of the Right Shoulder Sign Location by Headlight Type

Position	Number of Vehicles Measured	Headlight Type[a]	Illumination (fcd) Mean	Std.Dev.	Min. Value	Max. Value
T_1	16	A	.0469	.0301	.0166	.1295
T_2	16	A	.0618	.0496	.038	.2387
T_3	16	A	.0852	.0447	.0322	.2162
T_1	7	RND	.0260	.0179	.0107	.0639
T_2	7	RND	.0333	.0100	.0243	.0508
T_3	7	RND	.0617	.0140	.0421	.0866
T_1	52	RCT	.0464	.0376	.0119	.2247
T_2	52	RCT	.0533	.0314	.0212	.1752
T_3	52	RCT	.0611	.0297	.0281	.1839
T_1	61	U	.0362	.0297	.0111	.1807
T_2	61	U	.0517	.0426	.0176	.2681
T_3	61	U	.0624	.0331	.0280	.2537

[a]A - Aerodynamic
RND - Sealed beam, round
RCT - Sealed beam, rectangular
U - Unknown

TABLE 7 Illuminance of the Right Pavement Edge by Headlight Type

Position	Number of Vehicles Measured	Headlight Type[a]	Illumination (fcd) Mean	Std.Dev.	Min. Value	Max. Value
T_1	21	A	.4122	.2744	.1125	1.054
T_2	21	A	1.061	.7095	.3766	2.737
T_3	21	A	4.158	1.933	2.036	10.175
T_1	13	RND	.2273	.1812	.0259	.5898
T_2	13	RND	.6800	.4136	.1071	1.512
T_3	13	RND	2.609	.9329	.9242	4.569
T_1	67	RCT	.3016	.1853	.0010	.7629
T_2	67	RCT	.9008	.5281	.0194	2.582
T_3	67	RCT	3.084	1.652	.4089	8.685

[a]A - Aerodynamic
RND - Sealed beam, round
RCT - Sealed beam, rectangular

TABLE 8 Illuminance of the Glare Sensor Location by Headlight Type

Position	Number of Vehicles Measured	Headlight Type[a]	Illumination (fcd) Mean	Std.Dev.	Min. Value	Max. Value
T_1	16	A	.0400	.0191	.0222	.0928
T_2	16	A	.0696	.0355	.0339	.1780
T_3	16	A	.1655	.0843	.0860	.4143
T_1	6	RND	.0755	.1189	.0142	.3177
T_2	6	RND	.1191	.1662	.0306	.4571
T_3	6	RND	.2933	.3365	.0823	.9712
T_1	53	RCT	.0499	.0291	.0096	.1336
T_2	53	RCT	.1121	.1477	.0255	.8160
T_3	53	RCT	.2153	.1369	.0687	.8125

TABLE 9 Illuminance of the Left Pavement Edge by Headlight Type

Position	Number of Vehicles Measured	Headlight Type[a]	Illumination (fcd) Mean	Std.Dev.	Min. Value	Max. Value
T_1	16	A	.0856	.0444	.0364	.2006
T_2	16	A	.2073	.1065	.0738	.4649
T_3	16	A	1.5113	.9629	.4162	4.2411
T_1	6	RND	.1145	.1650	.0194	.4489
T_2	6	RND	.2617	.3515	.0516	.9711
T_3	6	RND	1.005	.5904	.2540	1.954
T_1	53	RCT	.0984	.0577	.0148	.2277
T_2	53	RCT	.2087	.1175	.0479	.4678
T_3	53	RCT	1.3383	.6982	.3121	3.589

[a]A - Aerodynamic
RND - Sealed beam round
RCT - Sealed beam, rectangular

(C-O-E) showed the lowest light intensity. The sensor located at the overhead sign indicated the lowest readings in the range of 0.006 to 0.012 fcd at T_1 (350 ft, 106.8 m), 0.009 to 0.028 fcd at T_2 (250 ft, 76.3 m), and 0.019 to 0.038 fcd at T_3. All these readings were approximately 3 to 5 times lower than those of the sensor located at the right shoulder. The highest readings were recorded for the right pavement edge sign location and varied in range, for example 0.27 fcd for passenger car at T_1 to 3.12 fcd at T_3.

From Tables 5, 6, 7, 8, and 9 it can be seen that the highest light intensity levels were for the aerodynamic type headlight for all the sensor positions except the glare sensor position.

CONCLUSIONS

At the time the study was being planned, the portable computer that was used was state-of-the-art. That is no longer the case, and a new generation computer should be incorporated into the system. During the course of the study the research staff determined that more expensive sensors (in the $6,000 to $10,000 range) could be custom made and incorporated into a more sophisticated, expensive system. However, for the resources available the system developed proved to be adequate for its main purpose, which was to determine the distribution of light output of the current fleet of highway vehicles at any set of desired points quickly, easily, and with reasonable accuracy with relatively inexpensive, off-the-shelf hardware. Details of the headlight measurement system used in this study are presented by Rys et al. elsewhere in this Record. The project final report and equipment systems manual and user's manual will provide valuable information for future studies to gather information on a larger sample under varied conditions, thus enabling one to make significant conclusions about the output of the fleet of highway vehicles and to see how that output varies with vehicle and headlight type, and over time. The final report contains an appendix with detailed suggestions on upgrading the system to increase its speed and capacity and make it universally applicable to an almost infinite number of traffic situations and large samples.

It is believed that the results presented are sufficiently accurate for the stated purpose, albeit at one point in time at two specific sites with their own specified set of characteristics. Any generalization or extrapolation of results and conclusions that the reader wishes to make should be made very cautiously.

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