HIGHWAY RESEARCH RECORD

Number 325

Traffic Signals and Signs

5 Reports

Subject Areas

51 Highway Safety
53 Traffic Control and Operations

HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

WASHINGTON, D.C. 1970

ISBN 0-309-01824-2

Price: $1.80

Available from

Highway Research Board
National Academy of Sciences
2101 Constitution Avenue
Washington, D.C. 20418

Department of Traffic and Operations

Harold L. Michael, Chairman
Purdue University, Lafayette, Indiana

E.A. Mueller
Highway Research Board Staff

COMMITTEE ON OPERATIONAL EFFECTS OF GEOMETRICS
(As of December 31, 1969)

John W. Hutchinson, Chairman
University of Kentucky, Lexington

John A. Dearinger, Secretary
University of Kentucky, Lexington

W.R. Bellis
Elmer N. Burns
Stanley R. Byington
Arno Cassel
Robert R. Coleman
B.K. Cooper
Joseph C. Corradino
James J. Crowley
Harley T. Davidson
W.A. Frick
William G. Galloway
Albert L. Godfrey, Sr.
William C. Habig
George F. Hagenauer
John W. Horn
Robert S. Hostetter
Rajendra Jain
Thomas W. Kennedy
Janis H. Lacis
Richard A. Luettich
Joseph M. McDermott
Thomas E. Mulinazzi
Jack Noblitt
John D. Orzeske
Stuart R. Perkins
E.F. Reilly
Neilon J. Rowan
James J. Schuster
R.B. Shaw
James Correll Spencer
Vasant H. Surti
Asriel Taragin
Paul R. Tutt
Walter C. Vodrazka
Earl C. Williams, Jr.
Jason C. Yu

COMMITTEE ON TRAFFIC CONTROL DEVICES
(As of December 31, 1969)

Robert E. Conner, Chairman
Bureau of Public Roads, Washington, D.C.

John L. Barker
James D. Barnett
O.I. Bermant
Donald S. Berry
Robert L. Bleyl
James W. Booth
Abner W. Coleman
Charles E. Dare
Robert D. Dier
Robert N. Doble
William H. Dorman
J. Robert Doughty
Roy D. Fonda
Joseph V. Galati
Daniel L. Gerlough
Michael J. Gittens
Albert L. Godfrey, Sr.
Alan T. Gonseth
George W. Gray
J.T. Hewton
George W. Howie
Rudolph J. Israel
Marshall Jacks
Lester R. Jester
Barnard C. Johnson
Anthony C. Kanz
Ken F. Kobetsky
Leonard I. Lann
Clyde E. Lee
Joseph E. Lema
Holden M. LeRoy
Sam E. Luebbert
S.A. Marshall, Jr.
Robert D. McMillen
Joseph A. Mickes
J.P. Mills, Jr.
Zoltan A. Nemeth
Robert J. Nolan
Herbert Norder
John J. O'Mara
Albert S. Palatnick
A.R. Pepper
Russell E. Puckett
Frank G. Schlosser
Stanley T. Siegel
R.G. St. John
Asriel Taragin
James I. Taylor
James A. Thompson
Robert E. Titus
Francis E. Twiss
Robert E. Wert
Arthur M. White
Earl C. Williams, Jr.
James K. Williams
Robert M. Williston
Jason C. Yu

Foreword

Practicing traffic engineers and those responsible for operations on streets and highways will find items of practical interest in the papers presented in this RECORD. The subjects cover the full range of traffic control devices, including signs, signals, and pavement markings. Some of the findings will also be of interest and value to those officials responsible for approval of national standards for uniform traffic control devices.

Are overhead flashers and traffic signals effective in reducing accidents at low-volume, high-speed rural intersections? Cribbins and Walton searched for the answer to this question by analyzing before-and-after accident data at 33 North Carolina intersections. A number of conclusions regarding effect of signals and flashers at different kinds of intersections are presented.

In the interest of traffic safety, most authorities have been emphasizing clear recovery areas adjacent to the traveled way. Concerned about the effects of increasing the lateral distance between the edge of the road and major guide signs, King analyzed the influence on the legibility of such signs. In addition to the obvious effect on letter size, King points to other side effects that could be detrimental to safety. Thoughtful discussions that extend the value of this work are also included.

The use of magnetic loop vehicle detectors in measuring the speeds and lengths of vehicles can lead to errors in excess of 10 percent. Gazis and Foote determined that the reliability could be greatly improved by careful calibration, and they also identified other avenues for improvement yet to be explored.

The paper by Roth is a report on a study to determine the effects of special color coding of signs, pavement markings, and delineators at interchanges on a rural freeway system. Roth used interviews with drivers, observations of driving patterns, and accident analyses to study the effects of modifying the standard yellow-white-green colors by using blue to identify exit ramps. He concluded that such color coding offered good potential for reducing confusion and accidents and for easing the driver's tasks.

Finally, Woods and Rowan analyzed driver information requirements in the area of street name signs, concluding that current design standards do not meet today's needs. They offer design criteria recommendations for this important, but somewhat neglected, signing area.

Contents

Traffic Signals and Overhead Flashers at Rural Intersections: Their Effectiveness in Reducing Accidents

PAUL D. CRIBBINS and C. MICHAEL WALTON,
Department of Civil Engineering, North Carolina State University, Raleigh

Two specific types of operational improvements—overhead flashers and traffic signals installed at low-volume, high-speed rural intersections—were selected for investigation in this study. The effectiveness of the devices in reducing traffic accidents was earmarked as the primary objective of the analysis. Initially, all flashers and signal devices installed in North Carolina since 1965 were considered, but subsequent investigation and a more restrictive definition of a test site reduced the original inventory from 72 flashers and 153 signals to 14 flashers and 19 signals. A before-and-after study was made encompassing minimum time frames of 1 year prior to and immediately after installation of the device. Accident exposure during the two periods was compared on the basis of exposure rates, severity indexes, and equivalent property damage only accidents and rates. It was determined that the equivalent property damage only rate, rather than the normally used accident rate, was the most reliable and significant indicator of accident consequences. If all other factors were constant, any significant change in rate after installation of the control device could be attributed to the presence of the device. The relationship between the installation of signals and equivalent property damage only rate reduction was not statistically significant except for undivided highway intersections. The relationship between the installation of a flashing beacon and rate reduction was found to be statistically significant at the 1 percent confidence level.

•IN RECENT YEARS, particular attention has been devoted to the construction of the Interstate Highway System and other access-controlled facilities and to the evaluation of the effectiveness of such facilities for increasing road-user benefits and reducing accident costs. To a great extent the remainder of the rural highway system has been overlooked, even though a large percentage of the vehicular traffic must ultimately travel over this network to reach its destination.

Unfortunately, records show that accident experience at rural intersections of low-volume highways is disproportionately high for the volume accommodated, and the need to devote attention to this problem is obviously long overdue. There are several conventional traffic control measures that can be utilized in an attempt to reduce the accident cost at these locations; however, there is no guarantee that the treatment will have a positive effect on the accident experience. A need exists to determine which treatment optimizes accident reduction so that the best treatment for specific conditions can be implemented. It is a well-documented fact that certain traffic control measures actually cause an increase in specific types of accidents or in the severity of the accident. Therefore, it appears that the impact of various traffic control treatments in use at high-accident locations should be evaluated prior to their installation.

Paper sponsored by Committee on Traffic Control Devices and presented at the 49th Annual Meeting.

Few factual data are available regarding the relative merits of specific minor improvements, such as flashing beacons and traffic signals, at rural intersections. The expenditures required to install such devices are normally relatively small, but the benefits that can be derived in the form of increased safety for the road user can be very significant. Many traffic engineering departments within highway agencies in the various states are installing traffic control devices at high-accident locations on rural highways without being able to predict with any success the subsequent accident reduction, if any, that can be expected. Traffic control measures should, of course, not be accepted as a panacea for all types of traffic intersection problems; indeed, improper or indiscriminate installation can sometimes create a less efficient and more dangerous condition than previously existed.

OBJECTIVES OF THE RESEARCH

The warrants presently cited in the Manual on Uniform Traffic Control Devices (1) for flashing beacons and traffic signals are general and are, at best, only a guide in relation to installation at high-accident locations on rural intersections. To avoid unwarranted use of the devices and a resulting reduction in efficiency, there is a real need for more specific guidelines that will assist the practicing engineer in evaluating the intersection and in selecting corrective measures that can be applied with reasonable promise of success. It is the primary objective of this study to measure the effectiveness of traffic signals and overhead flashers in reducing traffic accidents on low-volume, high-speed rural highway intersections.

Because the installation costs of the improvement are normally insignificant when compared with benefits from reduced accidents, attempts to evaluate the cost-effectiveness of the devices were not undertaken. The average cost of a flasher installation, for example, is approximately $500, and the anticipated benefits from accident reduction alone would often offset the cost for the device in 1 year. However, if it is necessary to evaluate cost effectiveness on future projects involving first costs of greater magnitude, the technique of investment return analysis (discounted cash flow) is recommended. Details of this procedure, along with examples involving accident costs on North Carolina highways, are discussed at length in a report published by the University of North Carolina Highway Safety Research Center (2). In essence, the approach encompasses a rational framework for comparison of installation, maintenance, and operating costs of the control device with anticipated reductions (benefits) in accident costs at the intersection. Cost effectiveness can then be quantified on the basis of rate-of-return on investment or payout period required for amortization.

METHODOLOGY

Site Selection

For both flashers and signal devices the initial requirement was to obtain a complete inventory of all flasher and signal installations from the Traffic Signal Section of the State Highway Commission's Traffic Engineering Department. Items of particular interest include the location of the devices, installation dates, and type of installation. Because all pertinent accident data were to be evaluated using before and after periods with the installation dates separating the periods, accuracy in determining exact installation dates was essential. Accurate records concerning installation dates for the devices were not available prior to 1965, and a computer printout of accident data was unavailable prior to 1963. Therefore, a prerequisite for site selection was an installation date of 1965 or later.

Other restraints on the site location included the requirements that the speed limit on at least one approach be 45 mph or greater to ensure that the intersection was rural and located on a high-speed facility, and that the installation be new rather than an existing device that had been upgraded.

Data Collection

Once installation dates were ascertained, before-and-after accident data were obtained from the computer printout available in the Traffic Engineering Department. In

all cases the before and after periods each encompassed a 1- or 2-year interval with equal and identical months in each period. Because the installation of flashers or signals requires a relatively limited amount of time, the before and after periods were immediately before and after the installation date. Care was taken not to overlook any accidents by reviewing all accident forms that might have been pertinent to the study. This procedure initially produced many unrelated accidents but provided the margin of safety necessary for the evaluation and also helped eliminate any uncertainty with regard to accuracy of available data. All accidents within 200 ft of the intersection were included in the actual analysis.

In the case of flashers, all sites were visited after analysis of the accident records, and each accident was re-evaluated on the site with regard to its relevance to the study. At the same time, a general information sheet was prepared, and a photograph of the intersection was taken. The impact of roadside friction on accident potential within the vicinity of the intersection was also investigated. In the process of evaluating the accidents, many of the locations were eliminated for the following reasons:

1. No approach had a speed limit of 45 mph or greater;
2. The signal or flasher was actually an upgraded installation;
3. Overall conditions or abutting land use had a significant change from the before to the after period; or
4. The installation date was too late to obtain an adequate amount of accident data in the after period.

As the final step in the data collection procedure the accumulated data were transferred to an inventory sheet (Fig. 1). The original list of 72 flashers and 153 signals was finally reduced to 14 flashers and 19 signals.

Data Analysis

In the before-and-after approach used to analyze the data on this project, the time frames were normally limited by the availability of accident information and the dates of installation. Maximum possible time frames were used, but a minimum period of 1 year was always required. It appears that the utilization of a minimum time period of 1 year would eliminate the effect of seasonal fluctuations in traffic volumes, while any shorter period would not be adequate for statistical analysis.

Accident data sheets were completed for each site in order to classify the accidents by type and severity. Use of the data sheets permitted the comparison of data by the following means:

1. Number of accidents and accident rates;
2. Number of equivalent property damage only (EPDO) accidents and EPDO rates; and
3. Severity index (SI).

The exposure for both the before and after periods was assumed to be the volume in the year of installation of the control device. The normal case would contain a lower traffic volume in the before period than in the after period. When the actual traffic volumes of the periods were used, the calculated rate differences increased, thereby keeping the assumption on the conservative side. The exposure rate for each intersection was calculated by summing the average daily traffic (ADT) volumes for each approach and dividing by two. This figure was expanded by the number of days in the period. This value represents the total number of vehicles entering the intersection for the given time frame and is expressed in terms of million vehicles. The following equation results:

$$ER = \left(\frac{\Sigma ADT}{2}\right) 365\ Y$$

where

ER = exposure rate in vehicles (expressed in million vehicles),
ΣADT = summation of average daily traffic for each intersection approach, and
Y = number of years in the time frame.

Division 2

Site Location (Site#)	County	Speed Limit	Installation Date	Number of Approaches	Hanger Type	Head Type	Number of New Poles
U.S. 70 & SR 1300 (10)	Carteret	60 MPH	21 Feb.'68	3	1-3 Way	3-8"	2
N.C. 58-SR 1001 (11)	Jones	25 & 35 MPH	6 Dec.'66	4	1-4 Way	4-8"	0
N.C. 11 - SR 1133 (12)	Pitt	20 & 35 MPH	4 Oct.'67	4	2-2 Way	4-8"	0
N.C. 43 - N.C. 102 (13)	Pitt	55 MPH	Apr.'67	4	1-4 Way	4-8"	0
N.C. 11-W.Railroad (Main St.-SR 1434)(14)	Pitt	35 & 20 MPH	4 Dec.'67	4	1-4 Way	4-8"	1
SR 1434(W.Railroad) & James St. (60)	Pitt	20 & 35 MPH	4 Dec.'67	4	1-4 Way	4-8"	1
SR 1225 (S.Fields) & W. Pine (61)	Pitt	45 & 35 MPH	26 Jul.'66	4	1-4 Way	4-8"	0
U.S. 264 (Dickinson) & Watauga (62)	Pitt	35 MPH	9 Feb.'68	4	2-3 Way	6-8"	0
N.C. 222-N.C. 43 (18)	Pitt	35 MPH	30 May '68	4	1-1 Way 1-3 Way	4-8"	2
U.S. 70 & SR 1001 - SR 1256 (15)	Craven	45 & 35 MPH	2 Nov.'67	4	2-1 Way 1-2 Way	4-8"	2
U.S. 258,N.C. 58 & N.C.91(bouncing ball) (59)	Green	55 MPH	10 Sept'64	4	2-2 Way	4-12"	2

Figure 1. Typical inventory sheet.

Equivalent property damage only (EPDO) is a term used to group all accidents in the periods and reflects both the number of accidents and severity. The following equation for determining EPDO, which is based on direct cost of accidents by severity class, is the same as the one used by the Traffic Engineering Department of the North Carolina State Highway Commission:

$$\text{EPDO} = 5.8\,(N_F + N_A) + 2\,(N_B + N_C) + N_D$$

where

N_F = number of fatal accidents,
N_A = number of type A accidents,
N_B = number of type B accidents,
N_C = number of type C accidents, and
N_D = number of property damage only accidents.

Each individual accident would have only one classification, and this would be the most severe. For example, an accident may have had one type A, one type B, and two type C injuries, in which case the accident would have been classified as type A. The injury severity classification was determined from the accident forms. The injury categories used follow the Manual of Uniform Definition of Motor Vehicle Accidents in which type A is a bleeding wound, distorted member, or any condition that required victim to be carried from the scene; type B is other visible injury such as bruise, abrasion, swelling, or limping or other painful movements; and type C is complaint of pain without visible signs of injury, or momentary unconsciousness.

The accident rate is normally defined as the number of accidents divided by the exposure rate:

$$\text{Accident rate} = \frac{\text{number of accidents}}{\text{ER}}$$

The EPDO rate is simply EPDO divided by the exposure rate, which in the case of intersections is in terms of million vehicles entering the intersection per year. This is expressed as

$$\text{EPDO rate} = \frac{\text{EPDO}}{\text{ER}}$$

The severity index, SI, does not have any real significance except to indicate the average severity at that particular intersection location. The SI is the number of EPDO accidents divided by the total number of accidents and is expressed as

$$\text{SI} = \frac{\text{EPDO}}{N_T}$$

where N_T is the total number of accidents before or after the improvement. It should be noted that, if considered alone, the severity index could lead to a false conclusion. For example, a single type A accident in the after period would reflect a high SI, but there could be an accompanying decrease in the accident rate and the EPDO rate.

Summation of Data

Upon completion of an accident data sheet for each signal and flasher site, five summary sheets were prepared for each traffic control device. The signal summary sheets include one comprised of all 19 sites (Fig. 2), four-leg intersections (Fig. 3), three-leg intersections (Fig. 4), divided highway intersections (Fig. 5), and undivided highway intersections (Fig. 6). The summary sheets for flashing beacons (Figs. 7 through 11) were essentially the same as those for signals except that there were separate sheets for channelized and nonchannelized intersections in place of divided and undivided highway intersections.

SITE LOCATION Signal Summary | DIVISION

SITE NUMBER All 19 Sites | COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	7	4	11	37	19	55	16	127	91	26	13	6	2	138 (38)	111.79	291.	2.11
Before	Rate	.06	.04	.10	.33	.17	.49	.14	1.14	.82	.23	.12	.05	.02	1.23 (1.02)		2.61	
After	No. of Accidents	5	6	11	45	47	15	27	134	107	18	11	9	1	145 (40)	111.79	251.2	1.76
After	Rate	.04	.05	.10	.40	.42	.13	.24	1.20	.96	.16	.10	.08	.01	1.31 (1.07)		2.30	
	% Rate Change	-33	+25	0	+21	+147	-73	+71	+5	+17	-30	-17	+50	-50	+7 (+5)		-12	-17

Installation Date | Speed Limit

Years of Experience 23 | Major ADT

Number of Approaches | Minor ADT

Channelized or
Non Channelized

Figure 2.

SITE LOCATION Signal Summary | DIVISION

SITE NUMBER 4 Leg Intersections (12 Sites) | COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	4	2	6	14	13	55	6	88	64	17	8	4	1	94 (25)	59.56	192.40	2.05
Before	Rate	.07	.03	.10	.24	.22	.92	.10	1.48	1.07	.29	.13	.07	.02	1.68 (1.26)		3.23	
After	No. of Accidents	3	2	5	26	27	14	15	82	65	13	3	5	1	87 (26)	59.56	162.2	1.86
After	Rate	.05	.03	.08	.44	.45	.24	.25	1.38	1.09	.22	.05	.08	.02	1.46 (1.31)		2.72	
	% Rate Change	-29	0	-20	+42	+105	-74	+140	-7	+2	-24	-62	+14	0	-13 (+4)		-16	-9

Installation Date | Speed Limit

Years of Experience 13 | Major ADT

Number of Approaches | Minor ADT

Channelized or
Non Channelized

Figure 3.

SITE LOCATION Signal Summary DIVISION

SITE NUMBER 3 Leg Intersections (7 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	3	2	5	23	6	0	10	39	27	9	5	2	1	44 (13)	52.23	99	2.25
	Rate	.06	.04	.10	.44	.11	0	.19	.75	.52	.17	.10	.04	.02	.84 (.75)		1.90	
After	No. of Accidents	2	4	6	19	20	2	12	53	42	5	8	4	0	59 (14)	52.23	95	1.61
	Rate	.04	.08	.11	.36	.38	.04	.23	1.01	.80	.10	.15	.08	0	1.13 (.80)		1.82	
	% Rate Change	-33	+100	+10	-18	+245	—	+21	+35	+54	-41	+50	+100	-100	+34 (+7)		-4	-28

Installation Date

Speed Limit

Years of Experience 10

Major ADT

Number of Approaches

Minor ADT

Channelized or Non Channelized

Figure 4.

SITE LOCATION Signal Summary DIVISION

SITE NUMBER Divided (9 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	5	4	9	23	12	30	13	78	57	16	9	4	1	87 (29)	75.98	181.6	2.09
	Rate	.07	.05	.12	.30	.16	.39	.17	1.03	.75	.21	.12	.05	.01	1.15 (1.15)		2.39	
After	No. of Accidents	1	5	6	34	35	14	21	104	81	13	10	5	1	110 (32)	75.98	192.2	1.75
	Rate	.01	.07	.08	.45	.46	.18	.28	1.37	1.07	.17	.13	.07	.01	1.45 (1.26)		2.53	
	% Rate Change	-86	+40	-33	+50	+188	-54	+65	+33	+43	-19	+8	+40	0	+26 (+10)		+6	-16

Installation Date

Speed Limit

Years of Experience 12

Major ADT

Number of Approaches

Minor ADT

Channelized or Non Channelized

Figure 5.

SITE LOCATION Signal Summary DIVISION

SITE NUMBER Undivided (10 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	2	0	2	14	7	25	3	49	34	10	4	2	1	51 (9)	35.81	109.80	2.15
	Rate	.06	0	.06	.39	.20	.70	.08	1.37	.95	.28	.11	.06	.03	1.42 (.75)		3.07	
After	No. of Accidents	4	1	5	11	12	2	6	31	26	5	1	4	0	36 (8)	35.81	65.0	1.81
	Rate	.11	.03	.14	.31	.34	.06	.17	.87	.73	.14	.03	.11	0	1.01 (.67)		1.82	
	% Rate Change	+83	—	+133	-21	+70	-91	+113	-37	-23	-50	-73	+83	-100	-79 (-11)		-41	-16

Installation Date Speed Limit

Years of Experience 11 Major ADT

Number of Approaches Minor ADT

Channelized or Non Channelized

Figure 6.

SITE LOCATION Flasher Summary DIVISION

SITE NUMBER All 14 sites COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	13	0	13	8	18	38	12	76	51	20	12	4	2	89 (20)	42.76	210.6	2.37
	Rate	.28	0	.28	.19	.42	.89	.28	1.78	1.19	.47	.28	.09	.05	2.08 (1.40)		4.93	
After	No. of Accidents	5	0	5	7	13	30	10	60	50	7	4	3	1	65 (11)	42.76	110.4	1.70
	Rate	.12	0	.12	.16	.28	.70	.23	1.40	1.17	.16	.09	.07	.02	1.52 (.77)		2.58	
	% Rate Change	-62	0	-62	-13	-33	-21	-17	-21	-2	-65	-67	-25	-50	-27 (-45)		-48	-28

Installation Date Speed Limit

Years of Experience 22 Major ADT

Number of Approaches Minor ADT

Channelized or Non Channelized

Figure 7.

SITE LOCATION Flasher Summary DIVISION

SITE NUMBER 4 Leg Intersections (11 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	5	0	5	8	14	37	8	67	44	12	11	3	2	72 (10)	37.73	153.2	2.12
Before	Rate	.13	0	.13	.21	.37	.98	.21	1.78	1.17	.32	.29	.08	.05	1.91 (.80)		4.06	
After	No. of Accidents	2	0	2	6	13	30	8	57	45	7	3	3	1	59 (9)	37.73	103.4	1.75
After	Rate	.05	0	.05	.16	.35	.80	.21	1.51	1.19	.19	.08	.08	.03	1.56 (.72)		2.74	
	% Rate Change	-62	0	-62	-24	-5	-18	0	-15	+2	-41	-72	0	-50	-18 (-10)		-33	-18

Installation Date

Speed Limit

Years of Experience 18

Major ADT

Number of Approaches

Minor ADT

Channelized or
Non Channelized

Figure 8.

SITE LOCATION Flasher Summary DIVISION

SITE NUMBER 3 Leg Intersections (3 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	8	0	8	0	4	1	4	9	7	8	1	1	0	17 (10)	5.03	57.40	3.38
Before	Rate	1.59	0	1.59	0	.80	.20	.80	1.79	1.39	1.59	.20	.20	0	3.38 (5.96)		11.41	
After	No. of Accidents	3	0	3	1	0	0	2	3	5	0	1	0	0	6 (2)	5.03	7.00	1.17
After	Rate	.60	0	.60	.20	0	0	.40	.60	.99	0	.20	0	0	1.19 (1.19)		1.39	
	% Rate Change	-62	0	-62	-	-100	-100	-50	-67	-29	-100	0	-100	0	-65 (-80)		-88	-65

Installation Date

Speed Limit

Years of Experience 4

Major ADT

Number of Approaches

Minor ADT

Channelized or
Non Channelized

Figure 9.

SITE LOCATION Flasher Summary DIVISION

SITE NUMBER Channelized Intersections (10 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	11	0	11	3	11	30	9	53	33	18	9	2	2	64 (15)	28.96	171.0	2.67
Before	Rate	.38	0	.38	.10	.38	1.04	.31	1.83	1.14	.62	.31	.07	.07	2.21 (1.55)		5.90	
After	No. of Accidents	4	0	4	5	4	15	6	30	27	3	2	1	1	34 (7)	28.96	56.2	1.65
After	Rate	.14	0	.14	.17	.14	.52	.21	1.04	.93	.10	.07	.03	.03	1.17 (.73)		1.94	
	% Rate Change	-63	0	-63	+70	-63	-50	-32	-43	-18	-84	-77	-50	-50	-47 (-53)		-67	-38

Installation Date Speed Limit

Years of Experience 16 Major ADT

Number of Approaches Minor ADT

Channelized or Non Channelized Channelized

Figure 10.

SITE LOCATION Flasher Summary DIVISION

SITE NUMBER Non Channelized Intersections (4 Sites) COUNTY

		Accident Type								Severity								
		Single Vehicle			Multiple Vehicle						Injury							
		Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	P.D.O.	A	B	C	Fatal	Total Accidents (Night Accidents)	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Before	No. of Accidents	2	0	2	5	7	8	3	23	18	2	3	2	0	25 (5)	13.80	39.60	1.58
Before	Rate	.14	0	.14	.36	.51	.58	.22	1.67	1.30	.14	.22	.14	0	1.81 (1.09)		2.87	
After	No. of Accidents	1	0	1	2	9	15	4	30	23	4	2	2	0	31 (4)	13.80	54.20	1.75
After	Rate	.07	0	.07	.4	.65	1.09	.29	2.17	1.67	.29	.14	.14	0	2.25 (.87)		3.93	
	% Rate Change	-50	0	-50	+1	+3	+88	+32	+30	+30	+100	-36	0	0	+24 (-20)		+37	+11

Installation Date Speed Limit

Years of Experience 6 Major ADT

Number of Approaches Minor ADT

Channelized or Non Channelized Non Channelized

Figure 11.

Summation of the individual site data in the various inventory sheets facilitates comparison of consequences from the installation of a particular control device. A positive rate of change indicates a percentage increase in that particular consequence in the after period; likewise, a negative value reflects a percentage decrease in the after period. Upon evaluation of these statistics a better understanding of the effects of a particular signal or flashing beacon on accident experience, severity, and equivalent property damage in relation to the intersection configuration can be achieved.

STATISTICAL EVALUATION OF DATA

Statistical Analysis

Traditionally, the accepted procedure of statistical analysis for before-and-after studies has been the chi-square distribution test. This is normally applied to the total number of accidents that are evenly divided between the before and after periods (expected frequencies) and compared to the actual number of accidents (observed frequencies) in respective time frames. The null hypothesis is that the expected frequencies are not significantly different from the observed frequencies. It was the opinion of the researchers that the chi-square test does not reflect the true significance of the investigation. In this type of statistical analysis, it is assumed that all accident types are homogeneous in regard to severity and EPDO. In reality, it is possible to have a significant increase in accidents in the after period accompanied by a reduction in EPDO accidents and rates and in SI. In the past, EPDO accidents and rates and SI were not statistically evaluated because they were based on weighted values and did not represent frequencies. The EPDO rate represents the total number of accidents, severity, and exposure and is, therefore, a more reliable and significant indicator of accident consequences than is the accident rate, which is only a measure of the total number of accidents.

The procedure adopted to statistically evaluate the EPDO rates is the Student "t" test for paired data, such as before-and-after tests. The null hypothesis is that the before and after EPDO data are essentially equal, and the difference in EPDO rates is zero. This is tested against the alternate hypothesis that there is a significant reduction in EPDO rates in the after period (one-tail test). This method can be best described by the following example.

Basic form for Student "t" test:

$$t = \frac{\bar{x} - \mu_0}{s/\sqrt{n}}$$

where

t = normal deviate to be compared to "t" table,
$\bar{x}$ = mean of given sample,
μ_0 = hypothetical mean,
s = standard deviation of sample, and
n = number in sample.

Form for paired data (before-and-after studies):

$$t = \frac{\bar{d} - \delta}{s_d/\sqrt{n}}$$

where

$\bar{d}$ = average difference of before and after EPDO rates,
δ = hypothetical difference for EPDO rates, and
s_d = standard deviation for difference.

Site Location No.	Before EPDO Rate	After EPDO Rate	Difference d_i
1	R_{B1}	R_{A1}	$(R_{B1} - R_{A1}) = d_1$
2	R_{B2}	R_{A2}	$(R_{B2} - R_{A2}) = d_2$
⋮	⋮	⋮	⋮
n	R_{Bn}	R_{An}	$(R_{Bn} - R_{An}) = d_n$

1. Estimate δ by $\bar{d}$, where $\bar{d} = \frac{\Sigma di}{n}$.
2. Estimate σ_d by S_d, where $S_d = \sqrt{\frac{\Sigma(di - \bar{d})^2}{n - 1}}$.

Null hypothesis: H_O: $\delta = 0$
H_A: $\delta > 0$

If the calculated t value is greater than the t distribution values for the various α (confidence) levels, the null hypothesis can be rejected. This suggests that the EPDO rate is significantly reduced in the after period or in other words that installation of the traffic control device had a positive effect in reducing EPDO rates.

Data Aggregation and Evaluation

All data were aggregated into two groups—one for signals and one for flashers. The analytical procedure described in the previous section was utilized on both sets of data. By aggregating the data, the accident experience, or population, increases, and the test becomes more realistic. The individual sites cannot be adequately evaluated because of the small amount of accident experience at each site. A small fluctuation in accidents at a single site location can cause a very significant change in the statistical analysis because of the low initial size and final sample size. Aggregation of the accident experience into the two major groupings greatly increases the sample size for the paired Student "t" test and the results become more meaningful.

The two sets of data were further divided into four subsets for analysis of three-leg intersections, four-leg intersections, channelized and nonchannelized intersections in the case of flashers, and divided and undivided highways in the case of signals. The statistical procedure for testing each subset is identical to the method used on the collective evaluation of the signals and flashers.

TEST FINDINGS

It was determined that there is an association between the installation of flashing beacons and a reduction in EPDO rates. A similar statement cannot be made concerning signals because the results of the analysis were not highly significant. The percentage rates of changes relating signals and flashers to accident type, severity, total accidents, and EPDO are given in Table 1.

In the signal analysis, it was determined that the only significant EPDO rate reduction after the installation of the signal was in the category of undivided highway intersections. There was one particular site (No. 135) that had experienced a high increase in EPDO rate in the after period and that could not be excluded because it met the selection criteria previously established. With all signal sites included, the correlation between signals at four-leg, three-leg, divided highway, and all intersection categories and EPDO rates was found to be nonsignificant. However, when site 135 was excluded from the analysis, all signal intersections and four-leg intersections were determined to be significant in the reduction of EPDO rates at a confidence level of 0.01 and 0.005 respectively. Such an exclusion emphasizes the importance of thoroughly investigating

TABLE 1

SUMMARY RELATING RATE OF CHANGE IN PERCENTAGE

Intersection Category	Accident Type								Severity					Total Accidents	SI	EPDO
	Single Vehicle			Multiple Vehicle					Fatal	Injury Type			PDO			
	Ran-Off-Road	Other	Subtotal	Left Turn	Rear End	Crossing	Other	Subtotal		A	B	C				
Signals																
Total	-33	+25	0	+21	+147	-73	+71	+5	-50	-30	-17	+50	+17	+7	-17	-12[a]
4-Leg	-29	0	-20	+42	+105	-74	+140	-7	0	-24	-62	+14	+2	-13	-9	-16[b]
3-Leg	-33	+100	+10	-18	+245	-	+21	+35	-100	-41	+50	+100	+54	+34	-28	-4[c]
Divided	-86	+40	-33	+50	+188	-54	+65	+33	0	-19	+8	+40	+43	+26	-16	+6[c]
Undivided	+83	-	+133	-21	+70	-91	+113	-37	-100	-50	-73	+83	-23	-79	-16	-41[d]
Flashers																
Total	-62	0	-62	-13	-33	-21	-17	-21	-50	-65	-67	-25	-2	-27	-28	-48[e]
4-Leg	-62	0	-62	-24	-5	-18	0	-15	-50	-41	-72	0	+2	-18	-18	-33[f]
3-Leg	-62	0	-62	-	-100	-100	-50	-67	0	-100	0	-100	-29	-65	-65	-88[d]
Channelized	-63	0	-63	+70	-63	-50	-32	-43	-50	-84	-77	-50	-18	-47	-38	-67[g]
Non-channelized	-50	0	-50	+1	+3	+88	+32	+30	0	+100	-36	0	+30	+24	+11	+37[c]

Note: Minus sign (–) indicates that there were no accidents of this type or severity in the before period and there was at least one in the after period.

[a]Not significant (excluding site No. 135, significant at α = 0.01).
[b]Not significant (excluding site No. 135, significant at α = 0.005).
[c]Not significant.
[d]Significant at α = 0.025.
[e]Significant at α = 0.01.
[f]Significant at α = 0.05.
[g]Significant at α = 0.005.

all sites applicable to the study in relation to established criteria for a nearly homogeneous grouping. This is very critical in that the results of the analysis can be altered by a single irregular site.

It was noted that there was a very sizeable increase in rear-end, left-turn, and "other" accidents after the installation of the signal device. Single-vehicle ran-off-the-road accidents (except in the case of divided highways) and multiple-vehicle crossing accidents tended to decrease after the installation of signals. Other consequences of importance included a decrease in fatal, type A, and type B accidents in the after period. This was reflected in the decreased SI and the EPDO rates, although the decrease is not statistically significant at a confidence level of 0.01. Type B, type C, and property damage only (PDO) accidents tended to increase after installation of the signal.

In the comparison of before and after data on flashing beacons, the statistical analysis proved that there was significant reduction in EPDO rates on the aggregated sites and the four-leg, three-leg, and channelized intersections. The EPDO rate increase in the after period on nonchannelized intersections was not significant. It was noted that single-vehicle ran-off-the-road accidents decreased and "others" indicated no change. In the multiple-vehicle accident consequences, left-turn accidents appeared to decrease in the after period except on channelized highway intersections. Rear-end collisions tended to decrease, as did crossing and "other" accidents (except at nonchannelized intersections) in the multiple-vehicle category. The severity comparison indicated a reduction or, at least, no change in the fatal group and a reduction in type A (except at nonchannelized intersections), type B (no change at three-leg intersections), and type C (no change at three-leg and nonchannelized intersections). The PDO category was mixed, with increases in four-leg and nonchannelized intersections and reductions in the other consequences. The SI for all groups decreased except at nonchannelized intersections.

As previously stated, the reduction in EPDO rates was significant at all intersection groupings except nonchannelized intersections. The level of significance was 0.01 for the grouping of all sites, 0.05 for four-leg intersections, 0.025 for three-leg intersections, and 0.005, or most significant, for channelized intersections.

CONCLUSIONS

The relationship between the installation of a flashing beacon and EPDO rate reduction was found to be statistically significant at the 1 percent confidence level. The relationship between the installation of signals and EPDO rate reduction was not statistically significant except for undivided highway intersections.

Specifically, within the limitations of the definition of selected sites for this study, the following conclusions can serve as guidelines for developing warrants for signals and flashing beacons:

1. The effect of signal installations on accident experience cannot be significantly predetermined. This applies to all signal installations after aggregation, as well as installations at four-leg, three-leg, and divided highway intersections.
2. On the average, installation of a signal at undivided highway intersections will reduce EPDO rates. Probability of observed results due to chance alone is 0.025.
3. On the average, installation of a flashing beacon at any typical site will reduce EPDO rates. Probability of observed results due to chance alone is 0.01.
4. On the average, installation of flashing beacons at four-leg intersections will reduce EPDO rates. Probability of observed results due to chance alone is 0.05.
5. On the average, installation of a flashing beacon at three-leg intersections will reduce EPDO rates. Probability of observed results due to chance alone is 0.025.
6. On the average, installation of flashing beacons at channelized intersections will reduce EPDO rates. Probability of observed results due to chance alone is 0.005.
7. The effect of installation of a flashing beacon on reducing EPDO consequences at nonchannelized intersections cannot be significantly predetermined.
8. Use of the Student "t" test for paired data is an acceptable and effective method for evaluating before and after tests on aggregated data.
9. The comparison of EPDO rates provides a more significant measure of the consequences of accidents before and after installation than other measures used in previous studies.

The future use of the methodology and analytical procedure developed in this study should be useful in the evaluation of other types of traffic control improvements on low-volume highways. Before making an improvement at an intersection, evaluation of prior accident experience by this methodology should lead to a more efficient and effective use of traffic control devices. It appears that future investigations of this type will enjoy the luxury of longer exposure periods for accident evaluation, more accurate and complete accident data from computer printouts, and a tested analytical technique of considerable promise.

ACKNOWLEDGMENTS

This research was conducted by the Department of Civil Engineering of the North Carolina State University in cooperation with the North Carolina State Highway Commission and the U.S. Bureau of Public Roads.

The opinions, findings, and conclusions expressed are those of the authors and not necessarily those of the state or the Bureau of Public Roads.

REFERENCES

1. Manual on Uniform Traffic Control Devices for Streets and Highways. U.S. Bureau of Public Roads, 1961, 333 pp.
2. Cribbins, P.D., and Brothers, B. T. Scheduling Improvements for High-Accident Locations by Means of Investment Return Analysis. Univ. of North Carolina, Highway Safety Research Center, Project ERD-260, Final Rept., 1967.

Some Effects of Lateral Sign Displacement

GERHART F. KING, Cutler-Hammer, Inc., Deer Park, Long Island

The effect of increasing the lateral offset of highway signs on the legibility of these signs was analyzed. An approach based on highway geometrics and human vision was employed. Highway curves as well as tangent sections were analyzed. It was found that larger letter sizes were indicated with increased offset distances and that certain combinations of horizontal curvature and offset would result in the sign falling entirely outside the normal field of vision. The effect of the lateral offset on night legibility of signs, when the signs are illuminated by headlight only, was also investigated. A computer program was developed to investigate the relationship of the five pertinent variables: degree of curvature, approach speed, maximum divergence angle, message content, and letter size.

• RECENT EFFORTS for safer highway travel include attempts to make the highway more forgiving especially by creating clear recovery areas adjoining the traveled way. This has resulted in increasing the lateral distance between the edge of the roadway and the location of major guide signs. This paper analytically investigates the effect of lateral sign displacement on legibility.

The basic approach used was first developed by Mitchell and Forbes (1). A driver is assumed to start reading a sign when it first becomes legible and to finish reading it before he reaches a point at which the sign falls outside the normal field of vision. The point of first legibility is a function of letter height, and the field of vision is defined in terms of the maximum divergence angle.

SIGNS ON TANGENT

In Figure 1, θ is the acceptable maximum divergence angle, and B is the point at which the driver should have finished reading the sign message. If t is the time in sec necessary to read the sign, the vehicle, traveling along the path MN, will traverse tV ft during that time, where V is the velocity in fps. The sign must, therefore, be legible at point C, a distance t ft upstream from B. For a three-lane highway with 12-ft lanes and with the driver's eye position assumed to be two-thirds of the lane width to the left, point B is $(S + 32 + W/2) \cot \theta$ ft upstream from point A. This is the location of the sign where S is the lateral displacement of the edge of the sign in ft, measured normal to the path of travel. The width of the sign in ft is W. The distance at which the sign must become legible is CD, which can be expressed for tangent approaches as

$$CD = \sqrt{[tV + (S + 32 + W/2) \cot \theta]^2 + (S + 32 + W/2)^2} \quad (1)$$

If L is the reading distance in ft/in. of letter height, the required minimum letter height, H (in in.), becomes

$$H = \sqrt{\frac{[tV + (S + 32 + W/2) \cot \theta]^2 + (S + 32 + W/2)^2}{L}} \quad (2)$$

Paper sponsored by Committee on Traffic Control Devices and presented at the 49th Annual Meeting.

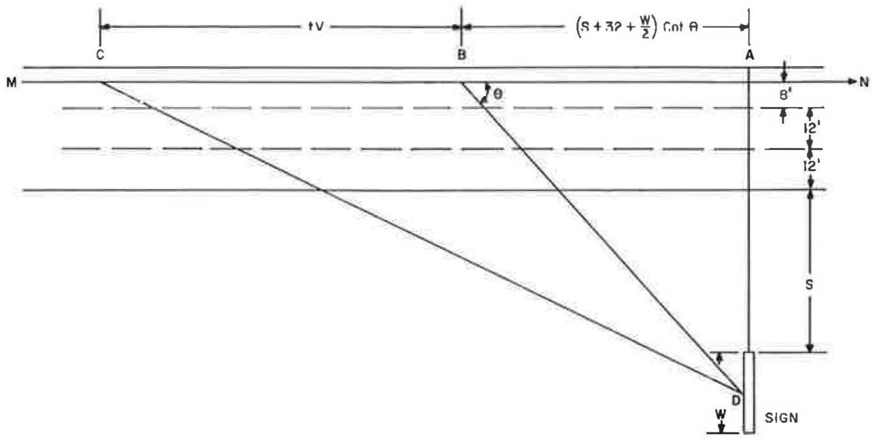


Figure 1. Geometry of sign location and horizontal displacement on a tangent section.

Equation 2 includes five variables. V and S are determined for the specific highway and specific sign position being investigated. The other variables are determined by considerations of human factors.

Mitchell and Forbes derived an expression for t in terms of N, where N is defined as the "number of familiar words on the sign". This expression is derived on the basis of 1-sec glances from the road to the sign and back to the road and the ability to read three familiar words during each glance. Adding a safety factor of 1 sec, this expression is

$$t = \frac{N}{3} + 1.0 \tag{3a}$$

Later work at the British Road Research Laboratory (RRL) reported by Moore and Christie (2) indicated that a more appropriate formula, designed to give the driver two chances to read the sign and to include the case where the name searched for is the last to be read, is

$$t = \frac{2N}{3} \tag{3b}$$

Continuing work at the RRL (3) has resulted in the selection of the following formula for determining letter sizes in preparing the British sign standards

$$t = 0.31N + 1.94 \tag{3c}$$

Equations 3a, 3b, and 3c are shown in Figure 2.

In Mitchell and Forbes' original work, a value of $\theta = 10$ deg, based on psychological considerations, was assumed. This value is now generally accepted in Great Britain (3) as well as in the United States (4). In Germany, on the other hand, Heller uses a value of $\theta = 15$ deg (5). He gives a curve relating t and N, but, because N is defined as syllables, the results cannot be compared with the equations given here.

L is defined as legibility in ft/in. of letter height assuming a straight-line relationship. Mitchell and Forbes use a value of 50 ft/in. British practice is the same for lower-case letters. For upper-case letters, a value of 37.5 ft/in. is advocated. Moore and Christie point out, however, that for the minimum legal vision requirements in the United Kingdom L should equal 21 ft/in. When the subtended angle definition of visual acuity is used (6), 20/20 vision results in L equaling 57 ft/in. For 20/70 vision (the

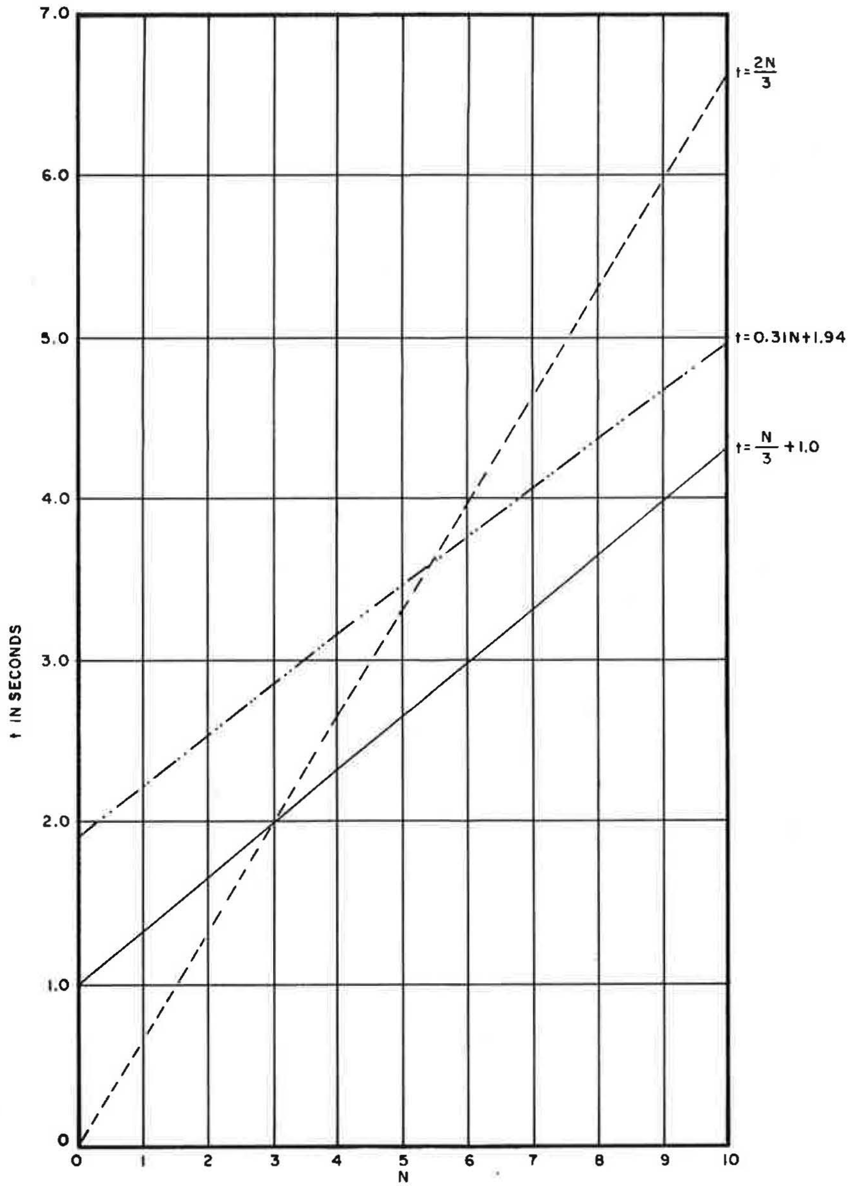


Figure 2. Reading time versus message composition.

Figure 3. Advance guide sign.

TABLE 1

LETTER HEIGHTS

L, ft/in.	H, in.	Notes
57	15.7	20/20 vision
50	17.8	"Rule of thumb"
37.5	23.8	U.K. for lower case
28	31.9	20/40 vision
21	42.5	U.K. minimum visual acuity
16	55.8	20/70 vision

legal minimum visual requirement for obtaining a driver's license in three states), L equals 16 ft/in. These figures represent daylight or equivalent illumination. Allen et al. (7) show that legibility distances decrease markedly when sign luminance drops below 20 ft-L.

A special AASHO committee (8) has recommended that a clear recovery area 30 ft from the edge of the traveled way be established. The maximum speed limit on any section of the Interstate System is 80 mph or 117 fps. Using $\theta = 10$ deg, $S = 30$, and $t = 0.31N + 1.94$, Eq. 2 can be written as

$$LH = \sqrt{[(0.31N + 1.94)\, 117.0 + (0.5W + 62) \cot 10]^2 + (0.5W + 62)^2} \qquad (4a)$$

The term LH permits computation of letter height under various assumptions for L. The value of this term can be seen to be a function of message content, N, and sign size, W. The sign size, in turn, is a function of letter height and message content because a change in either letter height or message length will lead to an increased sign panel size.

If the definition of N is expanded to include numerals and familiar shapes and symbols such as shields and arrows, its value can be determined for actual signs. For instance, the advance guide sign (Fig. 3) shown in Figure 7 of the Interstate Manual (9) has a value of N equals 7. When the specified letter sizes and common spacing rules are used, this sign has a W of about 20 ft. Equation 4a becomes

$$LH = \sqrt{[(2.17 + 1.94)\, 117.0 + (10 + 62)\, 5.671]^2 + (10 + 62)^2} \qquad (4b)$$
$$= 892$$

TABLE 2

LETTER HEIGHT REQUIRED FOR VARIOUS W AND N

W, ft	N												
	3	4	5	6	7	8	9	10	11	12	13	14	15
10	15	16	16	17	18	18	19	20	20	21	22	23	23
15	15	16	16	17	18	19	19	20	21	21	22	23	23
20	15	16	17	18	18	19	20	20	21	22	22	23	24
25	16	16	17	18	18	19	20	21	21	22	23	23	24
30	16	17	17	18	19	19	20	21	22	22	23	24	24
35	16	17	18	18	19	20	20	21	22	23	23	24	25
40	17	17	18	19	19	20	21	21	22	23	24	24	25
45	17	17	18	19	20	20	21	22	22	23	24	24	25
50	17	18	19	19	20	21	21	22	23	23	24	25	26
55	17	18	19	19	20	21	22	22	23	24	24	25	26
60	18	18	19	20	21	21	22	23	23	24	25	25	26

Note: Required letter height (in.) for L = 50 ft/in.; V = 80 mph; t = 0.31N + 1.94; θ = 10; and S = 30 ft.

The required letter heights for various assumptions as to the value of L discussed previously are given in Table 1.

The manual specifies 18-in. numerals and 12-in. capitals for this type of sign. Table 1 shows that, although the numerals are adequate for L = 50, the letters are not, and they should be increased. It is obvious that an increase in letter height would result in a larger sign panel and would require recomputation of the table. Because there is no easy formula relationship between letter height and message length, the required letter height must be determined by successive approximations.

Table 2 gives the letter height required for various combinations of W and N under the assumptions previously stated and for L = 50. Selected values are shown in Figure 4. Using Mitchell and Forbes' equation instead of the RRL formula would reduce each value in Table 2 by about 2 in.

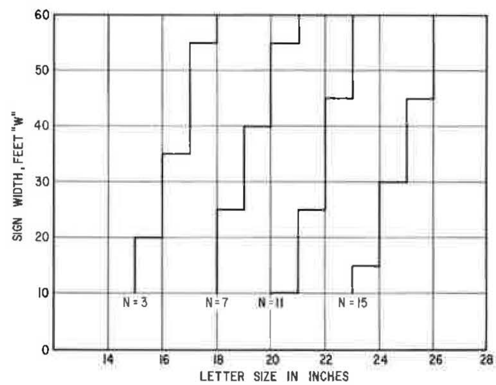


Figure 4. Letter size required for various W and N.

From Figure 1, it can be seen that θ was measured to the center of the sign. For small values of W, this method has no appreciable effect. For larger values of W, however, the right side of the sign may be considerably outside the field of vision. To in-

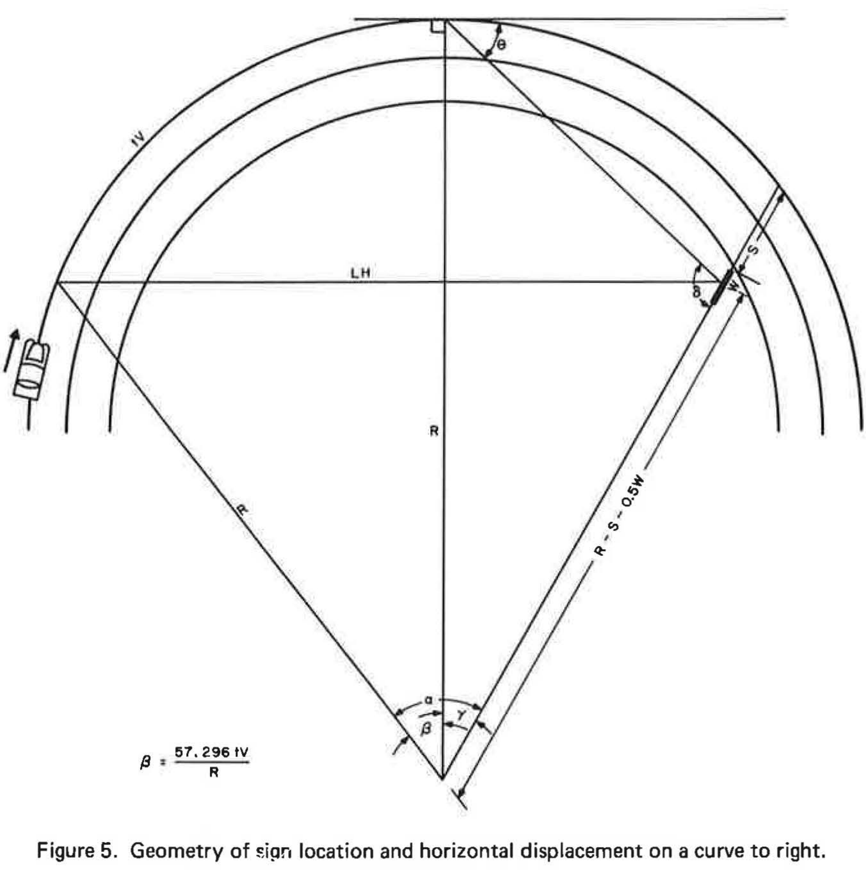


Figure 5. Geometry of sign location and horizontal displacement on a curve to right.

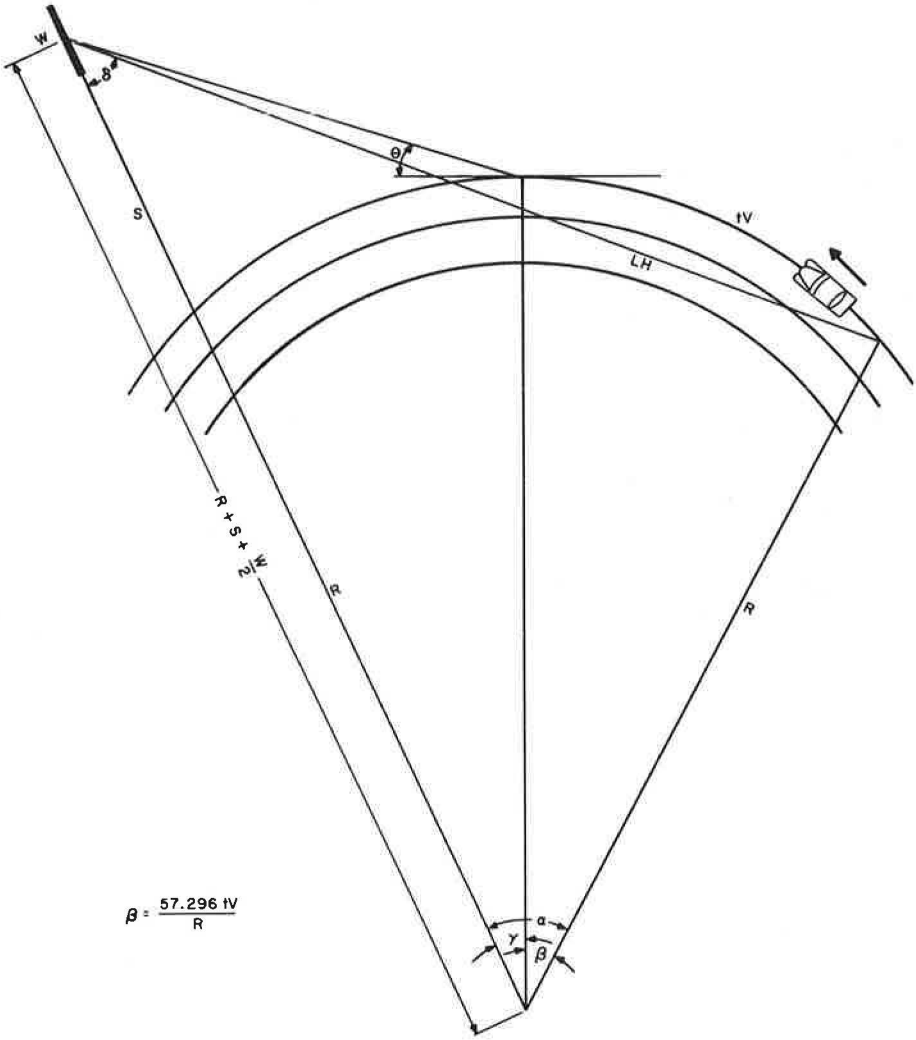


Figure 6. Geometry of sign location and horizontal displacement on a curve to left.

clude the entire sign in the field of vision, the term 0.5W + 62 in Eq. 4a must be changed to W + 62. This will increase each letter height given in Table 2 by about 0.057W in.

SIGNS ON CURVES

The discussion has been limited to tangent highway sections, but this same approach can be used for curves. Figures 5 and 6 show the geometry of sign location for curves to the right and to the left respectively. Using simple geometric and trigonometric relationships, we can show the corresponding equations.

For curves to the right,

$$\alpha = 57.296 \frac{tV}{R} + 90 - \delta + \theta \tag{5a}$$

$$LH = \sqrt{R^2 + (R - S - 0.5W)^2 - 2R\,(R - S - 0.5W)\cos\alpha} \tag{5b}$$

For curves to the left,

$$\alpha = 57.296 \frac{tV}{R} + 90 - \delta - \theta \tag{6a}$$

$$LH = \sqrt{R^2 + (R + S + 0.5W)^2 - 2R\,(R + S + 0.5W)\cos\alpha} \tag{6b}$$

Before some of the results computed from this relationship are discussed, it should be pointed out that, in the case of curved sections, two limiting cases that did not apply in the case of tangent sections must be considered. The first of these limitations occurs because the geometry of the sign location may be such that no point on the curve exists from which the deflection angle measured to the sign is θ deg or less. This will occur when $S + 0.5W$ is greater than $R(1 - \cos\theta)$. Even when this condition does not occur, it is possible for the divergence angle to exceed its maximum allowable value of θ at some point in the reading distance tV.

Table 3 gives the maximum offset ($S + 0.5W$) for various degrees of curvature that will result in at least one point on the curve having a deflection angle of 10 deg or less. For vehicles traveling in the left lane of two- or three-lane highways and with an offset of 30 ft, all curves to the right with a degree of curvature of 1.5 deg or more will result in signs being completely out of the normal field of vision. The relationship is shown graphically in Figure 7.

Figure 8 shows the effect of changing the permissible value of θ. For each value of θ and for each degree of curvature, the maximum offset is given in Table 4. It can be seen that, roughly, for each increase of 1 deg in the maximum permissible value of θ, the maximum degree of curvature is increased by ½ deg.

Tables 3 and 4 have been computed for positive values (curves to the right) only; the first limitation, that no point on the curve shows a divergence angle of θ or less, does not apply to curves to the left. However, the second limitation, that some point within the reading distance must have a divergence angle of more than θ, can apply.

The length of the reading distance, tV, is a function of reading time and velocity. Using standard sign 7 of the Interstate Manual, again, N = 7 and W = 20 ft. Table 5 gives and Figure 9 shows the effect on letter size of varying the velocity. The effect of the two limiting factors is evident.

TABLE 3

MAXIMUM OFFSET (S+0.5W) FOR VARIOUS DEGREES OF CURVATURE

Degree of Curvature[a]	Maximum Offset	Degree of Curvature[a]	Maximum Offset
0.1	870	2.0	43
0.2	435	2.1	41
0.3	290	2.2	39
0.4	217	2.3	37
0.5	174	2.4	36
0.6	145	2.5	34
0.7	124	2.6	33
0.8	108	2.7	32
0.9	96	3.0	29
1.0	87	3.5	24
1.1	79	4.0	21
1.2	72	4.5	19
1.3	66	5.0	17
1.4	62	6.0	14
1.5	58	7.0	12
1.6	54	8.0	10
1.7	51	9.0	9
1.8	48	10.0	8
1.9	45		

[a]Curves to the right.

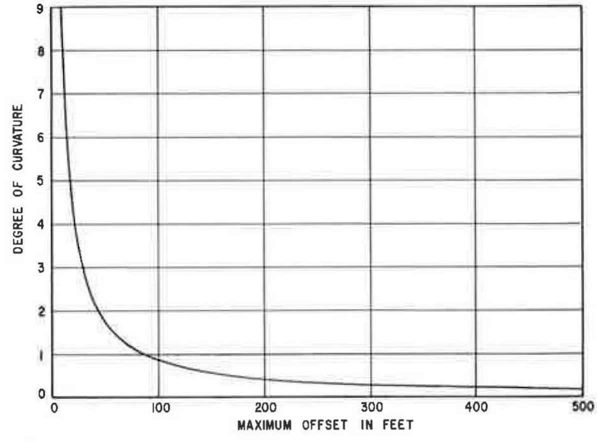


Figure 7. Maximum offset for various degrees of curvature.

TABLE 4

MAXIMUM OFFSET (S+0.5W) AS A FUNCTION OF θ AND DEGREE OF CURVATURE

θ	Degree of Curvature									
	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
5.0	43	21	14	10	8	7	6	5	4	4
6.0	62	31	20	15	12	10	8	7	6	6
7.0	85	42	28	21	17	14	12	10	9	8
8.0	111	55	37	27	22	18	15	13	12	11
9.0	141	70	47	35	28	23	20	17	15	14
10.0	174	87	58	43	34	29	24	21	19	17
11.0	210	105	70	52	42	35	30	26	23	21
12.0	250	125	83	62	50	41	35	31	27	25
13.0	293	146	97	73	58	48	41	36	32	29
14.0	340	170	113	85	68	56	48	42	37	34
15.0	390	195	130	97	78	65	55	48	43	39
16.0	443	221	147	110	88	73	63	55	49	44
17.0	500	250	166	125	100	83	71	62	55	50
18.0	560	280	186	140	112	93	80	70	62	56
19.0	624	312	208	156	124	104	89	78	69	62
20.0	691	345	230	172	138	115	98	86	76	69

A computer program has been written incorporating the relationship of Eqs. 1, 5, and 6. This program permits quick determination of required letter height for any combination of approach speed, sign size, message, and offset.*

EFFECT OF VERTICAL DISPLACEMENT

With a perfectly flat cross section, the top of the sign will be a distance equal to the depth of the sign panel plus its mounting height above the pavement. This vertical component has been ignored in the preceding computations. If, however, the sign is installed

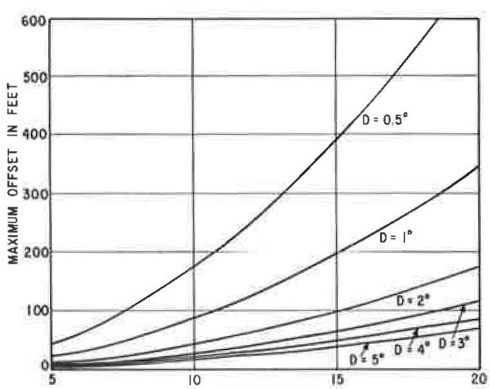


Figure 8. Maximum offset as a function of θ for various degrees of curvature, D.

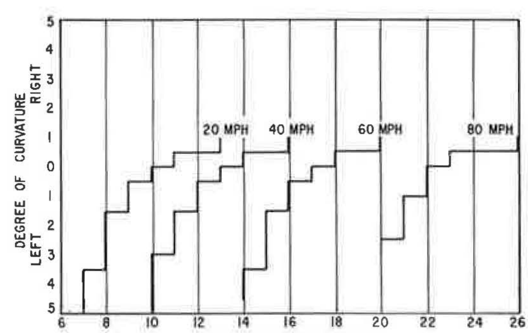


Figure 9. Effect of approach speed on required letter size.

*This paper originally contained a typical computer run relating sign size and message control for various degrees of curvature and for the following parameters: V = 70 mph; S = 30 ft; and vehicle in lane 3. Copies of this computer run may be obtained in Xerox form at cost of reproduction and handling from the Highway Research Board. When ordering, refer to XS-31, Highway Research Record 325.

TABLE 5

EFFECT OF APPROACH SPEED ON REQUIRED LETTER SIZE

Degree of Curvature	Approach Speed, mph												
	20	25	30	35	40	45	50	55	60	65	70	75	80
-5.0	7	8	8	9	10	11	12	13	0[a]	0	0	0	0
-4.5	7	8	8	9	10	11	12	13	14	0	0	0	0
-4.0	7	8	8	9	10	11	12	13	14	15	0	0	0
-3.5	7	8	9	9	10	11	12	13	14	16	17	0	0
-3.0	8	8	9	10	10	11	12	13	15	16	17	18	0
-2.5	8	8	9	10	11	12	12	14	15	16	17	19	20
-2.0	8	9	9	10	11	12	13	14	15	16	18	19	21
-1.5	8	9	10	10	11	12	13	14	15	17	18	19	21
-1.0	9	9	10	11	12	12	13	15	16	17	18	20	21
-0.5	9	10	11	11	12	13	14	15	16	17	19	20	22
0	10	11	11	12	13	14	15	16	17	18	19	21	22
0.5	11	12	12	13	14	15	16	17	18	19	21	22	23
1.0	12	14	15	15	16	17	18	19	20	21	23	24	26
1.5	1[b]	1	1	1	1	1	1	1	1	1	1	1	1
2.0	1	1	1	1	1	1	1	1	1	1	1	1	1
2.5	1	1	1	1	1	1	1	1	1	1	1	1	1
3.0	1	1	1	1	1	1	1	1	1	1	1	1	1
3.5	1	1	1	1	1	1	1	1	1	1	1	1	1
4.0	1	1	1	1	1	1	1	1	1	1	1	1	1
4.5	1	1	1	1	1	1	1	1	1	1	1	1	1
5.0	1	1	1	1	1	1	1	1	1	1	1	1	1

[a]A value of 0 denotes that the divergence angle exceeds 10 deg at point of beginning legibility.
[b]A value of 1 denotes that the divergence angle exceeds 10 deg for all points on the curve.

in a cut section, this vertical displacement can become significant. Figure 10 shows a sign installed on a 4 to 1 side slope beyond a 12-ft shoulder. The elevation of the top of the sign can be computed from the following equation

$$h = D + 2.5 + \frac{S + W - 12}{4} \tag{7}$$

For sign 7 of the Interstate Manual, previously used as an example, D equals 15.5 ft and W equals approximately 20 ft. If S equals 30 ft as previously used, L will be 27.5 ft above the pavement (ignoring any effect of the pavement crown or cross slope). If the driver's eye height is taken as 3.75 ft (10), the effective height of the top of the sign above the driver's eye is 23.75 ft. In the vertical plane, the angle of clear vision, θ_V, is less than it is in the horizontal plane. Matson, Smith, and Hurd state that θ_V is $\frac{1}{2}$ to $\frac{2}{3}$ θ, giving a value of about 6 deg. The same analysis as previously used results in

$$LH = -\sqrt{\left[tV + (h - 3.75)\cot\theta_V\right]^2 + (h - 3.75)^2} \tag{8a}$$

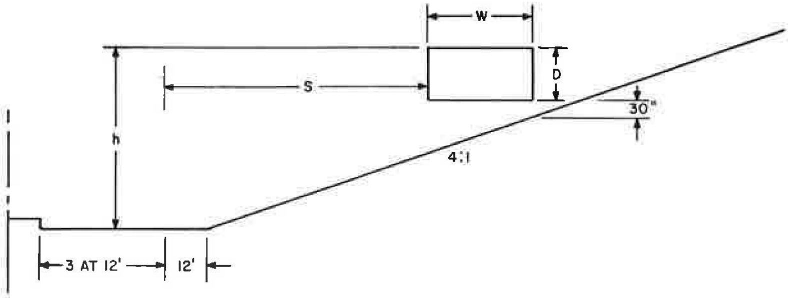


Figure 10. Geometry of sign location and vertical displacement.

Using these values (Eq. 4a) and dropping the second term, we reduce this to

$$LH = (2.17 + 1.94)\,117 + (27.5 - 3.75) \cot 6$$

$$= 707 \qquad (8b)$$

This indicates that for this set of parameters the horizontal displacement is controlling. Because $(h = 3.75) \cot \theta_V$ increases in proportion to W(cot 6)/4 (Eq. 7) and because this is always smaller than W cot 10 (Eq. 4a modified for the line of clear vision measured to the right edge of the sign), the horizontal displacement will control.

Although this is true for daylight conditions, it does not necessarily pertain to night conditions. Previous work at AIL (11) has indicated that there is a considerable decrease in luminance for signs illuminated by headlights only as their elevation above the pavement increases. This is especially true for low-beam usage. Figure 11 shows the combined effect of horizontal and vertical displacement on luminance. The curves shown are for a 20- by 12-ft sign with a horizontal offset of 30 ft measured to the left edge of the sign.

Superimposed on the graph is a horizontal line that, after Allen et al. (7), indicates the luminance in ft-L required to obtain the equivalent of daylight legibility (50 ft/in.). The effect of increasing lateral displacement can also be judged from Figure 12, which shows the effect on luminance of increasing horizontal displacement.

This discussion has only dealt with one consequence of the lateral displacement of signs: the changes in required minimum letter height caused by changes in the geometry of the line of sight. There is, however, another effect of horizontal curvature that must be considered. The driver uses all available visual cues to satisfy his need for information about the upcoming alignment of the road on which he is driving. At night and especially in rural surroundings, such visual cues may be scarce. A large sign, especially if illuminated or if the background is reflectorized, may be the only such cue or at least the most prominent one. The driver, based on past experience, will have an expectancy that the road will pass close by a sign when that sign is first perceived. If the sign is displaced laterally to an appreciable extent, it may tend to suppress an intervening horizontal curve or indicate a curve when in fact there is none. The possibility of such deceptive cues must be considered when a decision is made to displace a sign. It is suggested that, in all such cases, delineators be installed from the point of first perception to a point past the sign location.

Additional consequences of increased lateral displacement of signs lie in the area of economics. Right-of-way, construction, and maintenance costs will increase as the result of larger signs farther from the roadway. Furthermore, available sign locations will be sharply limited because of increased possibility of topographical restraints and the line of sight falling outside the traveled way, thus increasing the possibility of interference by piers, side slopes, and the like.

These relationships have been derived by strictly analytical means although some of the parameters, such as divergence angle, visibility distance, and the required level of illumination for night visibility, are derived from previous empirical investigations. Empirical validation of these relationships is still needed. Partial validation of the qualitative concept, i.e., decreasing visibility with increased lateral offset, is given by a recently published study (12) made in Connecticut. Although the study was limited to tangent sections (this is not specifically stated in the report but assumed in the absence of contradictory information), the effect of the divergence angle was not evaluated because (a) the test driver expected the sign, and (b) the absence of other traffic from the test section allowed the driver to take his eyes off the road with little potential penalty. The study was still able to conclude that "results indicated that an increased legend size was necessary with increased offset distance to retain original legibility distances".

This paper does not diminish the importance of clear recovery areas and the reduction in the potential of vehicle-sign collisions. Instead, it tries to emphasize the consequences of the removal of signs immediately adjacent to the roadway. The sign

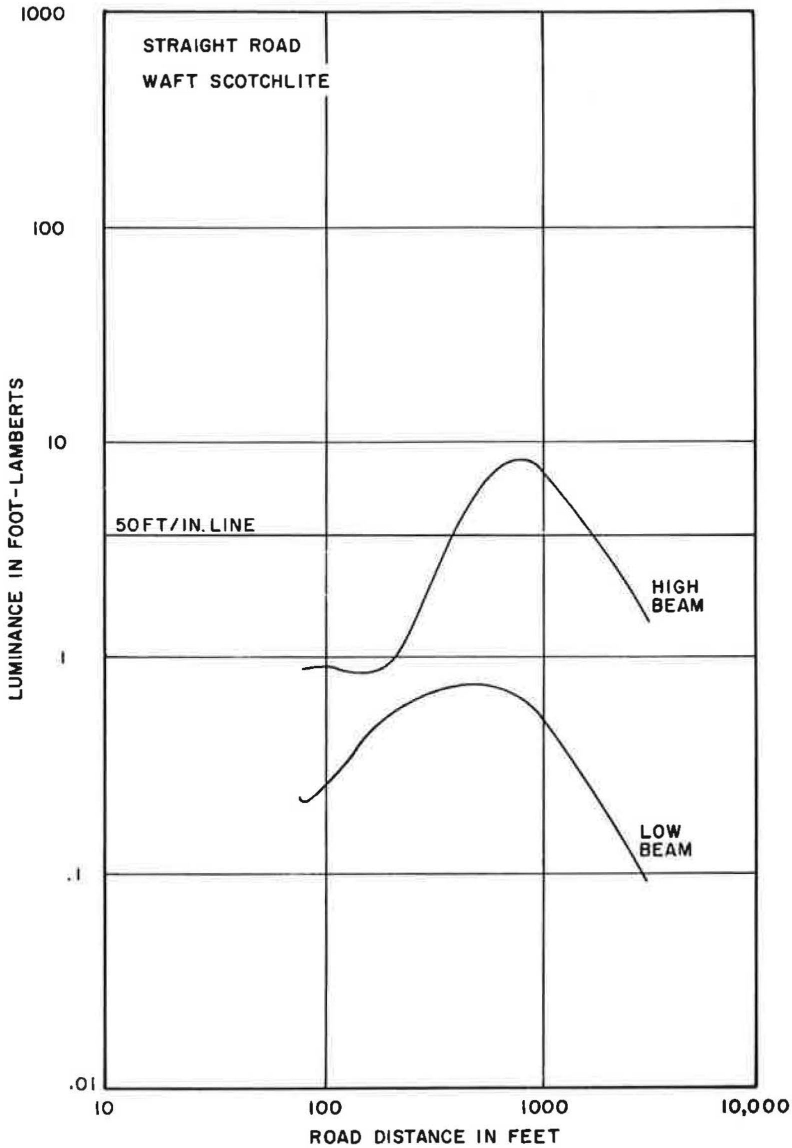


Figure 11. Sign brightness as an effect of combined vertical and horizontal displacement.

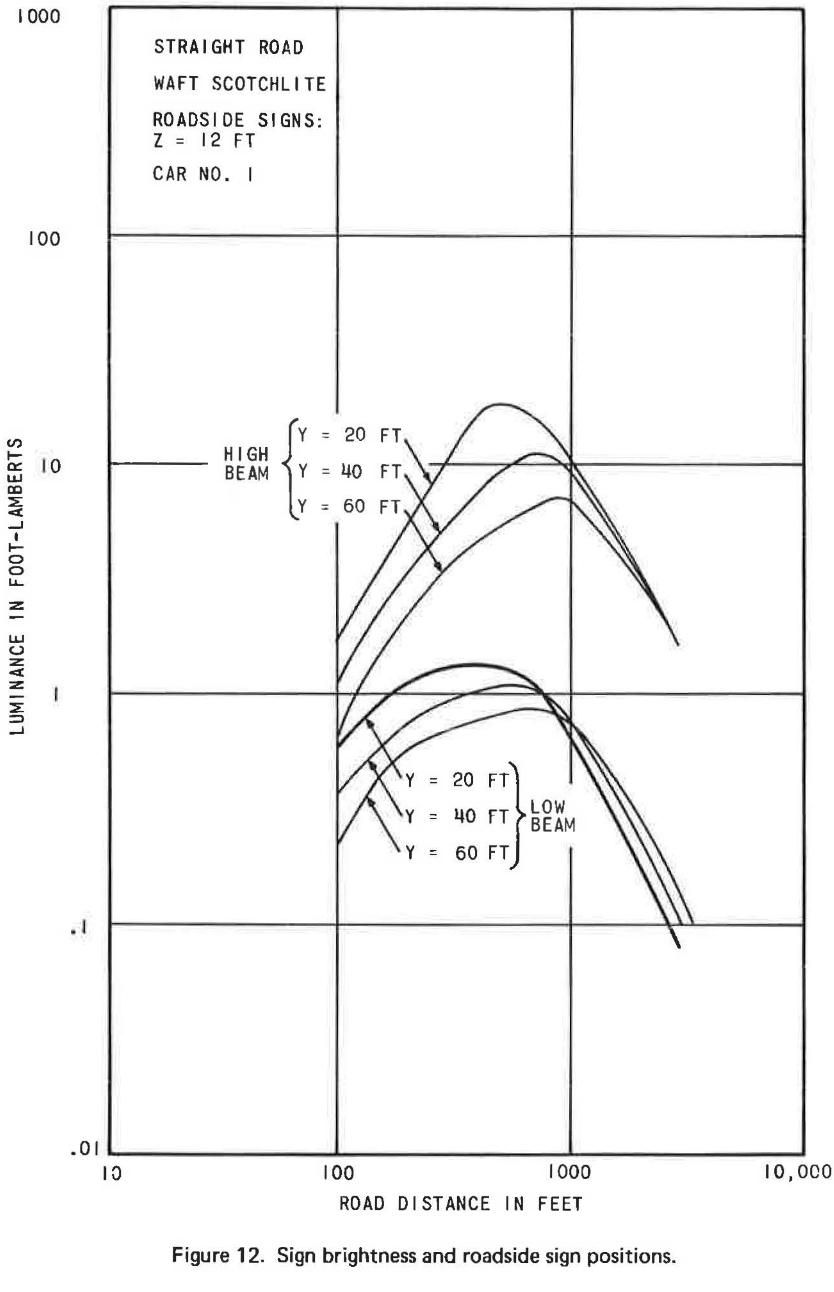


Figure 12. Sign brightness and roadside sign positions.

designer must be alert to these potential effects and ensure that signs installed are legible. This will require adequate letter sizes and a check that the sign actually falls within the cone of normal visions during the entire time when the sign is to be read. If the latter condition cannot be met, a different sign position, a change in message to reduce reading time, or the use of breakaway posts closer to the pavement should be considered.

REFERENCES

1. Mitchell, A., and Forbes, T. W. Design of Sign Letter Sizes. Proc. ASCE, Vol. 68, No. 1, 1942.
2. Moore, R. L., and Christie, A. W. Direction Signs for Motorways. The Engineer, May 13, 1960.
3. Road Research Laboratory. Research on Road Traffic. Her Majesty's Stationery Office, London, 1965.
4. Matson, T. M., Smith, W. S., and Hurd, F. W. Traffic Engineering. McGraw-Hill Book Co., New York, 1955.
5. Heller, F. Rules for the Dimensioning and Arrangement of Verbal Message Traffic Signs. Strassenverkehrstechnik, Vol. 1, No. 12, Dec. 1957. (In German.)
6. Schmidt, I., and Connolly, P. L. Visual Considerations of Man, the Vehicle and the Highway. SAE, Publ. SP279, March 1966.
7. Allen, T. M., Dyer, F. N., Smith, G. M., and Janson, M. H. Luminance Requirements for Illuminated Signs. Highway Research Record 179, 1967, pp. 16-37.
8. Highway Design and Operational Practices Related to Highway Safety. Report of the Special AASHO Traffic Safety Committee, 1967.
9. Manual for Signing and Pavement Marking of the National System of Interstate and Defense Highways. AASHO.
10. A Policy on Geometric Design of Rural Highways. AASHO, 1965.
11. Alexander, G. J., King, G. F., and Warskow, M. A. Development of Information Requirements and Transmission Techniques for Highway Users. AIL, New York, 1967.
12. A Study of Lateral Sign Placement, Connecticut Dept. of Transportation, Bureau of Traffic, June 1969.

Discussion

DONALD A. GORDON, U. S. Bureau of Public Roads—It is a pleasure to discuss King's excellent paper. It will be noted that this paper is an analytic contribution; that is, the conclusions are reached by a synthesis of previous work, rather than by data collected by the author. It seems to me that this is a particularly valuable approach and that it is worthwhile to state its advantages.

King has identified the factors that must be considered in selecting letter sizes on signs offset from the road. These variables include velocity of the car, the offset distance and angle of regard of the driver, and the number of messages on the sign. These factors are tied together in a formula (Eq. 2 of the report) that states the functional relationship between minimum letter height and the effective underlying variables. On a curved road, the offset angle and angle of regard differ from those in the tangent situation, and this case is treated separately.

The statement of sign standards as a dependent variable in a mathematical equation has a number of advantages. The standard is precise and its basis is clearly shown. Standards so stated can be altered and improved as further research more accurately indicates the numbers (constants) in the equation and the range of variables that must be considered in formulating the standard. For example, as King points out, the Germans accept a divergence angle of 15 deg. Americans and English consider 10 deg as maximum. This variance in national practice is a stimulus to researchers to deter-

mine which constant is more valid. The existence of three rival formulas relating reading time to the number of messages on the sign, representing the work of Mitchell and Forbes, Moore and Christie, and the Road Research Laboratory is another challenge to the researcher. Additional variables that might be included in the predictor equation include the effects of adverse weather and night and backlighting. So the analytic approach serves as a stimulus to research, the findings of which may be expected to apply generally to the design of informational road signs.

There are several comments that may be made in passing. The term "degree of curvature" is used in Tables 3 and 4. It might be helpful to those of us who are unfamiliar with the term if it were defined. A more complete interpretation of Tables 3 and 4 might have been helpful. Presumably, the unit in Table 3 is feet, although it is not actually stated in the table. In Figure 6, the parameter γ is not identified.

Aside from these minor criticisms, the author is to be commended on his thorough analytic approach to the setting of standards. It is to be hoped that King's formulas will be validated in further empirical studies and that the analytic approach will be more widely applied in the setting of highway standards.

ROBERT M. WILLISTON, Connecticut Department of Transportation—I am pleased to have been offered the opportunity to discuss this paper on a subject that most certainly is worthy of exhaustive study, inasmuch as the demands on the motorist's attention are increasing enormously.

If I considered myself competent to fully analyze the mathematical aspects of the paper, King's previous work and obvious ability would preclude such an endeavor. Therefore, my remarks will be confined to those areas in which the traffic engineer is most cognizant.

The author has reported on the results of studies conducted by the British Road Research Laboratory relative to the reading time of sign messages. Unfortunately, these papers were not available to the writer. King has noted that this formula is based on 1-sec glances from the road, to the sign, and back to the road. If the 1-sec interval is all the motorist feels he can spare from roadway observation on tangent sections, I wonder if the motorist would still allot the same interval when he is negotiating a curve that is other than minimal.

In Eq. 4a, the author has used 0.117 in lieu of 117 fps, which he previously mentioned as the maximum speed limit on the Interstate System. However, the answer for LH (892 ft) indicates that 117, not 0.117, was used in the computation.

Because the author has indicated that the acceptable maximum divergence angle is 10 deg, it is difficult to accept the equations for curves to the right. It appeared that any point along the arc tV would result in a divergence angle greater than the acceptable 10 deg. To substantiate this premise, a radius was drawn to a point on the curve that was 10 deg upstream from θ. For the sake of computation, the following values were assumed: the degree of curve is 5 deg; the point of curve is the intersection of line LH and arc tV; the point of tangent is the intersection of line S and the arc; and the original radius is the N-S bearing.

Using an Underwood-Olivetti desk computer, I found that a line drawn from W to the tangent of the new radius resulted in an angle of 18 deg, 31 min, 41.4 sec.

In Figure 4, it appears that the sign can never be oriented parallel to line S because the sign must be at or near the perpendicular to line LH. Because the angle of reflection is equal to the angle of incidence, any orientation other than 90 deg ± to line LH would result in reflectance that would be far below acceptance levels. If this approach is valid, sign width is not a factor. It is, therefore, suggested that 0.5W be deleted from Eq. 6b. If the recommended deletion is made, it would appear that Tables 3 and 4 would require revision.

These comments are not intended to be destructive criticism, but rather an indication of the reactions of the average reader. King's approach and evaluation are to be commended.

GERHART F. KING, Closure—It is very gratifying to any author when his work is considered interesting enough to merit discussion. It is especially gratifying to me in this case because the comments were received from both a researcher and a distinguished operational traffic engineer. I would like to thank Gordon and Williston for the time and effort they have expended on analyzing this paper.

Both reviewers point out, correctly, that assumptions as to allowable divergence angle and reading time are central to the derivation of numerical values for letter height. I fully agree that additional theoretical and empirical research in these areas is essential. Equally as important, if not more so, is the necessity to state under which assumptions these relationships are derived and what limitations should be placed on their use. None of the references quoted in my paper completely states these assumptions or limitations concerning the determination of reading time. Williston's point that reading time is a variable depending on the complexities of the driving task is well taken. Studies at AIL of driver information needs (13) have indicated that, under certain conditions, the driving task may be so complex as to make it impossible for the driver to spare any time to read signs. However, the analysis as presented in the paper limits itself to curves within the limits of Interstate design standards. Under these conditions I believe that the curves could be considered as minimal. I fully agree, however, that additional research is needed to define reading times for conditions of increased driving task complexity whether due to extremes in geometrics, traffic conflicts, or climatological conditions.

As far as an acceptable divergence angle is concerned, the accepted value of 10 deg appears to be based on sound theoretical foundations inasmuch as it is based on physiological considerations. However, Moore, in a later paper (14), questions the theoretical derivation by Forbes and Mitchell while stating that "the value seems to be about right in practice." Here, too, additional research is badly needed.

Williston questions whether any point on the arc TV for curves to the right would show the angle of divergence to be 10 deg or less. He works out a sample case for a curvature of 5 deg. I clearly stated in my paper that for degrees of curvature of 1.5 deg or greater, for the conditions stated in the paper, all points on the curve will show a divergence angle exceeding 10 deg. For smaller degrees of curvature there is a range within which the angle of divergence falls below 10 deg. With reference to Figure 6, a computation was made of the changes in θ due to change in γ.

Williston also questions the inclusion of the term $w/2$ in Eq. 6b because of the fact that the sign must be oriented normal to the visual axis. It is actually displaced, in usual practice, about 3 deg from the true normal to minimize specular glare. The term $w/2$ was included in the formulation to account for the fact that S, the offset distance, is defined to the edge of the sign, and the analysis is made for a point on the center of the sign. I am afraid that Williston was misled by the fact, which is my fault, that Figure 6 is not drawn to scale. It is true that some correction would be required in a completely rigorous treatment; this correction is so minimal, however, that it has been ignored. It is my fault, again, for not explicitly stating this simplification.

I would again like to thank both reviewers for the care they have taken in reviewing my paper. I would like, also, to repeat again that my results are completely analytical and that the parametric relationships, as well as the results of the analysis itself, require empirical validation.

References

13. King, G. F., and Lunenfeld, H. Development of Information Requirements and Transmission Techniques for Highway Users. Final Rept. of NCHRP Project 3-12/1, Dec. 1969.
14. Moore, R. L. Traffic Sign Design. Traffic Engineering and Control, March 1962.

Calibration and Correction of Magnetic Loop Detectors

D. C. GAZIS, IBM Thomas J. Watson Research Center; and
R. S. FOOTE, The Port of New York Authority

A procedure is discussed for the calibration and correction of magnetic loop detectors. The need for such a correction arises from the fact that the rise time of the detector signal is an appreciable percentage of the typical duration of the signal. The calibration and correction were carried out by comparing signals from magnetic loops with "benchmark" signals obtained at the same locations by using photocell detectors, which have a considerably shorter signal rise time. Both loops and photocells were arranged in pairs, about 13 ft apart, forming a detector "trap" that permitted measurement of speeds and lengths of vehicles. The signals were received and processed by using an on-line computer system. A regression fit of loop and photocell data yielded an effective length of the loop trap and corrections on the pulse profiles that could match them as much as possible with the benchmark profiles.

•DETECTION HARDWARE is an essential element of every traffic control system. In recent years, loop detectors have gained increasing acceptance because of their reliability and relatively low maintenance requirements. Although the electronic characteristics of these devices are acceptable in many traffic applications, they may be less than satisfactory in those cases where an accurate estimate of the speed and length of each vehicle is required. Errors in excess of 10 percent are not uncommon in the measurements of these parameters by means of magnetic loops, and it is not known how the configuration of the observed vehicle (e.g., its height) affects these measurements. Furthermore, it is not known to what extent careful calibration can increase the reliability of these measurements.

In an attempt to answer some of these questions, we undertook a study of performance characteristics of magnetic loop detectors using the extensive instrumentation of the Lincoln Tunnel traffic control experiment, which has been described in previous papers (1, 2, 3). In what follows, we give the results of a preliminary study of three pairs of magnetic loop detectors located at three points along one lane of the south tube of the Lincoln Tunnel. The results indicate that careful calibration may permit improved reliability of measurements of loop detectors. The calibration was carried out by comparing the magnetic loop data with those obtained from overlaid photocell detectors and from television recordings of a long sequence of vehicles.

BACKGROUND OF THE CALIBRATION TESTS

The Lincoln Tunnel traffic control experiment is one in which it is essential to have accurate measurements of the speed and length of each vehicle. The need for accurate speed estimates for control purposes requires no elaboration. The need for accurate estimates of lengths exists because the sequence of lengths is used to identify individual vehicles and thus to obtain the counts of vehicles between detection points (1, 2).

The detection requirements were adequately met, during the initial phase of the experiment, by photocell detectors aimed at high-intensity lights. In the planned extension

Paper sponsored by Committee on Traffic Control Devices and presented at the 49th Annual Meeting.

of the experiment and eventual instrumentation of all trans-Hudson tunnel tubes, however, it would be desirable to use magnetic loop detectors, largely because of their lower maintenance requirements. Unfortunately, the first results of experimentation with magnetic loops were disappointing. The loops generally have a signal rise time of about 70 msec or more, compared to a rise time of about 15 msec for the photocells, and a signal decay time of 40 msec, compared with only 5 msec for the photocells, as shown in Figure 1. This fact, coupled with the sensitivity of the magnetic loops to the vertical distance of the vehicle from the pavement, appeared to give a resolution of 50 to 100 msec in the measurement of the duration of the signal caused by a vehicle passing over the detector. This is to be compared with a resolution of about 10 msec for the photocells. Apart from this more sluggish response, and of greater importance for measuring speeds and lengths, is the greater variance of the loop response. The net effect is that, although the cells measure vehicle lengths accurately to ±3 in., the loops are accurate only to ±2 ft.

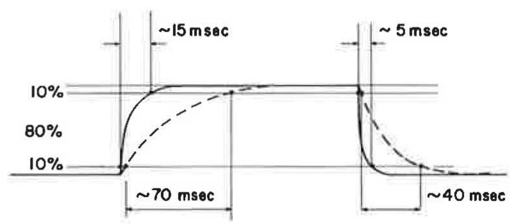


Figure 1. General shape of pulses obtained from cells (solid line) and loops (dashed line). The rise time is defined as the time it takes the signal to rise from 10 to 90 percent of its maximum value.

Nevertheless, it would be possible to improve the accuracy of these measurements if it could be proved that the errors were largely in the nature of a constant shift of the origin or end of a pulse. The existence of the photocell detecotrs made such an error analysis feasible, because the photocells could be used as benchmarks for this analysis. Accordingly, three pairs of magnetic loop detectors were placed virtually over pairs of photocells. The signals from all 12 detectors were transmitted to an 1800 computer located at Yorktown Heights, New York, about 40 miles away from the Lincoln Tunnel. This computer has been programmed for the on-line control of the Tunnel traffic by means of devices restraining the traffic input. An essential function of the control program is the identification of speed and length of all vehicles. This information is used for on-line processing leading to appropriate control commands. It can also be stored on a magnetic disc for future off-line processing. This, in fact, was the procedure followed for the calibration analysis. While the data were collected, portable television recorders were used to record all vehicles entering and exiting the tunnel. This information was used to identify the exact type of individual vehicles.

CALIBRATION PROCEDURE

The passage of a vehicle over a pair of detectors produces, in effect, two signals as shown in Figure 2. The actual electric pulse corresponds to the gradually rising and falling intensity. The square pulse corresponds to a "contact closure" equivalent, which is transmitted to the computer via a tone-multiplexing system. As indicated in earlier works (1, 2), from the four times, T_1, T_2, T_3, and T_4, we determine the speed and length of the observed automobile according to the relationships

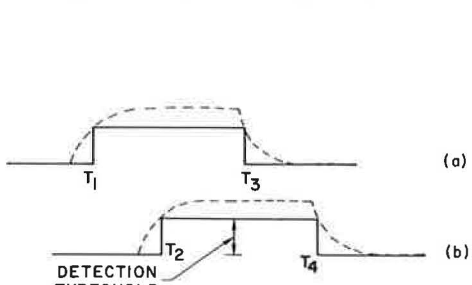


Figure 2. Pulses generated by a vehicle passing over the two detectors of a trap. Pulse (a) is from the upstream detector and pulse (b) from the downstream detector.

$$V = L/T_{34} \quad (1)$$

$$\ell = L\left(\frac{T_{13}}{T_{13} + T_{24}}\right)\left(\frac{T_{14}}{T_{12}} + \frac{T_{23}}{T_{34}}\right)$$

where V is the speed and ℓ the length of the automobile, L is the length of the trap (distance between the two detectors), and

$$T_{ij} = T_j - T_i \tag{2}$$

The following procedure was used for calibration. For each automobile, which was easily identified by the time of entrance into the trap, we computed the time differences, T_{ij}, given by the photocells and corresponding time differences, $\tilde{T}_{ij}$, given by the loops. A linear regression fit was obtained between the samples T_{ij} and $\tilde{T}_{ij}$, for $(i, j) = (1, 2)$, $(2, 3)$, and $(3, 4)$. This fit yields the relationship

$$\left[\tilde{T}_{ij}\right] = a_{ij} + \lambda_{ij}\left[T_{ij}\right] \tag{3}$$

where brackets denote averaging, a_{ij} is the cut-off of the regression line, and λ_{ij} is its slope in the plane $\tilde{T}_{ij}$ versus T_{ij}.

After obtaining the regression fit, we "correct" all the time differences $\tilde{T}_{ij}$ according to the relationship

$$T^{*}_{ij} = \left(\tilde{T}_{ij} - a_{ij}\right)/\lambda_{ij} \tag{4}$$

and use T^{*}_{ij} for the computation of speeds and lengths according to Eq. 1. It may be noted that the slope λ_{34} of the regression line $\tilde{T}_{34}$ versus T_{34} has a physical meaning: Its product with the distance of the corresponding two photocells is the average effective length of the loop trap.

RESULTS AND DISCUSSION

Some of the preliminary results of the calibration are given in Tables 1, 2, and 3 and shown in Figures 3 and 4. Table 1 gives the speeds and lengths of vehicles crossing trap 1, as well as the corresponding times T_i $(i = 1, 4)$. Below these numbers are listed the corresponding values obtained from the uncorrected loop data on the basis of an assumed effective length of 13 ft, which is the distance of the centers of the two loops. It is seen that the estimates of speeds from the loops are fair, but the estimates of lengths are grossly inaccurate. Table 2 gives the same times T_i, but the speeds and

TABLE 1

SPEEDS AND LENGTHS OF VEHICLES CROSSING TRAP 1 AS MEASURED BY CELLS AND UNCORRECTED LOOPS

Speed	Length	T_1	T_2	T_3	T_4	Time of Exit
41.67	14.56	49.479	49.799	49.812	50.148	16.24.34
41.80	20.06	49.777	50.078	50.244	50.555	
39.22	15.04	51.939	52.281	52.307	52.664	16.24.36
38.81	19.37	52.252	52.563	52.721	53.056	
40.00	16.55	53.069	53.410	53.473	53.823	16.24.37
40.12	21.85	53.386	53.694	53.909	54.233	
36.55	17.83	54.721	55.092	55.195	55.578	16.24.39
36.83	23.46	55.036	55.377	55.656	56.009	
38.15	13.10	56.383	56.733	56.710	57.077	16.24.41
37.57	19.15	56.687	57.010	57.168	57.514	
37.74	17.77	58.069	58.433	58.532	58.903	16.24.42
37.46	23.15	58.388	58.725	58.992	59.339	
38.46	17.95	59.060	59.413	59.514	59.878	16.24.43
38.35	23.36	59.362	59.691	59.957	60.296	
36.65	16.96	62.514	62.889	62.969	63.351	16.24.47
36.21	22.95	62.825	63.177	63.449	63.808	
33.90	17.47	64.529	64.931	65.032	65.445	16.24.49
33.86	22.60	64.857	65.229	65.508	65.892	
34.83	17.19	1.152	1.550	1.641	2.043	16.24.51
35.52	22.35	1.487	1.855	2.119	2.485	

lengths are now computed from the corrected loop data. Table 3 gives the results of the regression fits between T_{ij} and $\tilde{T}_{ij}$. The "corrections" are the a_{ij} and the "slopes" are the λ_{ij}.

Figure 3 shows a plot of the lengths as computed from the corrected loop data plotted versus the corresponding lengths computed from the photocell data. In addition to the scatter of the data about a straight line through the origin, there is an appreciable deviation of certain points in the range of lengths between 20 and 24 ft. It has been ascertained from examination of the television recordings that these vehicles were generally small trucks. Apparently the greater distance of these trucks from the pavement caused an error in the measurements of the pulses T_{13} and T_{24} different from that observed for passenger cars. A similar increase in the error was present in the case of long trucks as well, but the percentage error in length computation was smaller. Buses and passenger cars appeared to be consistently better measured from the corrected loop data.

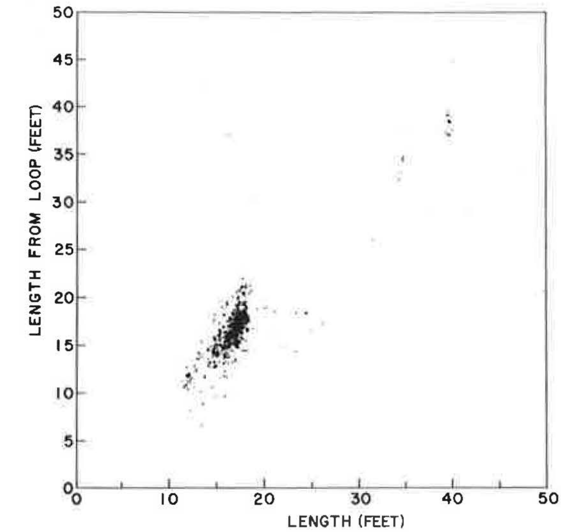


Figure 3. The lengths of 558 vehicles, computed from loop data after correction, plotted versus the lengths of the same vehicles computed from cell data.

Figure 4 shows the effect of the speed on the error of the length estimate. Again, the length is computed from the corrected loop data and compared with the value obtained from photocell data. This figure shows the relative percent error, given by

$$e = 100\left(\frac{\ell_{trap} - \ell_{loop}}{\ell_{loop}}\right) \tag{5}$$

TABLE 2

SPEEDS AND LENGTHS OF VEHICLES CROSSING TRAP 1 AS MEASURED BY CELLS AND CORRECTED LOOPS

Speed	Length	T_1	T_2	T_3	T_4	Time of Exit
41.67 41.62	14.56 14.38	49.479 49.777	49.799 50.078	49.812 50.244	50.148 50.555	16.24.34
39.22 38.66	15.04 14.04	51.939 52.252	52.281 52.563	52.307 52.721	52.664 53.056	16.24.36
40.00 39.96	16.55 16.39	53.069 53.386	53.410 53.694	53.473 53.909	53.823 54.233	16.24.37
36.55 36.70	17.83 18.59	54.721 55.036	55.092 55.377	55.195 55.656	55.578 56.009	16.24.39
38.15 37.44	13.10 14.04	56.383 56.687	56.733 57.010	56.710 57.168	57.077 57.514	16.24.41
37.74 37.33	17.77 18.19	58.069 58.388	58.433 58.725	58.532 58.992	58.903 59.339	16.24.42
38.46 38.20	17.95 18.25	59.060 59.362	59.413 59.691	59.514 59.957	59.878 60.296	16.24.43
36.65 36.09	16.95 18.22	62.514 62.825	62.889 63.177	62.969 63.449	63.351 63.808	16.24.47
33.90 33.75	17.47 18.22	64.529 64.857	64.931 65.229	65.032 65.508	65.445 65.892	16.24.49
34.83 35.40	17.19 17.80	1.152 1.487	1.550 1.855	1.641 2.119	2.043 2.485	16.24.51

TABLE 3

RESULTS OF CALIBRATION OF MAGNETIC LOOPS

Trap	Sample	Correlation Coefficients			Corrections			Slopes			Effective Trap Length
		T_{12}	T_{23}	T_{34}	T_{12}	T_{23}	T_{34}	T_{12}	T_{23}	T_{34}	
1	558	0.997	0.945	0.980	-0.013	0.157	-0.002	0.977	1.001	0.930	13.03
2	546	0.996	0.899	0.997	0.037	0.096	0.010	0.940	0.986	0.949	13.29
3	554	0.998	0.926	0.994	0.031	0.182	-0.029	0.967	0.967	1.025	14.35

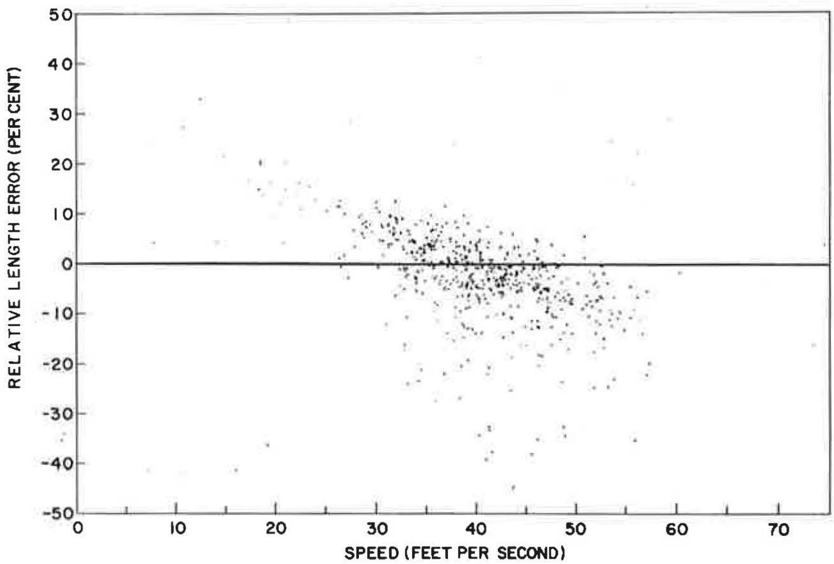


Figure 4. Influence of the speed on the relative error in estimating the lengths of the 558 vehicles from loop data.

plotted versus the speed obtained from photocell data. There appears to be a definite trend of diminishing error with increasing speed, from a maximum of about +10 percent at speeds of 20 fps to -10 percent at speeds of 50 fps.

The results shown in Figures 3 and 4 indicate that a better correction might be obtained by dividing the data into speed and length classes and correcting separately for each class. This is the next step planned in our analysis. At the present, we see no obvious solution, through computer logic alone, to the problem of misreading the length of high vehicles. However, another means of improving the detection of vehicles by magnetic loops is through the improvement of the hardware properties themselves. The large errors in the measurement of lengths of high vehicles are influenced by the shape of the magnetic field, which might be improved. Another possibility is to redesign the electronic hardware of the loops in order to increase the information concerning the signature of the pulse, e.g., by tapping the signal at two different threshold levels. We believe that this avenue for improvement of the detection hardware merits further investigation in conjunction with calibration measurements such as these reported here.

After all these corrections are made, the true test of the effectiveness of our correction procedure will be the use of the corrected loop data for the purposes of the online control program, namely, to identify patterns of lengths of cars and to use this information to initialize and update the counts of vehicles between traps. The results reported here appear to be encouraging regarding this prospect.

REFERENCES

1. Bennett, B. T., Gazis, D. C., Edie, L. C., and Foote, R. S. Control of the Lincoln Tunnel Traffic by an On-Line Digital Computer. Proc. 4th Internat. Symposium on Traffic Theory, Karlsruhe, Germany, June 1968, in press.
2. Gazis, D. C., and Foote, R. S. Surveillance and Control of Tunnel Traffic by an On-Line Digital Computer. Transportation Science, Vol. 3, No. 3, Aug. 1969.
3. Foote, R. S. Instruments for Road Transportation. High Speed Ground Transportation Journal, May 1968.

Interchange Ramp Color Delineation and Marking Study

WALTER J. ROTH, Michigan Department of State Highways, Lansing

This paper reports on a study of the effects of special color coding at a series of conventional interchanges on a 40-mile section of rural freeway. The 18 interchanges with the color coding contained diamond, loop, and connector ramps in the northbound direction only. The color scheme followed the lead of the pilot study on US-27 in 1965, and Minnesota's first approach to exit color coding. Interviews with drivers were taken at 14 of the exits to develop the degree of driver recognition and knowledge of the color-coded areas. While the interviews were being taken, observations were made of the driving patterns to the exits. In other studies accidents were reviewed for 1 year before to 1 year after the color coding. A selection of drivers was also interviewed on the through route to gain their knowledge and recognition concerning the exit and entrance color-coding scheme. Erratic driving was significantly reduced after the color codes were applied and was reduced most at other than diamond ramps. The color code was as effective during the day as during the night. Edge markings were hardly noticed until they were changed to blue, and then there was a 5 to 11 times increase in the percentage of drivers who noticed them. For information on their destination exits, drivers noticed ramp signs most, and the change to blue background was noticed more during the day than during the night. Of those who used the freeways only once a year or less, 66 percent of the day group interviewed and 83 percent of the night group knew the correct meaning of the color codes. Accidents were not directly relatable to the color codes; however, the increase in accidents in the non-color-coded direction was three times greater than that in the color-coded direction. The results indicate that color codes offer a good potential for easing the driver's task by presenting a better delineated path that is more easily understood, results in less confusion, and possibly produces fewer accidents.

•THE STUDY grew out of an earlier one in which color was used successfully to code two left exit interchanges on US-27 where the approaching driver's view gave the impression that the exits were the through roadway (1, 2). In this later study, color codes were applied at a series of conventional interchanges on a continuous route. Extensive data were obtained for 3 months in 1965, and further data were obtained in mid-1966.

The study area covers a 40-mile section of US-23 in the northbound direction from Territorial Road near Ann Arbor to Hill Road south of Flint. The northbound exit ramps include 14 diamonds, four loops, one left connector ramp, and one right connector ramp (Fig. 1). The rural freeway is a four-lane, divided, limited-access facility with average daily traffic varying from 12,000 to 19,000 vehicles. The summer study period variation was not this great, however, as the weekday northbound volumes through the study area averaged 7,800 vehicles per day. The purpose of this

Paper sponsored by Committee on Traffic Control Devices and presented at the 49th Annual Meeting.

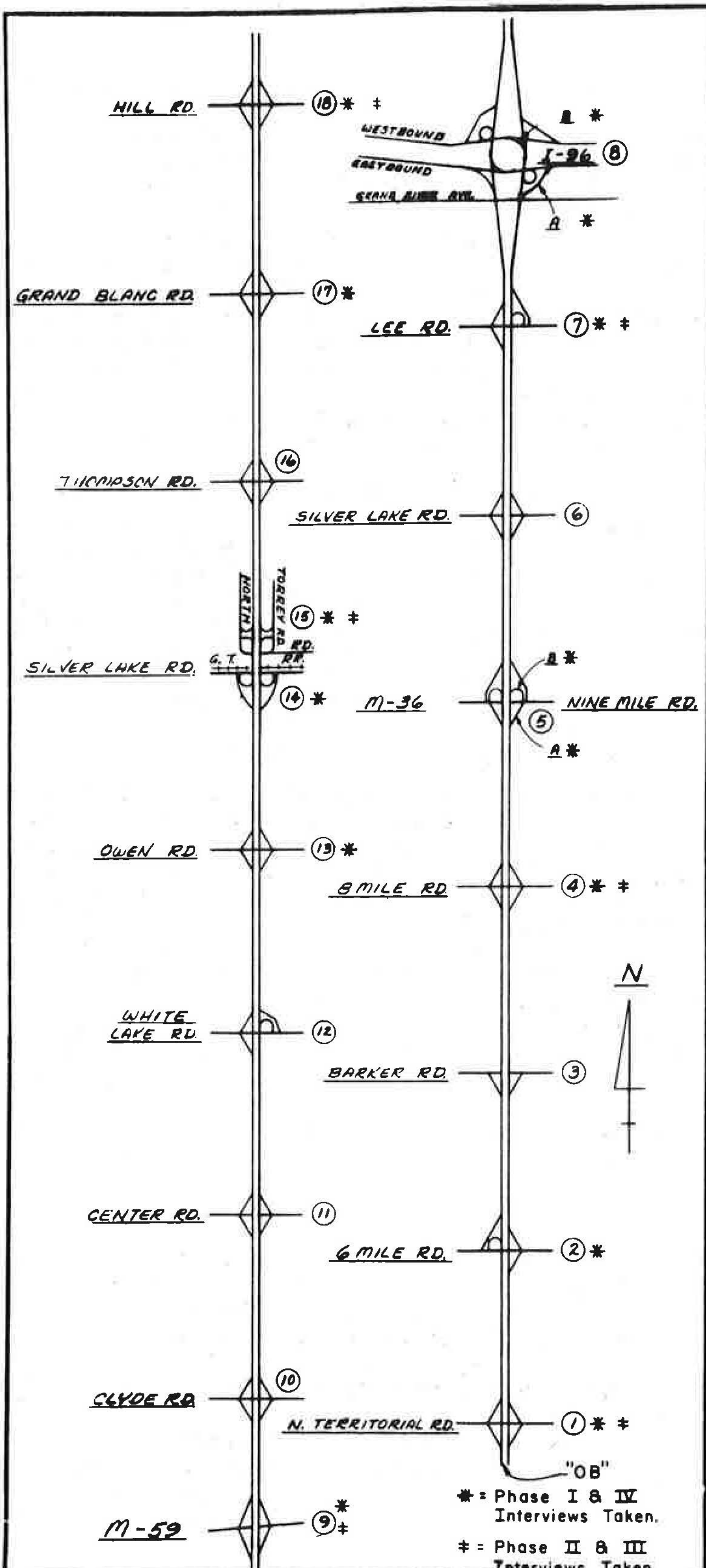


Figure 1. Type of interchanges and interview locations in the study area.

study was to evaluate the effects of color coding a series of conventional interchanges along a continuous route.

Experimentation with color coding at interchange ramps has been conducted by the Minnesota Department of State Highways and the Florida Department of Transportation (formerly the Florida State Road Department). The Minnesota experiment was conducted in 1960 in cooperation with the Minnesota Mining and Manufacturing Company and was reported by Mathew J. Huber, Yale Traffic Bureau. In this experiment color was applied to the entire surface of the ramps and corresponding delineators were installed. In the Florida experiment, color was applied only to edge marking with corresponding delineators (3). In both experiments, white color was used for through roadway, yellow for on-ramps, and blue for off-ramps. Both experiments reported that the application of color coding provided definite benefits for motorists and reduced confusion for both exiting and through traffic. "Glance" notification in advance of the interchange permitted earlier alignment of exiting traffic.

These experiments generated widespread interest in the color coding of interchange ramps for better notification and guidance to freeway motorists. The project in Michigan was planned in 1961 and initiated in 1962 with preliminary material investigations. Michigan added color coding to the exit and directional signs, whereas the previous projects were concerned only with edge marking and delineation. Since then, other states have become interested in similar projects. Ohio has experimented with color edge marking at interchange ramps on a limited basis. Oregon initiated a project in the summer of 1964 that consists of the application of color coding to five successive interchanges and includes the exit signs at the ramps in the color system. These latter projects continued the identical color designations in the color system initiated in Minnesota and used in the Florida and Michigan projects.

DETAILS OF THE COLOR-CODING SYSTEM

The selection of colors is based on compatibility and consistency with present highway usage, colors already selected in the related projects, and review of colors available for use. On this basis, white was selected for the through roadway, blue for the exit ramps, and yellow for the entrance ramps. White edge marking and clear delineators are already used to delineate the through roadway. Yellow for the entrance ramps is a natural extension of its use as a warning or caution color. Blue for exit ramps provides a distinct contrast with the through-roadway white and entrance-ramp yellow. The subdued characteristic of blue, by contrast, emphasizes the white of the through-roadway and still provides sufficient differentiation and attraction. In addition, blue is the standard background color for Interstate REST AREA and GAS-FOOD-LODGING signs and also for STATE POLICE signs in Michigan. These signs imply an exiting movement from the freeway, and the blue color conforms with these familiar uses.

The elements of the color system are shown in Figures 2 and 3 representing applications at typical diamond and loop ramps. Specific details pertinent to the study are as follows:

<u>At Exits</u>

1. The exit sign with arrow in the gore was changed from green to blue background.
2. Both sides of the ramps were painted with an 8-in. blue reflectorized line that began on the right side of the ramps approximately 250 ft in advance of the ramp take-offs from the through lane. This permitted initial visibility of the blue edge line along the through roadway for a clearer lead into the exit. These lines stopped on the ramps 100 to 150 ft short of the ramp termini. Motorists were interviewed at the stop sign position at the ramp ends.
3. All 3-in. amber exit delineators were replaced with special 4- by 10-in. blue retroreflective delineators. In addition, the delineator posts were painted blue. The extent of the delineators followed the length of the blue edge line.
4. The ramp destination signs were changed from a green to a blue background.

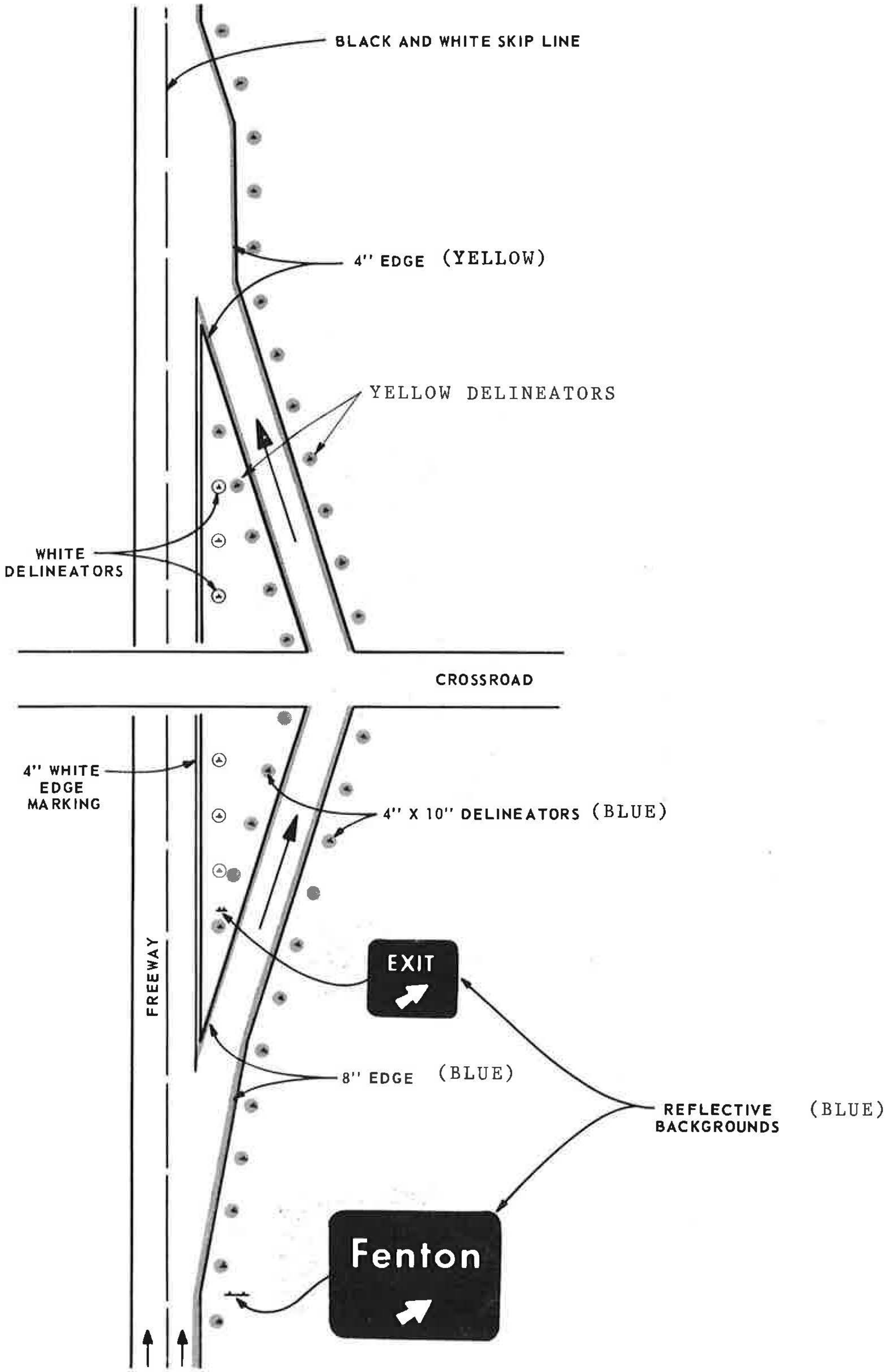


Figure 2. Color coding at typical diamond interchange.

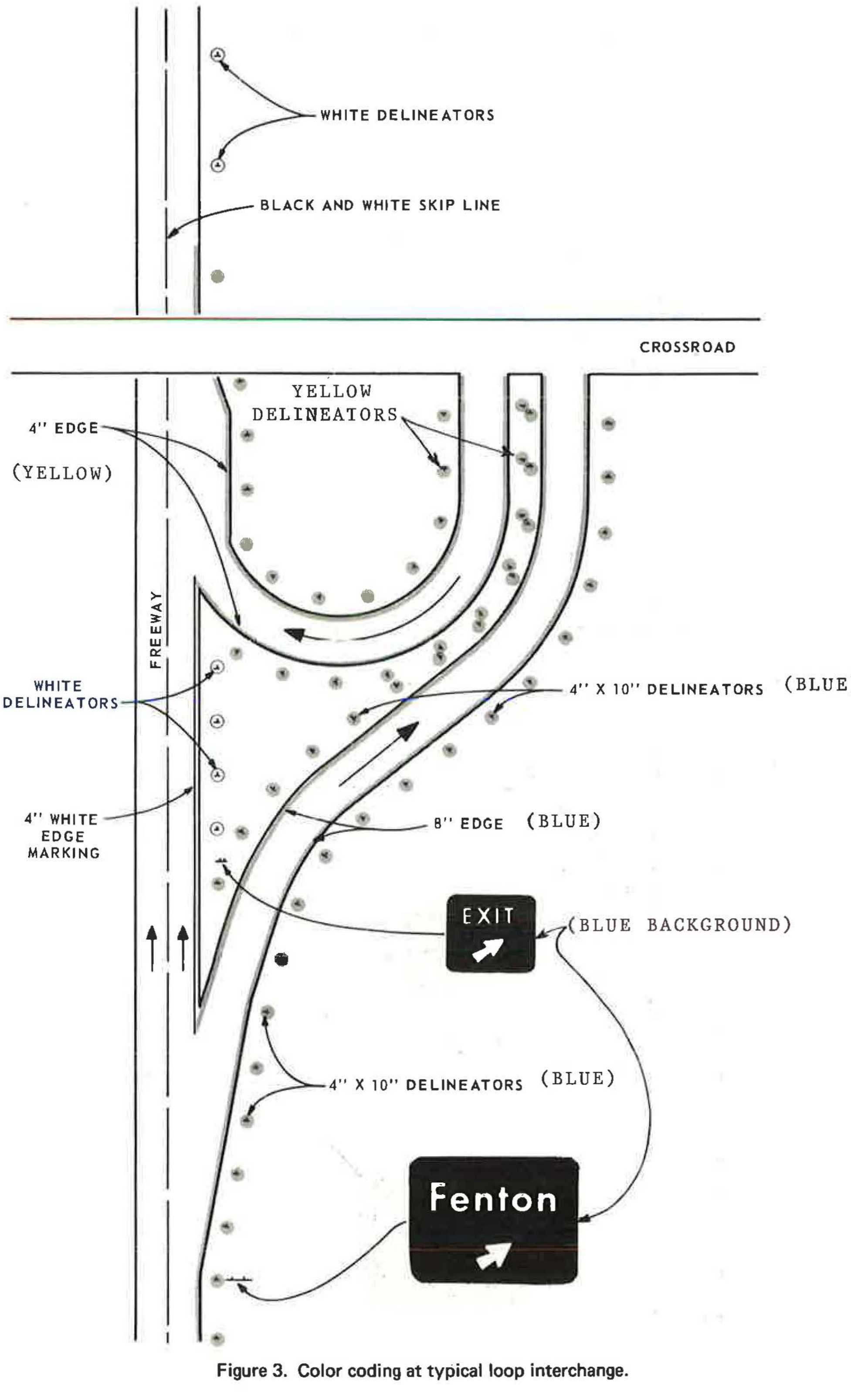


Figure 3. Color coding at typical loop interchange.

At Entrances

1. A 4-in. wide yellow reflectorized edge line was painted along each side of the ramp to the points of contact with the through lane.
2. The 3-in. amber standard delineators were retained throughout, and the posts were painted yellow.
3. North of the I-96 interchange, 3 study identification signs were placed: one stating that the next 25 miles is study area and two stating that exit is on blue and entrance is on yellow.

TABLE 1

ERRATIC MOVEMENTS BEFORE AND AFTER COLOR CODES

Movement	Before	After
Delayed exits	263	72
Stopped or slowed	133	61
Rapid deceleration or swerve	158	116
False exit	67	39
Backing at gore	7	8
Total	628	296

ERRATIC-MOVEMENT STUDY

An analysis was made of erratic movements as observed in July and August 1965 on the US-23 study section shown in Figure 1. Observations of drivers' exit performances were made before and after the application of the color code for 16 hours at each of 14 locations. The before study, Phase I, was conducted July 6 through July 20. The after study, Phase IV, was conducted August 23 through September 3. Hereafter, these observation periods will be referred to as before and after.

Erratic movements were classified according to the following descriptions:

1. Delayed Exit—A driver who delays his exit long enough to drive across the painted gore or dirt.
2. False Exit—A driver who begins his exit but returns to the through lane.
3. Backing at Gore—A driver who stops beyond the gore and then backs up either on the ramp or on the through roadway, to go in a different direction.
4. Rapid Deceleration or Swerve—A driver who drives the ramp on the path but has to take an abrupt action to do so.
5. Stopped or Slowed—A driver who comes to a complete stop to decide which way to go or is definitely confused about direction and slows down to decide.

A distribution of erratic movements is given in Table 1. By using the t-test, we found erratic movements were significantly reduced following the application of the color scheme. The reduction in rapid deceleration or swerve movements was greatest at exit ramps that were not diamonds. The reduction in delayed exit movements, false exit movements, and stopped or slowed movements was the same at both diamond and other ramps. The reductions in erratic movements were proportionately the same for day and night. This would indicate that the color scheme aids the motorist as much in the daytime as at nighttime.

US-23 EXITING-TRAFFIC INTERVIEW STUDY

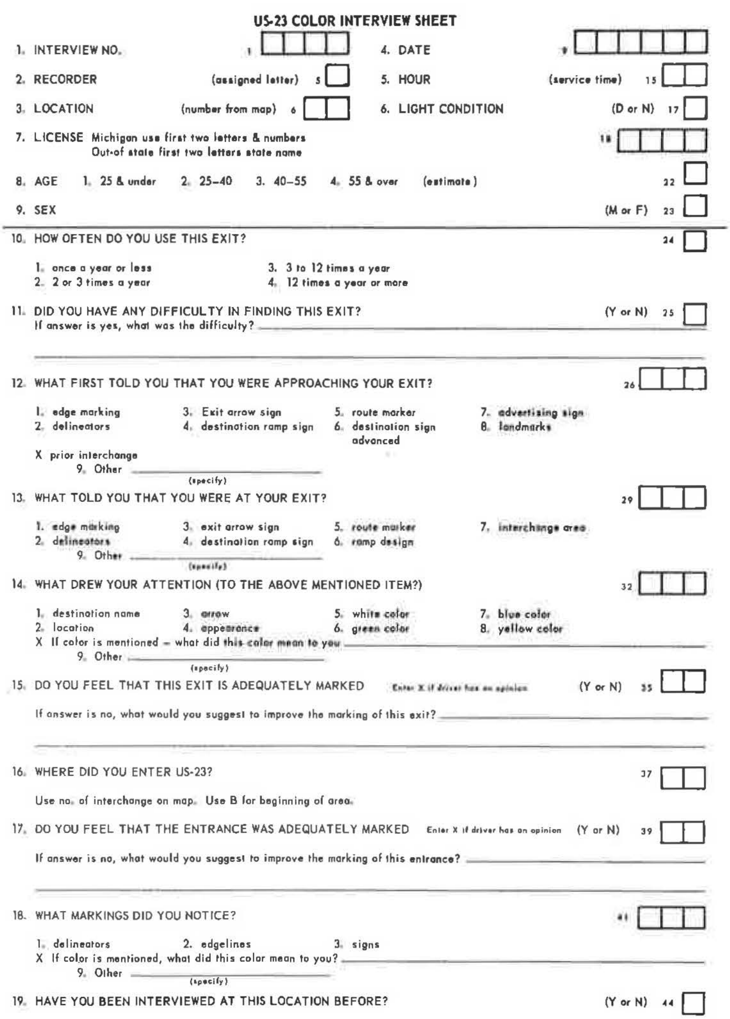
Table 2 gives the places and schedule of events during this study. Phase I or the before interviews were conducted at 14 interchange exits at the ramp termini shown on the map in Figure 1. The interview form, shown in Figure 4, was completed for each exiting vehicle at each of these ramps for a 16-hour period. The rest of the ramps carried volumes so small that meaningful data could not be obtained. An observer also recorded erratic movements at each of these exits for the total study period. Entrance data, discussed in a later section, were taken from these interview forms.

Following Phase I all the exit signs in the gore were changed to blue background, and Phase II interviews were then taken at the six exits shown in Figure 1. After the interviews, all of the edge marking and posts were painted, and the blue delineators were installed. Phase III interviews were then conducted at the same six exits shown in Figure 1 for Phase II. After the interviews, the last part of the color coding was placed by changing all the exit ramp destination signs to blue background. Phase IV included the final after interviews, and recording of the erratic movements at the same 14 exits as in Phase I.

TABLE 2

SCHEDULE OF EXITING-TRAFFIC INTERVIEW STUDY

Phase	Event	Date
I	Interviews taken	July 2
	Interviews taken and erratic movements observed at 14 exits	July 6 through 20
	18 exit and two gore signs with blue backgrounds installed	July 21 through 23
II	Interviews taken and erratic movements observed at six exits (1 control)	July 26 through 23
	Edges painted blue, yellow, and white	July 29 through August 6
	Blue delineators installed at exits	July 29 through August 6
	Posts painted blue and yellow	July 29 through August 6
III	Interviews taken and erratic movements observed at six exits	August 9 through 12
	Ramp destination sign overlays with blue background installed	August 13 through 20
IV	Interviews taken and erratic movements observed at 14 exits	August 23 through September 3



US-23 COLOR INTERVIEW SHEET

1. INTERVIEW NO. 1
4. DATE 9
2. RECORDER (assigned letter) 5
5. HOUR (service time) 15
3. LOCATION (number from map) 6
6. LIGHT CONDITION (D or N) 17
7. LICENSE Michigan use first two letters & numbers
Out-of state first two letters state name 18
8. AGE 1. 25 & under 2. 25–40 3. 40–55 4. 55 & over (estimate) 22
9. SEX (M or F) 23

10. HOW OFTEN DO YOU USE THIS EXIT? 24
1. once a year or less
2. 2 or 3 times a year
3. 3 to 12 times a year
4. 12 times a year or more

11. DID YOU HAVE ANY DIFFICULTY IN FINDING THIS EXIT? (Y or N) 25
If answer is yes, what was the difficulty?

12. WHAT FIRST TOLD YOU THAT YOU WERE APPROACHING YOUR EXIT? 26
1. edge marking
2. delineators
3. Exit arrow sign
4. destination ramp sign
5. route marker
6. destination sign advanced
7. advertising sign
8. landmarks
X prior interchange
9. Other (specify)

13. WHAT TOLD YOU THAT YOU WERE AT YOUR EXIT? 29
1. edge marking
2. delineators
3. exit arrow sign
4. destination ramp sign
5. route marker
6. ramp design
7. interchange area
9. Other (specify)

14. WHAT DREW YOUR ATTENTION (TO THE ABOVE MENTIONED ITEM?) 32
1. destination name
2. location
3. arrow
4. appearance
5. white color
6. green color
7. blue color
8. yellow color
X If color is mentioned – what did this color mean to you
9. Other (specify)

15. DO YOU FEEL THAT THIS EXIT IS ADEQUATELY MARKED Enter X if driver has an opinion (Y or N) 35
If answer is no, what would you suggest to improve the marking of this exit?

16. WHERE DID YOU ENTER US-23? 37
Use no. of interchange on map. Use B for beginning of area.

17. DO YOU FEEL THAT THE ENTRANCE WAS ADEQUATELY MARKED Enter X if driver has an opinion (Y or N) 39
If answer is no, what would you suggest to improve the marking of this entrance?

18. WHAT MARKINGS DID YOU NOTICE? 41
1. delineators
2. edgelines
3. signs
X If color is mentioned, what did this color mean to you?
9. Other (specify)

19. HAVE YOU BEEN INTERVIEWED AT THIS LOCATION BEFORE? (Y or N) 44

Figure 4. Interview form used for US-23 exiting-traffic interview study.

TABLE 3

CHARACTERISTICS OF DRIVERS INTERVIEWED

Characteristic	Phase I		Phase II		Phase III		Phase IV	
	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Age								
Under 25	1,078	11.5	508	13.8	375	9.9	1,253	14.3
25 to 40	4,894	52.4	1,830	49.7	1,988	52.4	3,930	44.9
40 to 55	2,522	27.0	965	26.2	1,117	29.4	2,637	30.1
55 and over	843	9.0	338	9.2	313	8.3	937	10.7
Sex								
Male	7,466	78.1	2,864	77.2	3,089	80.0	6,952	78.0
Female	2,099	21.9	847	22.8	774	20.0	1,958	22.0
Freeway Use								
Once a year	1,719	18.5	682	18.7	711	18.9	1,453	16.8
2 to 3 times a year	486	5.2	179	4.9	183	4.9	504	5.8
4 to 12 times a year	795	8.5	270	7.4	281	7.5	814	9.4
12 or more times a year	6,317	67.8	2,516	69.0	2,594	68.8	5,867	67.9
License								
Michigan	8,936	93.4	3,398	91.6	3,515	91.0	8,253	92.6
Out-of-state	629	6.6	313	8.4	343	9.0	657	7.4

Only six exits were studied in Phases II and III for two reasons: (a) Reducing the survey time between the separate applications of the color-coding elements made it possible to complete the study in the one summer period; and (b) the six exits provided a representation of the various interchange types with sufficient volumes to secure an adequate data base. This was apparently not accomplished entirely as there are indications that the one unchanged control interchange at Territorial Road probably was not completely adequate to act as a control. On the other hand, some degree of control data was present from the total information derived from Phase I, which was taken prior to any changes in the area.

TABLE 4

SIGNS AND MARKINGS THAT GAVE THE FIRST NOTIFICATION OF APPROACHING EXITS AND THE NUMBER AND PERCENTAGE OF DRIVERS NOTIFIED BY EACH

Phase	Edge Marking		Exit Arrow Sign		Ramp Destination Sign		Route Marker		Advance Destination Sign		Advertising Signs and Landmarks		Total
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	
Day, Usage Less Than Once a Month													
I	3	0.1	62	2.8	139	6.2	66	2.9	1,653	73.5	326	14.5	2,249
II	1	0.1	10	1.4	78	10.9	7	0.1	448	62.6	172	24.0	716
III	9	1.15	2	0.25	40	5.1	4	0.5	543	69.6	182	23.3	780
IV	45	2.5	15	0.8	196	10.6	83	4.5	1,178	63.8	327	17.7	1,846
Total													5,591
Day, Usage at Least Once a Month													
I	4	0.1	113	2.4	325	6.9	108	2.3	2,554	54.6	1,575	33.6	4,678
II	2	0.1	27	1.6	150	8.7	5	0.3	984	57.2	553	32.1	1,721
III	45	2.4	12	0.1	98	5.3	5		1,078	58.6	598	32.5	1,839
IV	129	2.9	30	0.1	377	8.4	39	0.1	2,413	54.0	1,478	33.1	4,471
Total													12,709
Night, Usage Less Than Once a Month													
I	0		11	4.0	18	6.5	19	6.9	177	64.4	50	18.2	275
II	0		3	3.6	15	18.1	0		46	55.4	19	22.9	83
III	8	8.0	1	1.0	11	11.0	0		60	60.0	20	20.0	100
IV	5	1.8	0		15	5.4	7	2.5	217	77.8	35	12.5	279
Total													737
Night, Usage at Least Once a Month													
I	4	0.5	11	1.5	60	8.1	33	4.4	338	45.4	295	39.7	744
II	0		1	0.4	19	7.1	0		146	54.5	102	38.1	268
III	10	3.6	0		12	4.3	0		164	58.8	90	32.3	279
IV	38	5.8	2	0.3	59	9.0	2	0.3	357	54.3	197	29.9	658
Total													1,949

Table 3 gives the characteristics of the drivers interviewed during the four phases. A comparison of the percentage distributions reveals that there is a marked likeness among the groups of drivers in each phase.

The answers to question 12 "What first told you that you were approaching your exit?" are given in Table 4. These data indicate that the primary first notification of a coming exit was the advance destination sign, as might have been anticipated. A significant difference does exist, however, between the less-than-once-a-month user (approximately 68 percent) and the at-least-once-a-month user (about 54 percent). The data also show a high reliance by the familiar driver on advertising signs or landmarks (about 33 percent), as opposed to the less familiar (about 20 percent). An average of 7 to 10 percent of the drivers seems to have been first notified of the exit by the destination sign at the ramp. The other items mentioned may have been caused in part by a misinterpretation of the question. Little change occurred among the phases in the percentage of drivers mentioning the advance destination sign, as might be expected inasmuch as no changes were made in these signs throughout the study. Very little use appears to have been made of route marker guidance; however, crossing numbered routes occurs at only four interchanges in the study section. Differences in the groupings on a day and night comparison appear insignificant.

Table 5 gives answers to question 13 "What told you that you were at your exit?" The ramp destination sign was the major answer given and had the greatest impact on the daytime drivers who noticed this sign when it was changed to blue background just before Phase IV. The more frequent users relied more than the less frequent users on general interchange area appearance, which is as expected. The point at which the blue edge markings were installed is reflected in the great increases (5 to 11 times) in percentages of drivers in all 4 groups who mentioned this item between Phases II and III. Drivers do not appear to have recognized delineators, as such; only a few frequent

TABLE 5

SIGNS AND MARKINGS THAT GAVE NOTIFICATION OF ARRIVING AT EXITS AND THE NUMBER AND PERCENTAGE OF DRIVERS NOTIFIED BY EACH

Phase	Edge Marking		Delineators		Exit Arrow Sign		Ramp Destination Sign		Route Marker		Ramp Design		Interchange Area		Total
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	
							Day, Usage Less Than Once a Month								
I	45	2.0	1		529	23.6	1,265	56.4	54	2.4	129	5.8	218	9.7	2,241
II	11	1.5	1	0.1	224	30.9	341	47.1	5	0.7	50	6.9	92	12.7	724
III	134	17.0	1	0.1	197	25.0	335	42.5	4	0.5	68	8.6	50	6.3	789
IV	291	13.4	7	0.3	477	22.0	1,098	50.5	60	2.8	68	3.1	172	7.9	2,173
Total															5,927
							Day, Usage at Least Once a Month								
I	204	4.3	7	0.1	911	19.1	1,866	39.1	62	1.3	430	9.0	1,290	27.0	4,770
II	76	4.3	5	0.3	322	18.3	685	39.0	3	0.2	232	13.2	434	24.7	1,757
III	551	29.6	9	0.5	327	17.5	515	27.6	2	0.1	202	10.8	258	13.8	1,864
IV	1,050	23.3	34	0.8	805	17.8	1,606	35.6	53	1.2	285	6.3	677	15.0	4,510
Total															12,901
							Night, Usage Less Than Once a Month								
I	12	3.9	7	2.3	94	30.8	137	44.9	9	3.0	4	1.3	42	13.8	305
II	4	4.8	0		13	15.5	52	61.9	0		5	6.0	10	11.9	84
III	39	39.0	2	2.0	14	14.0	40	40.0	1	1.0	3	3.0	2	2.0	101
IV	78	27.8	11	3.9	43	15.3	110	39.1	16	5.7	7	2.5	16	5.7	281
Total															771
							Night, Usage at Least Once a Month								
I	40	5.4	18	2.4	145	19.5	258	34.7	15	2.0	19	2.6	248	33.4	743
II	19	7.0	9	3.3	46	17.0	122	45.0	0		14	5.2	61	22.5	271
III	109	38.9	16	5.7	30	10.7	88	31.4	1	0.4	13	4.6	23	8.2	280
IV	287	43.0	44	6.6	56	8.4	186	27.9	11	1.6	25	3.7	58	8.7	667
Total															1,961

TABLE 6

FEATURE THAT ATTRACTED ATTENTION TO EXIT SIGN OR MARKING AND THE NUMBER AND PERCENTAGE OF DRIVERS ATTRACTED BY EACH

Phase	Destination Name		Location		Arrow		Appearance		White Color		Green Color		Blue Color		Yellow Color		Other		Total
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	
Day, Usage Less Than Once a Month																			
I	525	24.5	654	30.6	221	10.3	330	15.4	10	0.5	181	8.5	13	0.6	5	0.2	200	9.4	2,139
II	176	24.3	233	32.1	102	14.1	133	18.3	6	0.8	44	6.1	9	1.2	0		22	3.0	725
III	139	17.7	202	25.7	105	13.3	116	14.7	0		26	3.3	187	23.8	1	0.1	10	1.3	786
IV	475	21.9	519	23.9	224	10.3	266	12.3	3	0.1	25	1.2	619	28.5	1	0.04	39	1.8	2,171
Total																			5,821
Day, Usage at Least Once a Month																			
I	620	13.0	1,734	36.5	449	9.4	1,044	22.0	75	1.6	359	7.6	10	0.2	8	0.2	455	9.6	4,754
II	294	16.8	619	35.3	119	6.8	495	28.3	23	1.3	96	5.5	35	2.0	3	0.2	68	3.9	1,752
III	229	12.3	403	21.6	114	6.1	371	19.9	2	0.1	41	2.2	659	35.4	0	0.0	43	2.3	1,862
IV	595	13.2	957	21.3	290	6.4	811	18.0	3	0.06	37	0.8	1,749	38.8	2	0.04	59	1.3	4,503
Total																			12,871
Night, Usage Less Than Once a Month																			
I	48	15.7	90	29.5	63	20.7	42	13.8	6	2.0	30	9.8	2	0.7	4	1.3	20	6.6	305
II	24	28.9	12	14.5	5	6.0	22	26.5	5	6.0	7	8.4	3	3.6	0		5	6.0	83
III	24	24.0	12	12.0	2	2.0	12	12.0	1	1.0	1	1.0	46	46.0	0		2	2.6	100
IV	47	17.4	25	9.3	38	14.1	19	7.0	1	0.4	2	0.8	129	47.8	0		9	3.3	270
Total																			758
Night, Usage at Least Once a Month																			
I	47	6.4	292	39.5	105	14.2	145	19.6	22	3.0	64	8.7	1	0.1	14	1.9	49	6.6	739
II	44	16.4	74	27.6	16	6.0	67	25.0	16	6.0	23	9.6	6	2.2	6	2.2	16	6.0	268
III	43	15.4	44	15.7	7	2.5	38	13.6	1	0.4	8	2.9	128	45.7	1	0.4	10	3.6	280
IV	92	15.2	25	4.1	47	7.8	53	8.8	2	0.3	5	0.8	374	61.8	1	0.2	6	1.0	605
Total																			1,892

users at nighttime mentioned them. When the exit signs in the gore were changed to blue background between Phases I and II, only the less frequent user in the daytime noticed these to any degree (7 percent more in Phase II).

Table 6 gives the answers to question 14 "What drew your attention to the above-mentioned item?" (i.e., the item in question 13, Table 5). The most remarkable difference among the phases or the categories was the increase in the percentage of drivers noticing the blue color between Phase II and Phase III. The change to blue color in edge marking definitely caught the attention of drivers. This increase points up not only the effect of adding something different, inasmuch as very little attention was drawn to the exit signs when those were changed to blue background, but also the importance of adding edge markings on ramps. Also surprising is the extremely small reference to white color; however, answers given in Table 5 only minutely referred to edge marking in Phases I and II. The extent that blue was mentioned increased successively during the phases for frequent users at night. The marked increase in the number of night drivers, both frequent and not-so-frequent users, who mentioned blue would indicate that the blue edge marking shows up more pronounced at night than in day. In addition, the added exposure of the color scheme brought an increased awareness of the blue involvement. By and large, all the other features were noticed less when increased attention was drawn to blue. The green color was mentioned by about 9 percent of the drivers in Phase I but dropped successively each phase to approximately 1 percent for both day and night as attention to blue increased. On the basis of the answers to question 13 given in Table 5, it is not surprising that destination name, location, and appearance were often mentioned, as these features relate to the fact that the sign or marker was there and therefore drew some attention. Their percentages, however, decreased as those for blue increased. The mention of the arrow was apparently linked in most cases to the ramp destination sign. Some of the

TABLE 7

PERCENTAGE OF DRIVERS WHO MENTIONED BLUE AND KNEW ITS CORRECT MEANING

Group	Mentioned Blue	Knew Correct Meaning
Once a year or less		
Day	30	62
Night	55	83
More than once a year		
Day	40	76
Night	55	83

TABLE 8

PERCENTAGE OF DRIVERS WHO FELT EXITS WERE ADEQUATELY MARKED

Phase	Adequately Marked	Not Adequately Marked
I	93	7
II	94	6
III	96	4
IV	87	3

large variations in Phase II and III data are caused by the small samples occurring when the data were broken down into the different categories.

When a driver mentioned a color in responding to question 14, he was then asked to indicate what the color meant to him. The responses are given in Table 7 for the color blue. It would seem that the more frequent user would more likely mention the blue color and also be somewhat more familiar with the correct meaning of the blue; however, the nighttime stranger seemed to be as correct as the frequent user in knowing what the blue color meant. The percentage of drivers mentioning blue is still significant as the mentioning of any color was entirely voluntary.

The responses to question 15 "Do you feel that this exit is adequately marked?" are given in Table 8. It may be interpreted that for only 7 percent of the drivers to report an inadequacy in exit marking means little; however, it is also true that only a very small percentage of any group of drivers has an accident caused by any number of factors, one of which certainly is confusion. In any case, significant reductions are present with each successive phase except from Phase III to IV (a 2 by 2 chi-square test was used). In answer to question 11, more drivers in Phase IV stated some difficulty in finding their exits than would be indicated by these percentages; however, the Phase IV group of 8.1 percent who still reported some difficulty was a large reduction from the 18.1 percent in Phase I.

Three signs in the northern half of the study area identified the section with the experimental color markings and informed the driver that blue indicates exit and yellow, entrance. Nothing in the data showed any difference in the drivers' knowledge of the scheme by having this portion signed. Perhaps this is because a high percentage of the route users are very frequent users and are therefore rather familiar with the whole area.

ENTRANCE DISTRIBUTION GROUPS

The driver groups were studied and reviewed by frequency of use and by individual access points. Nearly 70 percent of the drivers who exited in the study area entered at the beginning of the study area. Approximately half of these were also very frequent users of US-23 and likely very familiar with the route. The other entry points were rather uniformly divided except that a somewhat larger percentage (5.1) of drivers entered from I-96.

The replies to question 17 regarding adequacy of entrance marking changed very little with each succeeding phase. These replies and those to question 18 "What (entrance) markings did you notice?" are given in Table 9. Some slight evidence of the color scheme is perhaps reflected in the changes between Phases II and III in the percentage of drivers who noticed delineators and edge marking. This seems to point up however, that (a) signs provide most of the

TABLE 9

PERCENTAGE OF DRIVERS ACCORDING TO RESPONSES TO QUESTIONS REGARDING ENTRANCES

Phase	Not Adequately Marked	Marking Noticed at Entrance			
		Delineators	Edge Marking	Signs	Other
I	7.5	0.6	1.7	82.7	14.9
II	6.3	0.3	2.1	86.4	11.1
III	7.0	0.7	2.4	85.7	11.2
IV	7.1	0.9	4.3	83.4	11.4

direction necessary for entering vehicles and (b) very few entrance problems are connected with a rural facility of this type carrying the present volumes.

ACCIDENT DATA ANALYSIS

The accident data for the study section of US-23 cover 1 year before the color coding was used (July 21, 1964, through July 20, 1965) and 1 year after the complete scheme was installed (August 23, 1965, through August 22, 1966). The data include all accidents that occurred within one-quarter mile in each direction of the crossroad and all accidents on ramps or involving ramp terminals at the crossroad. Table 10 gives the number and location of accidents, which are quite randomly distributed. All of the usual accident involvements occurred, from going asleep to striking deer. Comments are given for those accidents involved with ramps in the northbound direction and for the 13 icy-bridge accidents at the Silver Lake Road overpass, which also crosses a railroad. This is an example of how a single factor can at times drastically change the accident picture at a location.

None of the accidents can be positively related to the addition of the color coding, and it is probable that the color coding affected accidents beneficially. The northbound accidents increased 21 percent, the southbound accidents increased 69 percent (where no changes were made), and the before-to-after total accidents increased 40 percent.

US-23 THROUGH-TRAFFIC INTERVIEW STUDY

The through traffic study was conducted in the summer of 1966 to determine the degree that through drivers noticed the presence of the color-coded interchanges and, if they did notice them, whether they understood the intent of the application.

TABLE 10

ACCIDENT DISTRIBUTION IN STUDY AREA

Location	Before		After		Ramp Accidents[a]		
	Northbound and Southbound	Northbound Only	Northbound and Southbound	Northbound Only	Before or After	No.	Comment
Territorial Road[b]	7		5			2	At ramp terminals
Six-Mile Road	7	5	7	5	After	1	At ramp terminal, northbound
Barker	0		0				
Eight-Mile Road	6	5	5	3			
M-36	4	2	4	1	After	1	Wrong-way
Silver Lake Road	4	4	2	2	Before	1	Backing when missed exit
Lee Road	1	1	9	3	Before	1	Out-of-control on exit
					After	2	Lost control on exit: 1 racing, 1 blowout
I-96	15	6	18	7	Before	2	Confused in direction
					After	1	Lost control exiting too fast
						1	Struck in rear slowing at ramp
M-59	10	5	6	2	Before	3	Rear-end at ramp terminal
						1	Right-angle at ramp terminal
					After	1	Rear-end
						1	Right-angle at ramp terminal
Clyde	1	1	2	2	Before	1	On slippery bridge
Center	2	1	4	4	Before	1	On slippery bridge
White Lake Road	2	1	3	1			
Owen Road	0		4	2	After	1	Entering
Silver Lake Road	4	2	17	7	Before	2	On icy bridge northbound
						2	On deck of icy bridge southbound
					After	5	On icy bridge northbound
						4	On icy bridge southbound
Torrey Road	5	2	4	3	After	1	Rear-end and head-on in left turn at ramp terminal
Thompson Road	3	3	6	4			
Grand Blanc Road	4	3	4	2	Before	1	Missed exit and turned around
						1	Rear-end at ramp terminal
					After	1	Right-angle at ramp terminal
Hill Road	4	2	6	4			
Total northbound		43		52			
Total southbound	29		49				
Total	72		101				

[a]Also includes 13 accidents on icy bridge. [b]This interchange was unchanged.

Drivers were interviewed near the north end of the study area such that the color coding would have had maximum exposure to all drivers while passing the various interchanges.

The driver was directed into the interview lane, informed that the Michigan Department of State Highways was performing a traffic survey, and asked whether he would mind answering a few questions. The interviewer then completed items 1 through 9 on the interview form shown in Figure 5. Up to this time, no questions were asked of the driver. The interviewer was instructed to record only the driver's answers with no prompting.

Table 11 gives the responses to the question relating to items noticed. The delineators and edge markings were noticed much more than any of the signing. It would seem that the recognition was primarily of the edge markings rather than the delineators as the driver did not seem to recognize them per se. As expected, the infrequent user was much more familiar with the blue color at exits than with the yellow color at entrances, because as he passed interchanges the entrance markings were not

US-23 COLOR INTERVIEW SHEET
THRU STUDY

1. INTERVIEW NO. 1 ☐☐☐☐ 4. DATE 9 ☐☐☐☐☐☐
2. RECORDER (assigned letter) 5 ☐ 5. HOUR (service time) 15 ☐☐
3. LOCATION (number from map) 6 ☐☐ 6. LIGHT CONDITION (D or N) 17 ☐
7. LICENSE – Michigan use first two letters & numbers; Out-of-state first two letters state name 18 ☐☐☐☐
8. AGE 1. 25 & under 2. 25–40 3. 40–55 4. 55 & over (estimate) 22 ☐
9. SEX (M or F) 23 ☐

10. HOW OFTEN DO YOU TRAVEL THIS ROUTE? 24 ☐
 1. Once a year or less
 2. 2 or 3 times a year
 3. 3 to 12 times a year
 4. Once a month or more

11. WHERE DID YOU ENTER US-23? 25 ☐☐
 Use number of interchange on map. Use OB for beginning of area.

12. DO YOU FEEL THAT ENTRANCE WAS ADEQUATELY MARKED? (Y or N) 27 ☐
 If answer is no, what would you suggest to improve the marking of this entrance? ______

13. WHAT DID YOU NOTICE WHILE DRIVING ON THIS ROUTE? Item / Color
 (a) Delineators 28 ☐ 29 ☐
 (b) Edge marking 30 ☐ 31 ☐
 (c) Signing 32 ☐ 33 ☐
  1. Exit arrow sign 3. Destination sign advanced
  2. Destination ramp sign 4. Route marker
 (d) Color coding information sign Y if mentioned 34 ☐
 (e) Geometrics 35 ☐
  1. Ramp design 2. Interchange area
 (f) Other Y if mentioned 36 ☐

Coding for colors
1. White
2. Yellow
3. Blue
4. Green
5. Red

What did this color mean to you? ______

Figure 5. Interview form used for US-23 through-traffic interview study.

TABLE 11

ITEMS AND COLORS AND THE NUMBER AND PERCENTAGE OF THROUGH DRIVERS NOTICING EACH

Item[a]	Color[a]	Once-a-Year-or-Less User		More-Than-Once-a-Year User	
		No.	Percent	No.	Percent
	Mentioned a color	249	52	191	42
Delineators and edge marking	Mentioned blue	234	64	211	59
	Liked blue	41		42	
	Objected to blue	3		5	
	Had no opinion	190		164	
	Associated blue with exit	191		161	
	Mentioned yellow	70	19	99	28
	Liked yellow	14		21	
	Objected to yellow	2		8	
	Had no opinion	54		78	
	Associated yellow with exit	25		74	
Signing[b]	Mentioned blue	60	17	46	13
	Liked blue	7		5	
	Objected to blue	0		1	
	Had no opinion	53		40	
	Associated blue with exit	42		29	

[a]All items mentioned by a driver were recorded separately; only one color for the same item was recorded even though more might have been mentioned.

[b]Includes gore signs, ramp signs, and advance destination signs and route markers.

particularly obvious nor were extensive changes made in the entrance scheme. The once-a-year-or-less users came mainly from the beginning of the section or at interchange 8, and only those entering at interchange 8 would have passed a full entrance scheme. This group seems to have readily understood the blue exit scheme; 82 percent of those mentioning blue associated it with the exit.

The items mentioned by the more frequent users were more numerous, but the percentages were rather close to those of the infrequent users. This group did associate yellow with the entrances much more than the unfamiliar drivers, as one would expect.

The degree that the nonexiting driver observed and commented on the color scheme indicates it may in fact be of some assistance to him by better identifying the exit areas such that he can more easily stay on the through path.

CONCLUSION

The pilot study on US-27, which was the forerunner of this extensive review of a color-coding application, definitely showed that erratic-driver movements were reduced at two left exits, apparently a result of reduced driver confusion.

This study on US-23 has shown that reductions in driver confusion can be reduced also at more conventional exits. Reductions in erratic movements were larger at other than diamond ramps. That is to say, movements at the more intricate interchanges were aided more than those at a simple diamond type. The study also shows that the color scheme was equally as effective in the daytime as at night.

Edge markings were noticed very little before they were changed to blue at the various exits. The major item that drivers noticed at exits was the ramp destination sign, and this sign was noticed more by the less familiar driver. When changed to blue, these signs were noticed more at night than in daytime; apparently the edge marking part of the scheme stands out more forcibly at night. The color scheme seems to have been readily understood as even 66 percent of the once-a-year-or-less users knew its correct meaning in daytime and 83 percent of the night group understood it.

The major area where guidance is needed appears to be at exits; the entrance areas pose few problems. This is based on information received from the interviewees, nearly 70 percent of whom were local drivers rather familiar with the route. Erratic-movement studies or other specific studies were not made at entrances, however. The

placement of painted posts and yellow edge marking at entrances resulted in very little notice of these changes.

The accident studies from 1 year before the color coding to 1 year after seemed inconclusive, as accidents were not directly relatable to the color application. This stems in part, however, from insufficient information on the accident reporting forms. The fact remains that the increase in accidents in the color-coded direction was only one-third as large as that in the uncoded direction, which was marked by current standards.

The through-traffic study bears out the fact that this scheme was readily understood by those (about 80 percent) who noticed the color scheme even though they were just passing through the area.

The application of a color scheme to 18 interchanges over the 40-mile section has produced some beneficial results and seems to be readily identifiable by the motorist. Dozens of users have in fact testified in correspondence to the highway department that they feel the color-code approach is logical and beneficial to them and that they would like to see the scheme extended statewide.

RECOMMENDATIONS

In 1970 AASHO has recommended certain new applications of color to markings and signs, none of which has apparently been placed or adequately studied. Because benefits from making the driver's task easier through color coding are imminent, it is logical to investigate the proposed schemes further, at least to determine whether any changes would be an improvement.

One element of the color scheme, the reflective granules used in the blue edge marking, was expensive to place. If corresponding benefits can be derived from applications of more economical materials, then a change from some of the present standards becomes far more feasible.

The proposed further review of color-coding applications seems to be a very reasonable extension to the investigation of simplifying the driving task and thereby contributing to overall route safety. We heartily agree with AASHO's approach that looks toward a modification of existing standards, as it seems highly improbable that the best method for identifying exits and entrances is to use identical color-marking schemes.

REFERENCES

1. Interchange Ramp Color Delineation and Marking Study. Michigan Department of State Highways, Lansing, 1965.
2. The Effect of Color in Guidance of Traffic at Interchanges. Minnesota Department of Highways, St. Paul, 1963.
3. Fitzpatrick, J. T. New Techniques in Interchange Traffic Guidance. World Traffic Engineering Conference, Washington, D. C., Aug. 1961.

Street Name Signs for Arterial Streets

DONALD L. WOODS and NEILON J. ROWAN,
Texas Transportation Institute, Texas A&M University

This paper presents the results of an engineering analysis of the street name signing problem to meet driver informational needs on modern arterial streets. Evaluation of the relative contrast between the letter and sign background for various combinations of letter color and background color are discussed, and an analytical analysis of the letter size required for the driver to properly execute a left-turn maneuver onto an intersecting roadway is presented. Alternative designs for providing street name information to the driver are discussed, and a design procedure that considers the driver's perception-reaction time and visual acuity and the physical characteristics of the intersection is presented.

•THE HISTORY of the development of arterial streets in this country is one of remarkable change. From the narrow two-lane streets of 30 years ago to the broad six- and eight-lane streets of today, there has been considerable progress in the design and operation of arterial street systems. Unfortunately, street name signing practice has not kept pace with the modern trends. The standards developed for 25 mph operation on streets that did not have special turn lanes and on which the traffic volumes were relatively light are still being used. A few cities have attempted to realistically treat street name signing as a part of the basic redesign of urban streets, but such activities have been the exception rather than rule. This paper will discuss street name signing in relation to traffic operation and will suggest logical minimum design standards for street name signing for modern geometric design standards and operating speeds.

Arterial streets are designed with a wide variety of geometric features that the driver is expected to negotiate in a "smooth and natural manner". It is generally assumed that the driver knows where he is going or will be able to pick up the necessary information en route. From recent field studies conducted by the Texas Transportation Institute in several states, it has become very clear that operation in a smooth and natural manner is quite difficult for the unfamiliar driver. The inability to locate or read street name signs forces the driver to commit himself to a maneuver without being certain of the name of the intersecting street ahead. Often, this results in his being in a turn lane at an intersection at which he did not desire to turn or at the intersection desired but in the wrong lane to make the desired turn. Either situation could result in the driver's performing a rather erratic maneuver that is hazardous to both the driver and other motorists.

It can be said that unfamiliar drivers constitute a relatively small percentage of the traffic stream, and thus the concentration on devices primarily for this group is not warranted. However, if only 2 percent of the traffic on an arterial street with a traffic load of 15,000 vehicles per day is unfamiliar with the route, 300 drivers are potential accidents. Traffic accident reduction is the result of improvements that reduce the accident potential by a small amount but are significant when applied on a large scale.

SIGN REQUIREMENTS AND DESIGN

One of the five elementary requirements of any traffic control device is that it should give adequate time for response. This requirement carries with it three specific design requirements for arterial street name signs.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 49th Annual Meeting.

1. The sign must have sufficient background contrast so that the driver can pick it out in the complex driving environment (target value).

2. The sign must have sufficient contrast between the letters and the sign background so that drivers can read the mesage easily.

3. The message must be conveyed a sufficient time in advance of the intersection to permit the driver to make a "smooth and natural" maneuver into the turn lane.

TABLE 1

COLOR COMBINATIONS OBSERVED ON STREET NAME SIGNS

City Code	Letter Color	Background Color	Relative Contrast, Unreflectorized Material	Relative Contrast, Reflective Sheeting
A	White	Black	–	–
B	Black	White	–	–
C	White	Green	6.14	8.32
D	White	Yellow	0.47	1.16
E	White	Orange	4.00	N/A
F	White	Red	3.76	3.10
G	White	Blue	4.00	17.64
H	Black	Yellow	–	–
I	Black	Orange	–	–

Sign Background Color

The primary factors in obtaining a high target value are the size of the panel used and color of the sign background. The size of panels needed to accommodate the letter sizes associated with the driver vision requirements are generally sufficient to produce an acceptable target value, and thus the background color becomes the dominant factor.

Several color combinations for street name signs have been observed throughout the country. Table 1 gives a summary of these observations. These background colors were selected for several reasons, including the following: aesthetic value (to match beach sand), local school colors, and economics. However, an engineering evaluation is appropriate, particularly in relation to their effects on traffic operation.

As indicated previously, driver response time is a function of target value and legibility. In turn, these factors are dependent on contrast of the sign colors and contrast of the sign with its environment. Of the color combinations in Table 1, the black and white combination offers the greatest color contrast, but it is probably the least desirable because of its poor contrast with the environment. This combination tends to blend with the environment, which is predominantly black and white.

Red, yellow, and orange are undesirable as background colors because of their association with other control devices. Thus, the logical background colors for street name signs are blue and green. From the standpoint of color association, green is the logical choice as most drivers tend to associate the green background with informational signing. The choice of green as the background color is also supported by the fact that the human eye is most sensitive to bluish-green color at night and yellowish-green under conditions of daylight illumination (1). Also, blue is relatively dark and has a somewhat lower target value than green. Figure 1 shows the relative sensitivity of the human eye to various wave lengths of light (2). Green then would have the greatest target value as a background color for a fixed sign.

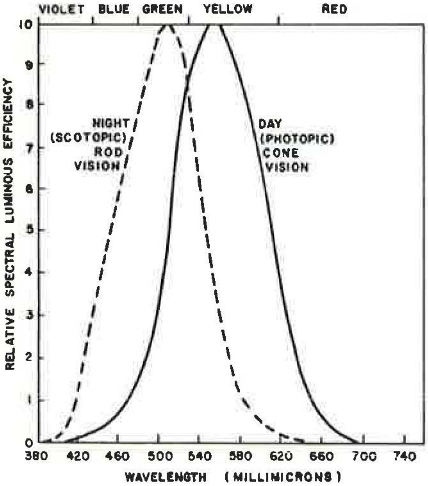


Figure 1. Human eye sensitivity to color.

Letter-Background Contrast

The relative contrast between the letters and the background is the vital factor in obtaining high legibility. The best contrast exists when a light letter color is used on a dark background. The reverse combination tends to "wash out" because of the diffusion of light from the lighter background. Ignoring the washout effect, the contrast between the letter and the sign background can be computed by the relationship

$$C = \frac{L - B}{B} \tag{1}$$

where

C = relative contrast,
L = reflectance of the letter, and
B = reflectance of the background.

The relative contrast values for several combinations of colors are given in Table 1. From these results, it is apparent that white letters on a blue or green background are the preferred combinations. Table 2 gives the reflectance values for various sign colors.

TABLE 2

REFLECTANCE VALUES FOR VARIOUS COLORS

Unreflectorized Materials		Reflective Sheeting[a]	
Color	Reflectance	Color	Relative Reflectance
Violet	0.05	High-intensity silver	450
Blue	0.20	Parkway silver	205
Green	0.14	Yellow	95
Yellow	0.68	Red	50
Orange	0.20	Green	22
Red	0.21	Blue	11
White	1.00[b]		

[a] 0 deg incidence angle and ⅓ deg divergence angle. [b] Assumed.

LETTER SIZE

Having established the optimum color combinations for street name signs, we should establish a procedure whereby letter size may be selected based on driver requirements. The proper letter size should provide the driver with sufficient legibility distance to read the street name sign and then safely maneuver to make a turn at the intersection. Perhaps the most critical situation is the separate left-turn lane situation shown in Figure 2. If this situation can be satisfied, then it is assumed that all other maneuvers can be safely accomplished.

The distance at which the driver must be able to read the sign to make a smooth maneuver is the sum of the three elements shown in Figure 2.

$$D = W + d_T + d_{P-R} \tag{2}$$

where

W = the distance from the face of the curb on the near side of the intersection to the street name sign on the far side;
d_T = the distance from the face of the curb on the near side of the intersection to the beginning of the transition to the left-turn bay; and
d_{P-R} = the distance traveled during the driver's perception-reaction time (the posted speed limit of the major street is used in this computation).

This distance can then be converted to required letter height by using the known distance per inch of letter height at which the driver will be able to read the letter.

To determine the legibility distance by using Eq. 2, we must make realistic assumptions regarding the character of traffic operation and the visual ability of the driver. These assumptions are as follows:

1. Vehicle approach speed equals the posted speed;
2. Turns are made at a speed of 15 mph;
3. Normal deceleration rate is 6 mph/sec;
4. Minimum perception-reaction time is 1.0 sec, and a desirable value for design is 2.5 sec;
5. A series D letter has legibility distance of 50 ft/in. of letter height for persons with 20/20 static visual acuity;
6. Minimum driver static visual acuity is 20/40; and
7. Street name sign is located on the far side of the intersection on the driver's right.

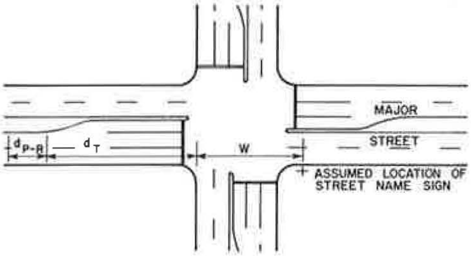


Figure 2. Major street with separate left-turn lane.

As a further explanation of these assumptions, it appears that the sign should at least be effective at the maximum legal speed. For

perception-reaction time, the 1.0-sec minimum value represents the alerted driver (4), whereas the 2.5-sec desired value is the design value recommended by current AASHO design policy.

The assumption regarding the legibility distance of the series D letter is based on work presented by Forbes and Holmes in 1939. The 50 ft/in. of letter height is valid for daylight conditions only and should be reduced to 40 ft/in. of letter height for nighttime conditions (Fig. 3). It should be noted that these values are for a static visual acuity of 20/20, and that they represent the 80th percentile of the distribution of the observed legibility distances. (A minimum static visual acuity of 20/40 is commonly used for the issuance of an operator's license.)

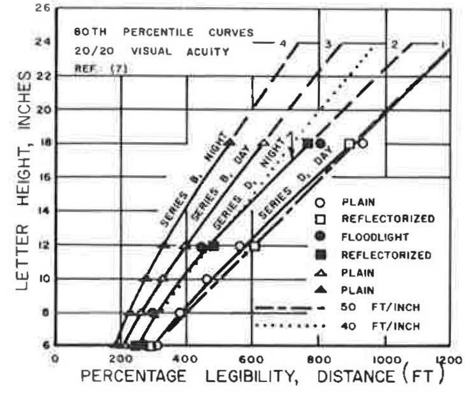


Figure 3. Legibility distances for series B and series D letters.

If the assumption outlined is considered, Eq. 2 may not be applicable in cases where the turn bay is very short or where there is no turn bay. To develop an expression to apply in these situations, we shall consider a condition with no turn bay where smooth operation depends on the normal deceleration of the vehicle. This minimum distance at which the driver must read the sign is expressed as

$$D_{min} = W + \left(\frac{V_2 + V_1}{2}\right)\left(\frac{V_2 - V_1}{6}\right) 1.47 + d_{P-R} \tag{3}$$

where

W = width of intersecting street;
$(V_2 + V_1)/2$ = average speed during deceleration;
$(V_2 - V_1)/6$ = time required to decelerate; and
d_{P-R} = distance traveled during the perception-reaction time.

The minimum letter height can be determined by using the 50 ft/in. factor previously discussed.

Equations 2 and 3 were used to prepare the design charts (Figs. 4 through 9) for determining the required letter size based on legibility distance. These charts were prepared by using posted speeds of 25, 35, and 45 mph and turn bay lengths (d_T in Fig. 2) of 0, 150, 200, 250, and 300 ft. Also, the computations were made for minimum and desired perception-reaction times of 1.0 and 2.5 sec respectively.

To illustrate the use of the design charts, assume that an arterial street has a posted speed limit of 35 mph, a 90-ft left-turn bay with a 60-ft transition, and a distance of 60 ft from the face of the curb on the near side to the sign. According to Figure 6, the minimum letter height for a 1.0-sec perception-reaction time is 5½ in. for a person with 20/20 static visual acuity and 10½ in. for a static visual acuity of 20/40. If the perception-reaction time is assumed to be 2.5 sec (Fig. 7), these values are 7 and 13½ in. respectively.

To further illustrate the use of the curves, consider the case of a four-lane divided street intersecting a major street with a 150-ft turn bay and a 90-ft transition. A perception-reaction time of 2.5 sec is desirable for the approach speed of 40 mph. The intersecting street is 64 ft wide, and the street name sign is placed 10 ft from the far side face of the curb. Thus, T_{P-R} = 2.5 sec, d_T = 240 ft, W = 74 ft, and A = 20/40.

Referring to Figure 7, we find that interpolation is necessary to determine the required letter height for 40 mph. The letter size required is 18 in. for an approach speed of 35 mph, and the corresponding value for 45 mph is 19 in. Linear interpolation

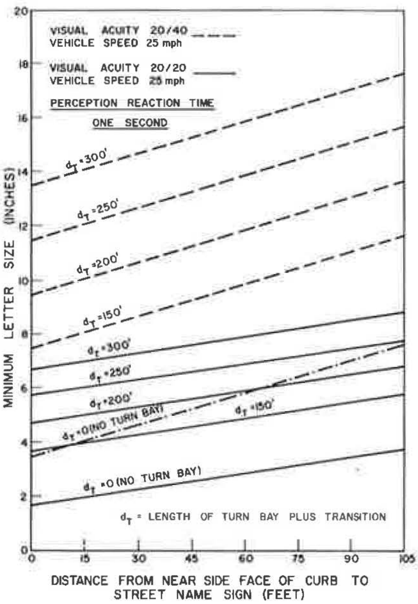


Figure 4. Letter size requirement for various turn bay lengths and driver visual acuity at 25 mph with 1-sec perception-reaction time.

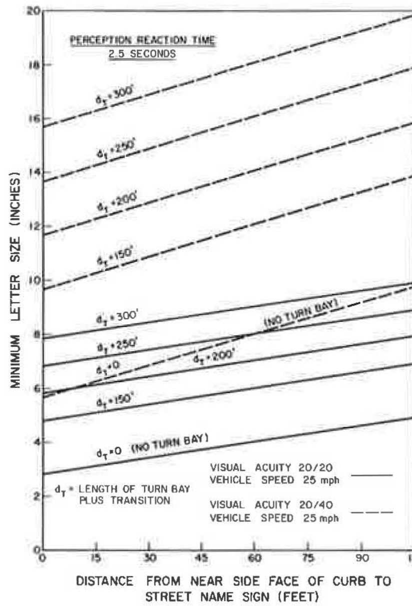


Figure 5. Letter size requirement for various turn bay lengths and driver visual acuity at 25 mph with 2.5-sec perception-reaction time.

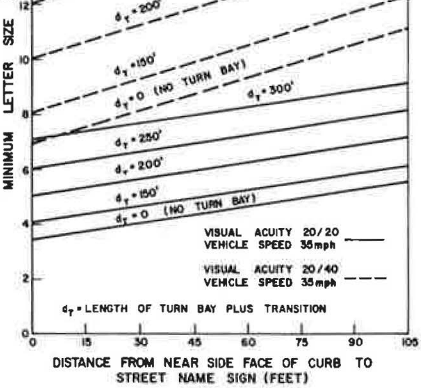


Figure 6. Letter size requirement for various turn bay lengths and driver visual acuity at 35 mph with 1-sec perception-reaction time.

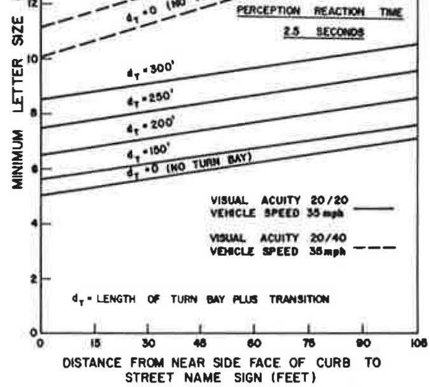


Figure 7. Letter size requirement for various turn bay lengths and driver visual acuity at 35 mph with 2.5-sec perception-reaction time.

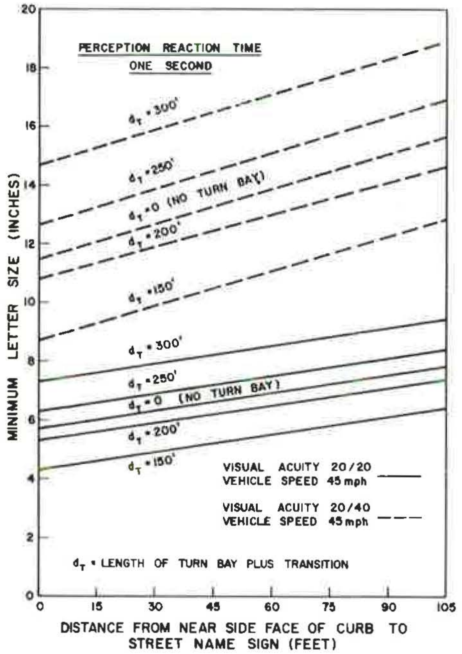


Figure 8. Letter size requirement for various turn bay lengths and driver visual acuity at 45 mph with 1-sec perception-reaction time.

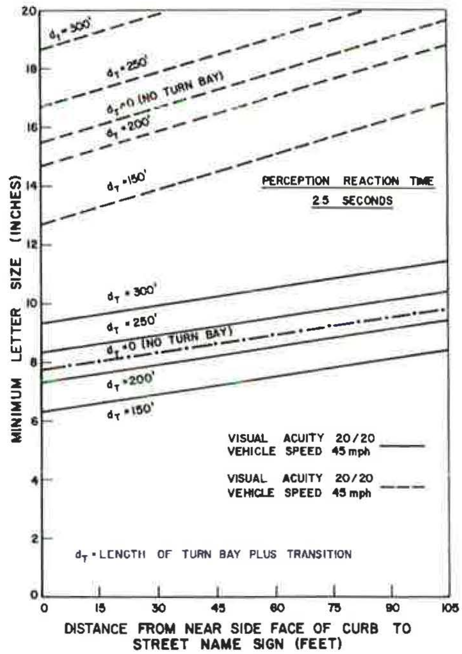


Figure 9. Letter size requirement for various turn bay lengths and driver visual acuity at 45 mph with 2.5-sec perception-reaction time.

yields a value of $18\frac{1}{2}$ in. for the 40 mph speed. A similar analysis for a perception-reaction time of 1 sec yields a letter height of 15 in. It is apparent that assuming 20/20 visual acuity will reduce the required letter size by 50 percent. It is appropriate, therefore, to discuss the criteria to be used in determining the required letter height.

LETTER DESIGN CRITERIA

The solution to Eq. 2 for perception-reaction times of 1.0 and 2.5 sec and various street widths are shown in Figures 4 through 9. A brief examination of these figures indicates that two interrelated variables are critical in determining the required letter size to meet the driver's needs. These are the assumption regarding the distance at which the letters can be read, and the assumption regarding the driver's visual acuity.

In reference to Figure 3, the assumption of a legibility distance of 50 ft/in. of letter height appears to be very accurate for series D letters under daylight conditions. At night, however, a figure of 40 ft/in. appears to be more appropriate. These values are the 80th percentile figures, which means that 20 percent of the population tested was unable to read the letters at the distances indicated. This would suggest that a value somewhat less than 50 ft/in. would be appropriate in design.

The visual capability of the drivers on the roadway is equally difficult to evaluate. The most common minimum uncorrected static visual acuity required to obtain an operator's license is 20/40. It is reasonable to assume that very few drivers actually have this reduced visual acuity; however, the 95 percent percentile value probably would not be less than 20/30.

The perception-reaction time for an alerted situation would be just slightly less than 1 sec (4). In the dense traffic on an urban arterial street, this value could scarely be expected to exist. A value of 2.5 sec would seem appropriate if one considers the driver's difficulty in locating the street sign in the complex urban environment while safely operating a motor vehicle. Thus, it is recommended that the following values be used

in the design of street name signs to meet the driver's needs:

1. Perception-reaction time is 2.5 sec;
2. Relative visibility of letters is 50 ft/in. of letter height (20/40 static visual acuity and 50 ft/in. of letter height is equivalent to 20/20 acuity and 25 ft/in. of letter height); and
3. Driver visual capability is 20/40 static visual acuity.

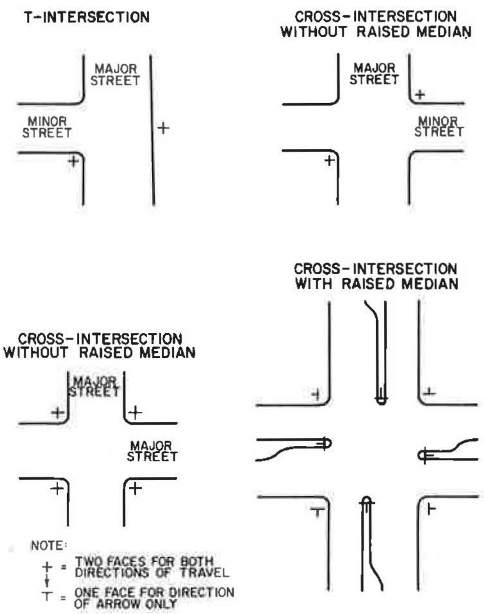


Figure 10. Suggested locations for street name signs.

Sign Location

The location of the sign is an important factor in the driver's ability to locate the sign in the environment. Recent research by the Airborne Instruments Laboratory established the importance of driver expectancy in the driving task (5). It has long been known that a sign placed on the right has a higher probability of being read by the driver than one placed on the left (1). This is undoubtedly due to the fact that a vast majority of all roadside signs are placed on the right and, therefore, the driver expects signs to be on the right. Recent unpublished data from driver interviews conducted in the field by the Texas Transportation Institute support the assumption that the driver first scans the right side of the intersection for the street name sign. Failing to locate it on the right, the driver successively scans the median area and then the left side of the intersection searching for the needed information.

The Manual on Uniform Traffic Control Devices (6) suggests that street name signs be placed on the "far side right" for the major street approaches. The sign so placed would be readily visible to the driver when stopped at the intersection. The letter size could be reduced by placing it on the near side of the intersection. However, it is apparent from Figures 4 through 9 that near side placement produces only a very slight reduction in the letter size and, thus, the far side location suggested in the Manual seems appropriate. Based on this discussion, we recommend the sign locations shown in Figure 10.

The special case created when the opposite sides of the intersection have different names is particularly difficult for the driver. Consistent with keeping the driver fully informed, street name signs for the approach should carry both street names along with directional information to locate the two streets. Figure 11 shows this special situation.

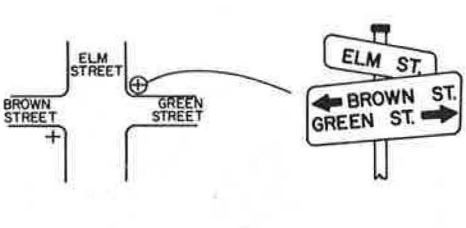


Figure 11. Suggested street name sign for special case in which street names on opposite sides of intersection are different.

The problems created when three or more streets intersect at a common point are very difficult to handle in regard to street name signing. If the street name signing is to be effective, the information must be conveyed before the intersection. A panel of the type shown in Figure 12 located 200 to 500 ft in advance of the intersection would be most effective in conveying the information. When there are more than three lanes in one direction, overhead installation may be desirable.

Minimum Letter Size

The design charts (Figs. 4 through 9) can be used to establish minimum letter sizes to

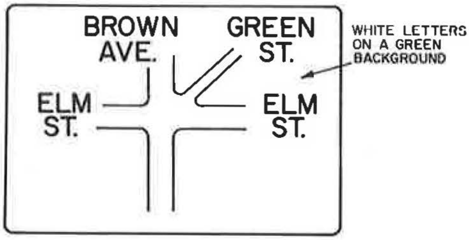


Figure 12. Advance signing for multilegged intersection.

TABLE 3

MINIMUM LETTER SIZE FOR VARIOUS APPROACH SPEEDS AND PERCEPTION-REACTION TIMES WITH 20/20 VISUAL ACUITY ASSUMED

Speed (mph)	Perception-Reaction Time	
	1.0 sec	2.5 sec
25	3 in.	4 in.
35	5 in.	6½ in.
45	7 in.	9 in.

be used in typical conditions. Table 3 gives these minimum sizes for different speeds, based on 20/20 visual acuity, a 50-ft-wide intersecting street, and the sign located 10 ft from the face of the curb. Data given in Table 3 show that a letter size of less than 6 in. would only be appropriate for very low operating speeds or very short perception-reaction times combined with near perfect vision. Conversely, a letter size of greater than 6 in. would be required only for relatively high-speed roadways and rather long perception-reaction times. The use of a minimum letter size of 6 in. on street name signs seems appropriate. It is important to note that the minimum letter size is computed assuming that no special turn lane exists.

Alternative Designs

There are two basic approaches to the problem of conveying the necessary information on intersecting street names to the drivers. These approaches are to (a) place large signs at the intersection, and (b) use advance street name signs to complement the smaller signs at the intersection proper.

Typical of the first alternative is the Metro sign design now being used in Tucson, Arizona. Figures 13 through 16 show the use of this type of design. These signs have 8-in. letters with a "D" stroke on a 16-in. background. Referring to Figures 4 through 9, it is apparent that this design is adequate for virtually all situations, assuming a static visual acuity of 20/20. If, however, a static visual acuity of 20/40 is assumed, the Metro sign would be inadequate for most arterial street operation.

In applying this concept, one must exercise care to ensure adequate night visibility. If the street name plate is placed too near the signal head (Fig. 17), the green signal indication will tend to "wash out" the sign and will make it extremely difficult to read. Another point of interest in Figure 17 concerns the illumination of the street name sign.

Figure 13. Metro sign for 150 ft.

Figure 14. Metro sign from 100 ft.

Figure 15. Close-up of Metro sign.

Figure 16. Relative size of Metro sign.

When the sign is mounted more than 10 ft above the pavement surface, external illumination is required to make the sign effective. There is some indication that the use of high-intensity sheeting could eliminate the need for illumination, and existing roadway lighting may be sufficient if it is properly located with respect to the sign.

The Metro sign has received very favorable acceptance by the public in Tucson and is a considerable improvement over the older designs.

ADVANCE STREET NAME SIGNS

Design and Location

In dense traffic on multilane arterial streets, the driver may not be able to search out the signs at the intersection because of the necessary concentration on the driving task. Here, advance street name plates are desirable (Fig. 18). Advance street name signs should be designed in accordance with the same criteria used in the design of street name signs at the intersection. For example, if the required letter height is 14 in. for a sign to be placed at the intersection only and the advance sign is to be placed 200 ft in advance of the intersection, the advance sign letter size should be 14 in. less 200 ft divided by 25 ft/in. (20/40 visual acuity and 50 ft/in. legibility distance) or 6 in. in height.

The location of the advance sign is very important in obtaining the desired degree of effectiveness. For one or two lanes in one direction, right-side or median mounting is more desirable. An example of this type of installation is shown in Figure 18. When there are three or more lanes in one direction, the requirements of the driving task will consume a large percentage of the driver's time and there is a high probability that one roadside sign will be missed. Therefore, advance street name signs should be placed

Figure 17. "Wash out" effect caused by Metro sign being placed close to signal head.

Figure 18. Advance street name sign.

Figure 19. Suggested design for advance street name sign.

Figure 20. Suggested design for advance street name sign for a street that is also a numbered route.

on both the left and right or overhead. Overhead installations are required where a raised-curb median is not used. Overhead installations may also be desirable when there are only two lanes in one direction if the roadside is intensively developed and curb parking is permitted. In such cases, an advance sign tends to blend into the roadside activities. All overhead installations should be externally illuminated.

Intersecting streets, alleys, and local driveways between the advance sign and the intersection to which it refers tend to be confusing to the driver, and it is essential that the driver be fully informed as to which intersection the advance sign applies. It is also important to distinguish the advance sign from the sign located at the intersection. It has been suggested that a different color of background might accomplish this goal. The use of many different background colors throughout the country and often two or more within one urban area make it difficult for the driver to associate background color with intended meaning. Therefore, the use of a supplementary legend is desirable. This legend should convey to the driver the distance or some reference to the intersection in clear and concise terms. The supplementary messages NEXT SIGNAL and 2 BLOCKS have proved to be very effective, whereas a supplementary message of the type 500 FEET has proved ineffective as the driver has little concept of distance in such exacting terms.

Suggested Minimum Letter Size for Advance Signs

In reference to Figures 4 through 9, it is apparent that, if the advance sign is placed 250 ft in advance of the intersection and if 6-in. letters are used, the driver requirements for a vast majority of all situations will be met. An advance placement of 350 ft or more would satisfy virtually all such situations. Therefore, it is suggested that the advance street name sign have white 6-in. letters on a green background with any supplementary message in 4-in. letters (Fig. 19).

According to some references, there may be an advantage in using a combination of capital and lower-case letters on street name signs, apparently because the driver is more familiar with this style inasmuch as it is used predominantly in printing. This combination does reduce the comprehension or perception time, but it does not appear to be a critical factor.

Another common problem in urban areas is the lack of identification on advance street name signs of streets that are also designated as highway routes. In many cases, the unfamiliar driver will be following route numbers and the street name may be of little value to him. Thus, it is important that the highway number be associated with the street name in the advance signing. One solution to this problem is shown in Figure 20.

SUMMARY

This paper deals with an analytical approach to the design of street name signs based on the driver's informational needs for smooth traffic operation. The major points of consideration in this analytical approach are summarized as follows:

1. The most desirable combination of colors for street name signs is white letters on a green background from both a target value and legibility standpoint (excluding any psychological response to color).
2. A perception-reaction time of 1.0 sec is too short for design on urban arterial streets; a value of 2.5 sec is more realistic.

3. A static visual acuity of 20/40 probably should be assumed in designing sign installations.

4. A minimum letter size of 6 in. should be used on all street name signs.

5. Advance street name signs should be placed a minimum of 200 ft in advance of the intersection, and a distance of 300 to 350 ft is desirable for 40- or 45-mph operation.

6. The advance street name sign should include the U.S. or state highway number when the street is also a designated highway.

REFERENCES

1. Traffic Engineering Handbook. Institute of Traffic Engineers, 1965, p. 88.
2. Kaufman, John E., ed. IES Lighting Handbook. Illuminating Engineering Society, New York, 1966, pp. 2-5.
3. Sears, F. W., and Zemansky, M. W. University Physics, Vol. 2, Electricity, Magnetism and Optics. Addision-Wesley Pub. Co., Inc., 1956, pp. 824-825.
4. Matson, T. M., Smith, W. S., and Hurd, F. W. Traffic Engineering. McGraw-Hill Book Co., New York, 1955.
5. Development of Information Requirements and Transmission Techniques for Highway Users. Airborne Instruments Laboratory, unpublished agency draft of NCHRP Project 3-12, November 1967.
6. Manual on Uniform Traffic Control Devices for Streets and Highways. U.S. Bureau of Public Roads, June 1961.
7. Forbes, T. W., and Holmes, R. S. Legibility Distances of Highway Destination Signs in Relation to Letter Height, Letter Width, and Reflectorization. HRB Proc., Vol. 19, 1939, pp. 321-334.
8. Product Bulletins 85A-3-(25S) and 102-1, Reflective Products Technical Service, Minnesota Mining and Manufacturing Co., April 1967 and Oct. 1969.