HIGHWAY RESEARCH RECORD

Number 221

Guidance Aspects of Colored Pavements and Pavement Markings

5 Reports

Subject Area

53 Traffic Control and Operations

HIGHWAY RESEARCH BOARD

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Foreword

Applications of traffic engineering are being given new scrutiny as a means of enhancing driving performance, reducing congestion and improving safety. Such measures are usually cheaper than construction of miles of new or improved highways. However, the successful application of new traffic engineering techniques often requires a process of education for both public officials and drivers. It is often difficult to measure the relative success or failure of some traffic engineering improvements in terms of safety and dollars. Constant experimentation and correlation takes place in the research and operating fraternity in attempts to prove or disprove uses of traffic engineering.

The five papers in this RECORD, all concerned with some aspect of traffic control devices for the most part, indicate experiments and evaluations with colored pavements and white and yellow pavement marking systems. Two of the papers evaluate aspects of traffic signals, such as length of clearance interval and value of flashers in reduction of accidents.

Those who are concerned with traffic engineering, design, and operations at Federal, state, and local governmental levels should find much information to stimulate their thinking. Some of the papers do present research, that if taken advantage of and applied, could result in lessening of traffic problems.

The first paper discusses the value of colored pavements used to guide traffic through intersections. Concentrating on left-turn problems at intersections with different geometrics, it was found that use of green pavement for the left-turn lane and yellow for the channelizing lane was effective in directing left-turn vehicles to enter the left-turn lane. Velocity patterns throughout the intersection did not change appreciably and at night it was found that the colors were barely visible under vehicle headlighting conditions.

The second paper is also concerned with colored pavement, in this case, red. A ramp intersection with a one-way road was studied, both as originally paved and then as paved with red. It was found that, using speed as a tool of differentiation, daytime speeds were significantly lower with the red pavement, but there were not any significant differences at night. Gap acceptance characteristics were about the same for both colors during night and day conditions.

Three systems of applying yellow and white pavement markings were studied in Ohio. Using a five-phase methodology to ascertain effectiveness of markings for the three different systems, it was found that it was possible to design a revised coloring system to convey more meaningful information to drivers. It was also found that drivers would need a period of education and adjustment to different systems, and that colored systems appear to have good potential. Three formal discussions of this paper bring out opinions of operating traffic engineers concerning these possible systems.

An extensive review of the problems of proper amber clearance interval at traffic signals is given in the fourth paper. It was found that increasing the amber phase from 3 to 5 seconds in urban areas increased the percentages of motorists driving unsafely. At rural locations, increase of the amber clearance period from 5 to 7 seconds decreased the percentages of motorists operating unsafely. Transverse pavement markings at both urban and rural locations tend to influence the respective percentages in the opposite direction. The author carefully points out the pilot study aspects of the research and its dependency on California law enforcement practices.

The last report attempted to determine existing relationships between accident reduction and application of flashing control devices. Statistically significant reductions in accidents were found when certain types of flashers were used at certain types of intersection configurations—generally under low traffic volume conditions. Flashing devices which consist of a horizontal or vertical "bouncing ball" produce the greatest reduction in accidents.

Contents

Colored Pavement for Traffic Guidance

WILLIAM C. TAYLOR, Wayne State University

This study was conducted to determine the value of colored pavement material in guiding traffic through intersections. The intersections chosen for study included three geometric types, each with left-turn lanes included as an element of the intersection. These intersections had experienced accident problems related to left-turn maneuvers. The introduction of color was limited to the island area preceding the left-turn area and the left-turn land itself. The island was paved with yellow asphalt and the turning lane with green.

The study consisted of evaluation of vehicle velocity through the intersection, lateral placement of vehicles entering the left-turn lane, and channelizing effect of through vehicles. A before-and-after procedure was used to analyze the effectiveness of color. The results of these analyses indicate that the application of color is effective in channelizing left-turn vehicles entering the left-turn lane. This is accomplished primarily by directing the vehicles past the approach island rather than by inducing the driver to enter the lane earlier. Coloring the pavement did not significantly change the velocity patterns through the intersection. Also, the effectiveness of the colored pavement is lost at night. In fact, the color is barely visible under the light from vehicles.

•THE trend toward increased speeds and the growing complexity of the highway network have created new problems for the driver. The driver, in his role as decision-maker, must assimilate and process information much more rapidly than before. Several research studies (1, 2, 3) are being conducted in an attempt to define the driver's decision-making capabilities as a function of time and information input rate. The results of these studies will provide additional insight into the need for improved means of communication at specified locations along the highway.

There is a need for concurrent research in the field of driver acceptance and recognition of communication aids. Thus, the identification of the need for increased communication can be supplemented with the knowledge of possible means of supply. The most common means of information transfer to the driver is through signs, signals, and pavement marking. The science of communication through these media has made great advances.

For example, studies have resulted in the recognition of the value of standardization as an aid in decision-making. This recognition eventually resulted in a requirement for such standards. Distribution of federal grants for highway construction will be contingent upon the respective states showing substantial conformance with the Manual of Uniform Traffic Control Devices (4) by December 1968. To this end, part of the HPS money in each state has been set aside for inventory and upgrading of signs and markings.

Standards of size, shape, color, and legend have been established for traffic signs throughout the country. These standards, as well as standards for the use of signals and pavement markings, have been published in the Manual. The need for standardization should not be ignored. As vehicle speeds increase, any time reduction in the

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decision-making process will result in increased safety. However, in this period of dynamic industrial and professional growth, some avenue for testing new ideas and products must remain open. The results of such tests can be used to periodically update our standards.

One of the basic elements of the standardization of traffic control devices is color. Certain colors have been adopted as having a specified meaning in traffic control. Red, green, and yellow are probably the best examples.

The National Joint Committee on Uniform Traffic Control Devices has named a special subcommittee on color to expand the list of colors and meanings to convey additional information to the driver. It appears that there are some 14 colors that are easily discernible and could be used in a standardized color-coding scheme (5). Research involving the application of these colors must be conducted to establish the value of expanding the use of color for information transfer.

Both controlled and noncontrolled experiments have been conducted with colors. Michigan, Washington, Minnesota, and Ohio have conducted studies on the use of colored delineators and edge lines to code the on- and off-ramps of freeway interchanges. A different color is used to distinguish the on-ramps, the off-ramps, and the through lanes. Some cities and states use colored route markers to help the motorist distinguish the proper marker within an array. Crosswalks, especially near schools, have been painted in some cities in an attempt to attract the driver's attention. The purpose of these studies is not to establish meanings for particular colors, but to determine the effect of using color for guidance.

Color contrast is another possible use closely allied to color coding. There are situations where the effectiveness of a device is independent of the particular color used so long as a contrast is provided. An example of this is the use of an asphalt overlay on the shoulder area of a concrete highway. This provides a white-black contrast which enables the driver to discern the pavement edge. In some cases where this color contrast was not provided, it was necessary to return and place painted lines on the berm to provide this distinction.

The use of black center lines and lane lines on concrete pavement in lieu of the standard white line is another example of the application of color contrast. The purpose in this case is to identify the location of the center of the pavement. When additional information such as the existence of a no-passing zone must be conveyed to the driver, the principle of color coding is used. The yellow no-passing line is used with both the black and white lines. While the preparation between color contrast and color coding can be established for some uses, the two are not always mutually exclusive.

The use of color for signs, signals, and pavement markings in an attempt to guide or control traffic is well established. The use of color for delineation or for sections of pavement has not yet progressed to this point. The use of colored sections of pavement as a means of providing direction and guidance to the driver is the subject of this study. At present, the study should be classified as one of color contrast, but care must be taken to avoid conflicts with established color codes.

A survey of the literature on the use of colored pavement materials indicates that the distinction between color coding and color contrast is not being maintained. In Chicago, yellow asphalt was used for median strips, whereas in New York it was used for a deceleration lane. Red asphalt has been used for entrance ramps, exit ramps, and approaches to stop signs at various locations. The primary function of each of these is basically color contrast. However, the diversity of uses will make it difficult to standardize a color code which is meaningful to the driver.

There appear to be two basic approaches to the problem of the extension of color coding. One is to standardize the meaning of various colors and then experiment with these color combinations to find situations in which they are effective. The other is to determine, through research studies, the situations where color contrast is valuable, and thus establish a set of standard colors to achieve this contrast. This project would be classified in the latter category. We have attempted to determine the value of the application of color contrast to a specific problem in traffic flow.

This project was designed to evaluate the use of colored pavement as a control and guidance device through intersections with left-turn slots. This evaluation includes an

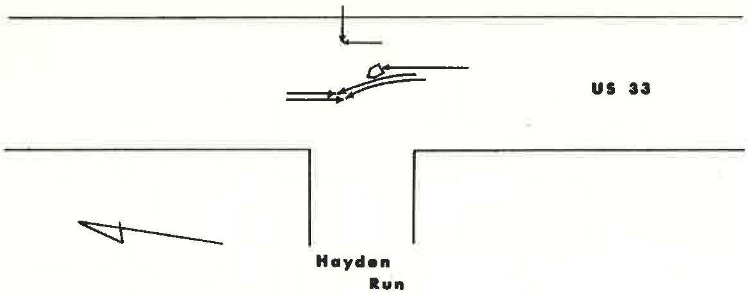


Figure 1. Two-year accident history for US 33 and Hayden Run Road.

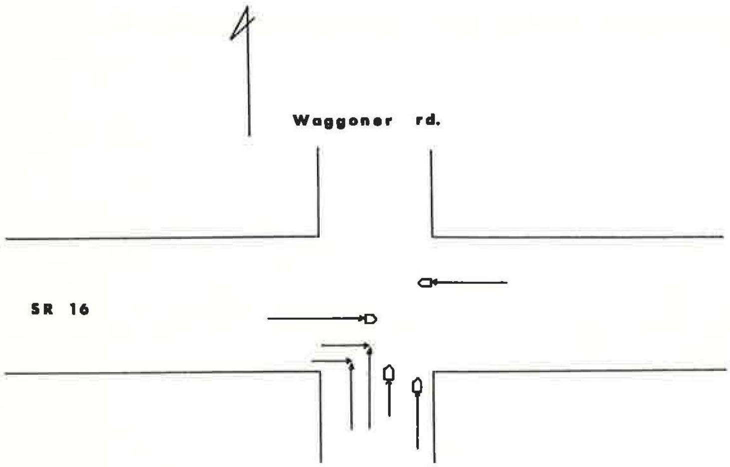


Figure 2. Two-year accident history for SR 16 and Waggoner Road.

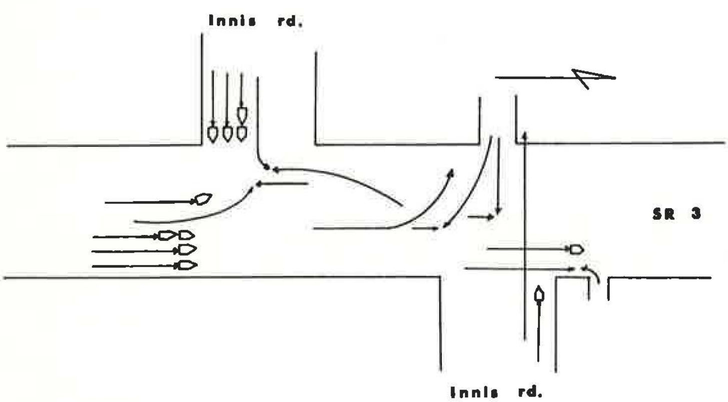


Figure 3. Two-year accident history for SR 3 and Innis Road.

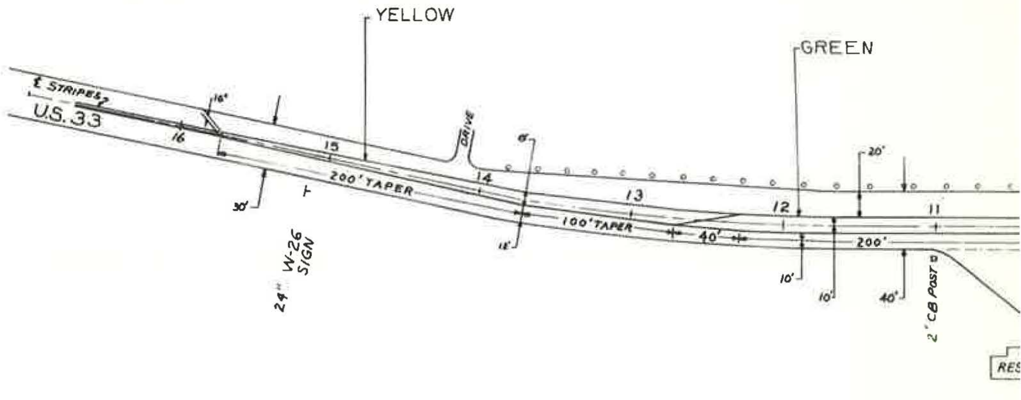


Figure 4. The intersection of US 33 and Hayden Run Road showing details of colored asphalt.

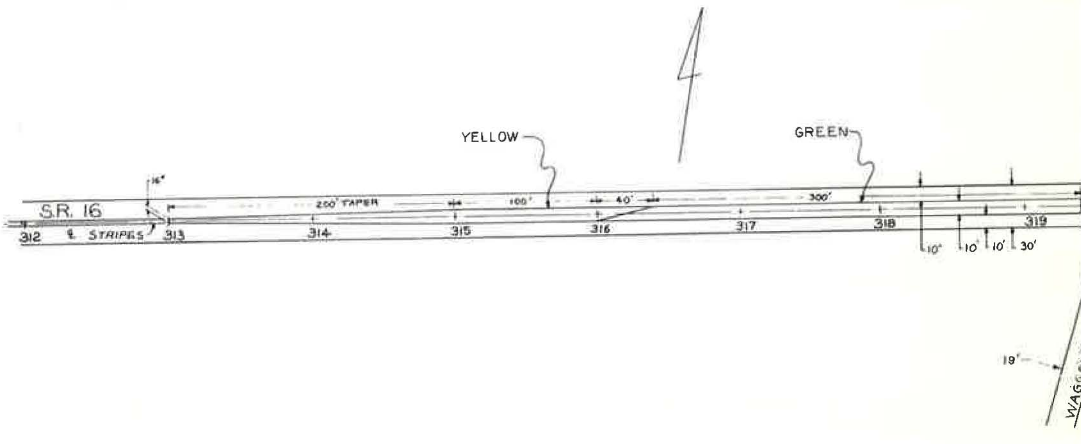


Figure 5. The intersection of SR 16 and Waggoner Road showing details of colored asphalt.

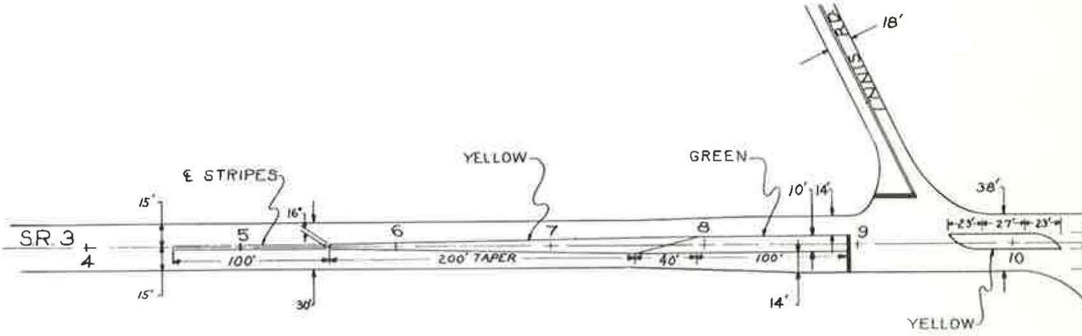
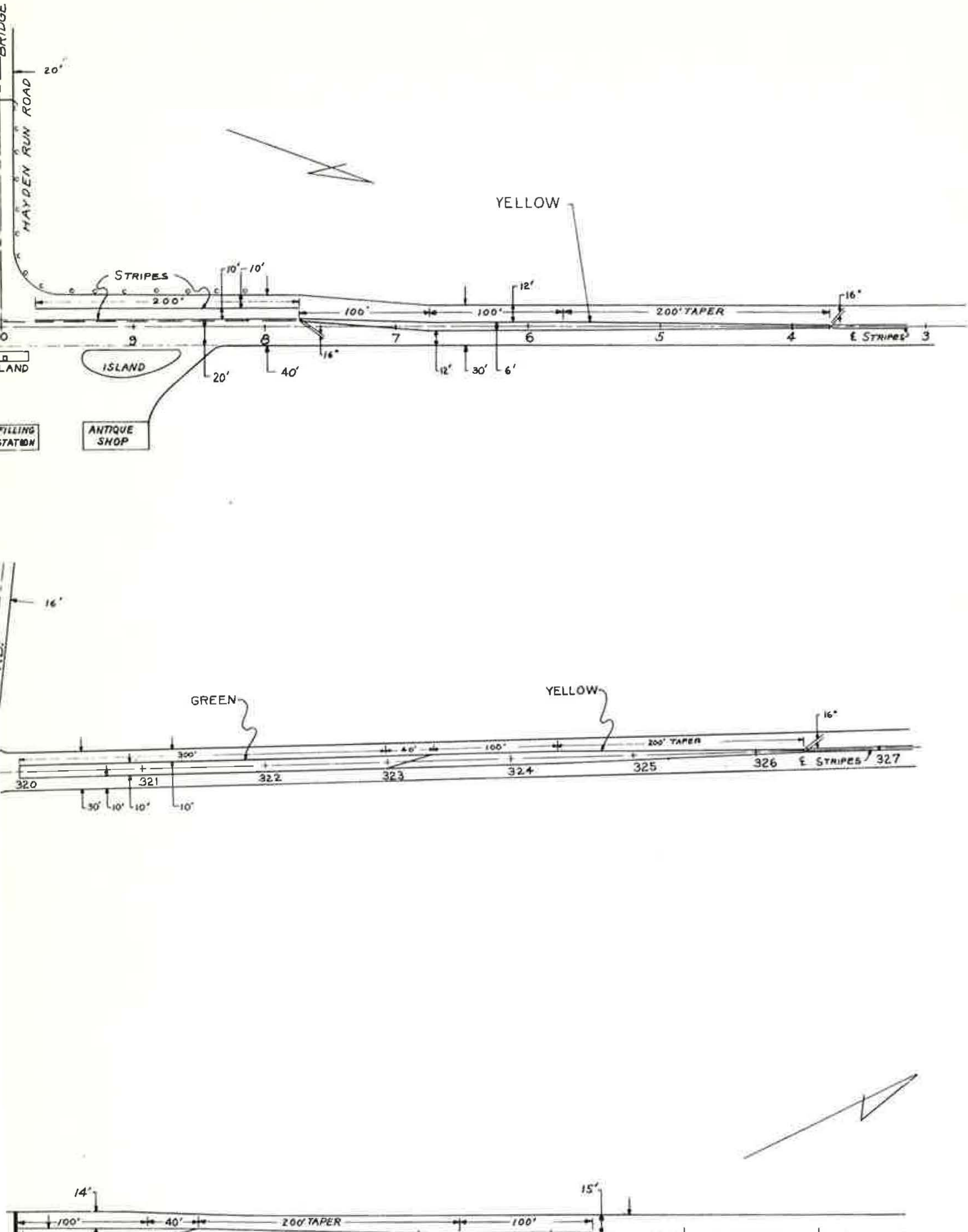


Figure 6. The intersection of SR 3 and Innis Road showing details of colored asphalt.



BRIDGE
HAYDEN RUN ROAD
20'
YELLOW
STRIPES
10'
10'
200'
100'
100'
200' TAPER
12'
16"
9
8
7
6
5
4
3
℄ STRIPES
ISLAND
20'
40'
16"
12'
30'
6'
FILLING STATION
ANTIQUE SHOP
16'
GREEN
YELLOW
16"
300'
40'
100'
200' TAPER
320
321
322
323
324
325
326
327
℄ STRIPES
30'
10'
10'
10'
14'
15'
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200' TAPER
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14'
10'
GREEN
YELLOW
16"
℄ STRIPES
15'
30'
20'
INNIS

analysis of changes in approach speeds, traffic flow patterns, and lane position. The study period established for this project is insufficient to include an analysis of changes in the accident experience. However, accident records will be maintained so that an analysis of this type can be made in the future.

PROCEDURE

The procedure used in this study was based on measurements of traffic flow before and after the installation of color. Comparisons of the data were then made and conclusions drawn from these comparisons.

Selection of Test Sites

The size of the project limited to three the number of intersections that could be paved with colored asphalt. To expedite the project and to include the desired variables within the three sites, intersections with different geometric features were selected.

The intersection of US 33 and Hayden Run Road was one of the sites selected for study. This is a "T" intersection, US 33 being the continuous leg. A copy of the accident history at this location for a period of 24 months preceding the installation is shown in Figure 1. The intersection of SR 16 and Waggoner Road was the second site selected for study. This is a four-leg intersection with a slightly skewed approach. The accident history for this intersection is shown in Figure 2. The third selection was a four-leg intersection with an offset of 80 ft between the two legs of the minor road. This location is the intersection of SR 3 and Innis Road. The accident history for this location is shown in Figure 3.

The design of the left-turn lanes was in accordance with the specifications in the Ohio Manual on Uniform Traffic Control Devices. These designs reflect the differences in approach speeds and turning volumes found at each of the three locations. The intersections are shown in Figures 4, 5, and 6.

Installation

This color was obtained by placing a ½-in. layer of colored asphalt over the existing pavement. Pavebrite II, a two-component synthetic binder plus pigment, was used in this study. The asphalt was placed as a standard maintenance project under the supervision of the Division Maintenance Engineer. The area cross-hatched with white paint in the standard treatment was covered with yellow asphalt, and the left-turn lane was covered with green asphalt.

The color section was based on existing practice within the Ohio Department of Highways, and in concurrence with the recommendations in the "Research Prospectus on Colored Pavements" prepared by the U.S. Bureau of Public Roads. The use of yellow as a barrier of indication of an area of caution has long been a practice. The Research Prospectus states that the color meaning should be:

> Yellow: The color yellow can be used to indicate areas of no trespassing. This would, generally, restrict its use to islands and medians.
>
> Green: The color green can be used to indicate areas of traffic merging and diverging or potential lane changing.

A left-turn lane was considered to be an area of traffic divergence and, thus, within the defined limits for the use of green.

RESULTS

Field Data

The collection of field data was designed to achieve the established objectives as nearly as possible. Vehicle speed, lateral placement, and the flow pattern are treated separately, but it would require the combination of these three to adequately describe the effectiveness of color as a guidance device.

TABLE 1

SPEED DATA FOR THE INTERSECTION OF SR 16 AND WAGGONER ROAD

Distribution	Before	After 1	After 2
(a) Eastbound—650 ft			
Average speed	46	45	49
15%	40	37.5	40
50%	46	44	48
85%	52	51	54
(b) Eastbound—300 ft			
Average speed	45	43	39
15%	33.5	34.5	31
50%	45	40	37
85%	54	46	43
(c) Westbound—650 ft			
Average speed	52	48	51
15%	46	41.5	45
50%	51	47.5	50
85%	53.5	57.5	56
(d) Westbound—300 ft			
Average speed	52	48	47
15%	40	41	38
50%	47.5	47	45
85%	54	54	54

The data on speeds and placement were collected at three different times, and the flow-pattern data twice. The time periods are referred to as "before," "after 1," and "after 2." The "before" data were taken approximately six months prior to the installation of the colored asphalt, the "after 1" data were taken in the first month following the installation, and the "after 2" data were taken six months after the installation.

Speed

Two vehicular spot speed checks were taken at the approach to each left-turn lane. The first check was made at a point approximately 600 ft from the intersection. This point was selected because it was the distance at which the driver could first see the island. The second speed check was taken at the point where the driver could first see the left-turn lane. This distance was approximately 250 ft from the intersection for the selected study sites. In all cases, this point was adjacent to the island. The factor of interest in the speed measurements was the initial effect on the driver. For this reason the speed distribution as well as the mean speed was used in the analysis.

The spot speeds were taken with a radar meter concealed in a mailbox. A 100-ft cord is used in conjunction with the mailbox so that the recorder can be concealed some distance from the meter. This arrangement was used to avoid the influence of the observer on the speed pattern. An analysis of the speed checks taken immediately after the application of the colored asphalt indicated generally lower speeds. In most cases the speed reductions were quite small and well within the variance normally associated with speed checks had there been no changes made.

Tables 1 and 2 give the values of four parameters of the speed distribution before and after the installation of the colored asphalt. Speed checks were not taken at the intersection of SR 3 and Innis Road because this is a signalized intersection. The columns titled "After 1" refer to data taken immediately following the installation, and the "After 2" columns refer to data taken six months later.

The 15 and 85 percentile points were chosen to determine whether the use of color affected the speed distribution. There are no significant speed differentials in any of the parameters recorded. Likewise, there are no apparent trends in the speed profiles when compared by location in respect to the intersection. The mean speed is lower at the location nearest the center of the intersection, but this is also true of the other parameters of the speed profile. The eccentricity or skewness of the distribution appears to be a property of the location and is not affected by the addition of color to the pavement.

The results of this speed study tend to dispel one possible criticism of the use of colored

TABLE 2

SPEED DATA FOR THE INTERSECTION OF US 33 AND HAYDEN RUN ROAD

Distribution	Before	After 1	After 2
(a) Northbound—600 ft			
Average speed	51	45	46
15%	43.5	37.5	40
50%	50	44	47
85%	57	49.5	54
(b) Northbound—250 ft			
Average speed	45	45	43
15%	36.5	37	35
50%	45	43	40
85%	52.5	49	49
(c) Southbound—600 ft			
Average speed	46.04	44	44
15%	38.5	37	37
50%	43.5	42.5	43
85%	51.5	49.5	48
(d) Southbound—250 ft			
Average speed	42	43	42
15%	34.5	36.5	34
50%	42	41	40
85%	47.5	48	48

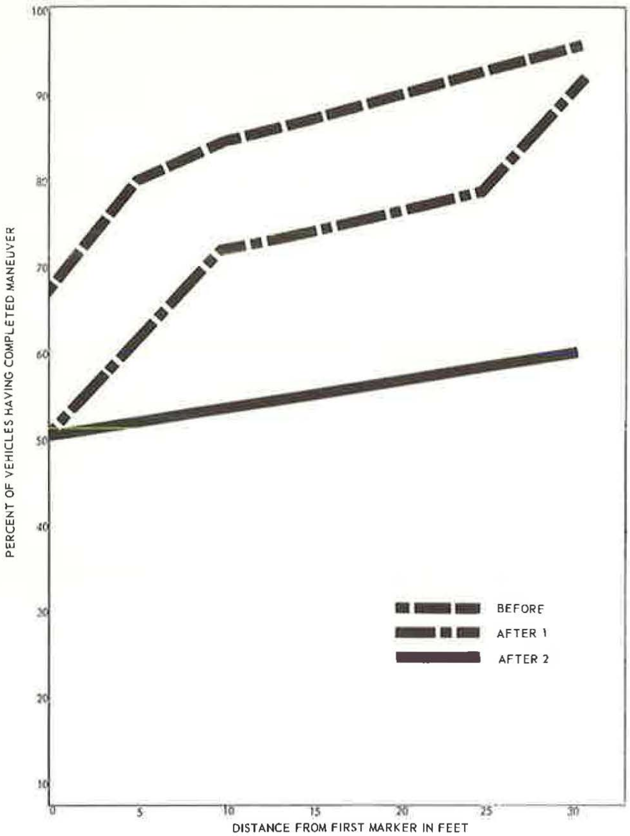

Figure 7. The percentage of left-turn vehicles clearing the through lane at selected locations—eastbound traffic at SR 16 and Waggoner Road.

pavement. The application of color does not cause the driver to decrease his speed when traversing the intersection. This is particularly significant in that the normally cautious driver, as determined by the 15 percentile point on the speed distribution, is not affected.

The conclusion drawn from this part of the study was that the application of color did not adversely affect the drivers continuing through the intersection on the main line.

Lateral Placement

Vehicle placement was the measure selected to determine the effectiveness of the colored pavement as a method of guiding left-turning vehicles into the proper channel. The vehicles were recorded at selected locations along the left-turn lane as either encroaching on the through lane or clear of the through lane. The effectiveness of the color was determined by the difference in the percentage of vehicles encroaching on the through lane at the selected locations.

The lateral placement measurements were taken to determine the distance from the beginning of the left-turn lane to the point at which the turning vehicle completely cleared the through lane. The lateral measurement was recorded as positive or negative, depending on whether the through lane was cleared, at selected distances from the beginning of the left-turn lane.

The outside edge of the left-turn lane was marked at 5-ft intervals, starting at the location marked on Figures 1 through 3, and moving picture films were taken of vehicles entering the lane. This picture was analyzed with the aid of an analyst projector to determine the lateral position of the vehicles at each of the marked locations. The percentage of the vehicles that had completed the maneuver (and thus freed the through lane) was recorded at each location. The percentages were used in the analysis of the effect of the colored pavement on traffic operations.

Figure 7 is a graph of the data for the eastbound approach to the intersection of SR 16 and Waggoner Road. The pattern of entry is substantially different in the "before" and "after" periods. Without the colored asphalt, a greater percentage of the vehicles enter the left-turn lane early. In fact, 66 percent of the left-turning vehicles completely cleared the through lane before reaching the first marker. This percentage dropped to 50 percent when the measurements were taken after the installation of the colored asphalt. In the "After 2" data, the percentage remained constant, but the slope of the line changed. In fact, only 10 percent of the vehicles entered between the beginning point and the 30-ft mark. The percentage of vehicles still encroaching on the through lane is consistently higher for the "after" situation at all measured points.

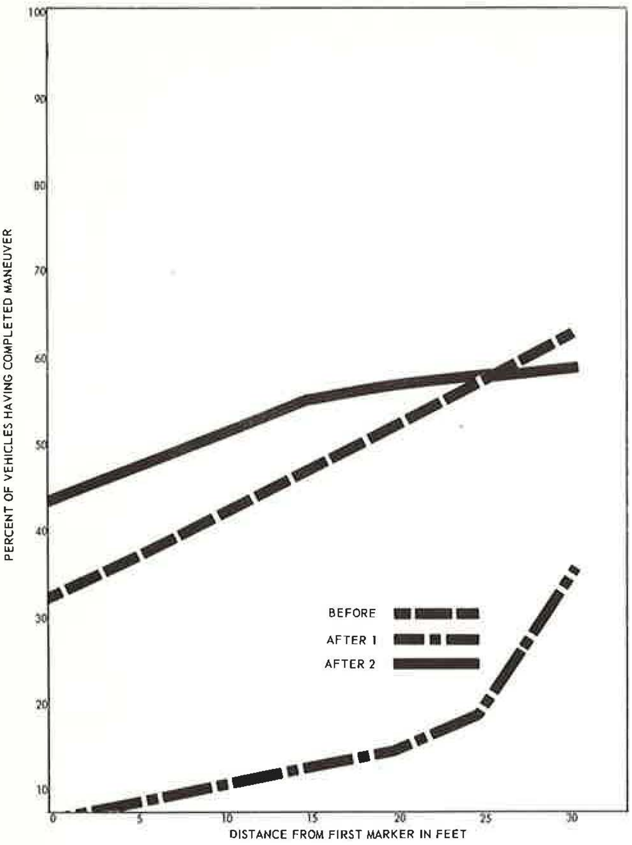

Figure 8. The percentage of left-turn vehicles clearing the through lane at selected locations—westbound traffic at SR 16 and Waggoner Road.

The westbound approach to this same intersection exhibited an entirely different pattern of entry into the left-turn lane. On this approach, 31 percent of the vehicles cleared the through lane before reaching the first marker in the "before" period, and only 6 percent in the "After 1" period. In fact, only 34 percent of the vehicles in the "After 1" period had cleared the through lane within 30 ft. In contrast, 63 percent of the vehicles had cleared the through lane in the "before" period. These data are shown in Figure 8.

In the "After 2" period, the pattern of entering vehicles reverted back to the original. For this time period, 44 percent of the vehicles had entered the left-turn lane prior to the first marker and 58 percent had completed the maneuver within 30 ft. It was apparent that the introduction of color had the immediate effect of discouraging drivers from entering the lane, but with time this hesitancy subsided.

The effect of the colored asphalt was more consistent on the two approaches to the SR 3 and Innis Road intersection. Most of the left-turning vehicles from both directions entered the turning lane within the first few feet. The percentage of vehicles which had completely cleared the through lane before the first marking was 80 percent for the northbound direction and 84 percent for the southbound direction. After the colored asphalt was installed, these figures dropped to 61 percent and 56 percent respectively. Figures 9 and 10 were prepared from these data.

The "After 2" data follow the "After 1" data remarkably well. There seems to be little immediate effect of the color which is not retained through at least a six-month period. If there is any difference at all, it is an amplification of the changes that occurred immediately after the colored asphalt was placed. This is in direct opposition to the behavior observed at the first location.

The influence of the signal and the lower speeds at this intersection probably contributed to the large number of vehicles entering the left-turn lane within the first sections. However, it was observed that many of these vehicles were violating the painted island areas and were driving across the island area in their approach to the intersection. This practice was reduced when the colored asphalt was placed. This change accounted for the difference in the "before" and "after" period at the intersection.

Results from the third study site, US 33 and Hayden Run Road, are shown in Figure 11. This is a "T" intersection, so there is only one left-turn lane at this site. The pattern of entry into the left-turn lane is similar to that found at SR 3 and Innis Road. Before the colored asphalt was added, 95 percent of the vehicles entered the left-turn lane before passing the first marker. Many of these vehicles were driving over the painted island to accomplish this.

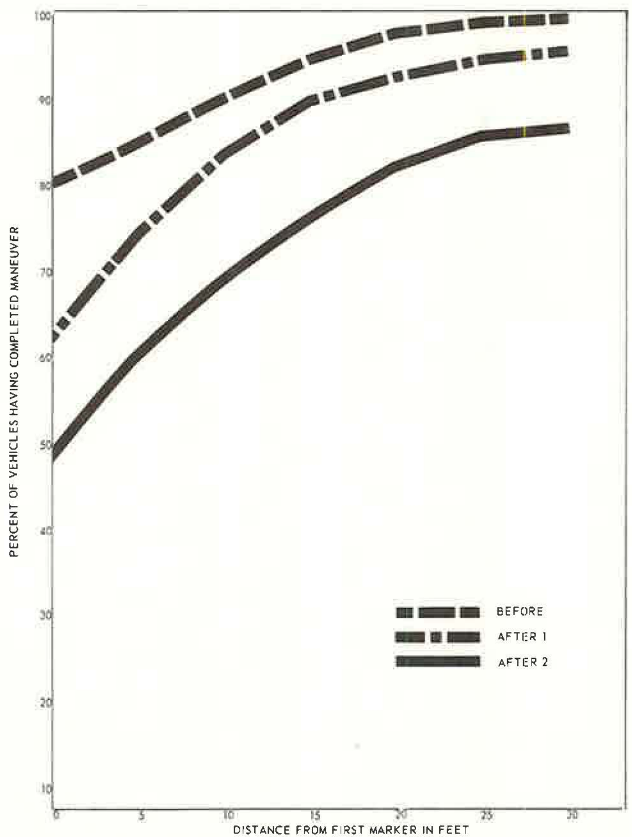

Figure 9. The percentage of left-turn vehicles clearing the through lane at selected locations—northbound traffic at SR 3 and Innis Road.

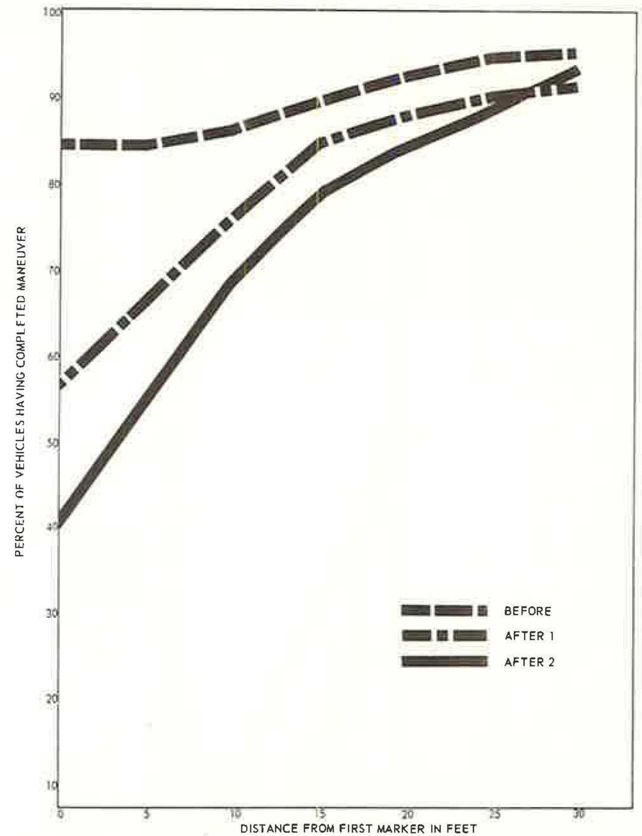

Figure 10. The percentage of left-turn vehicles clearing the through lane at selected locations—southbound traffic at SR 3 and Innis Road.

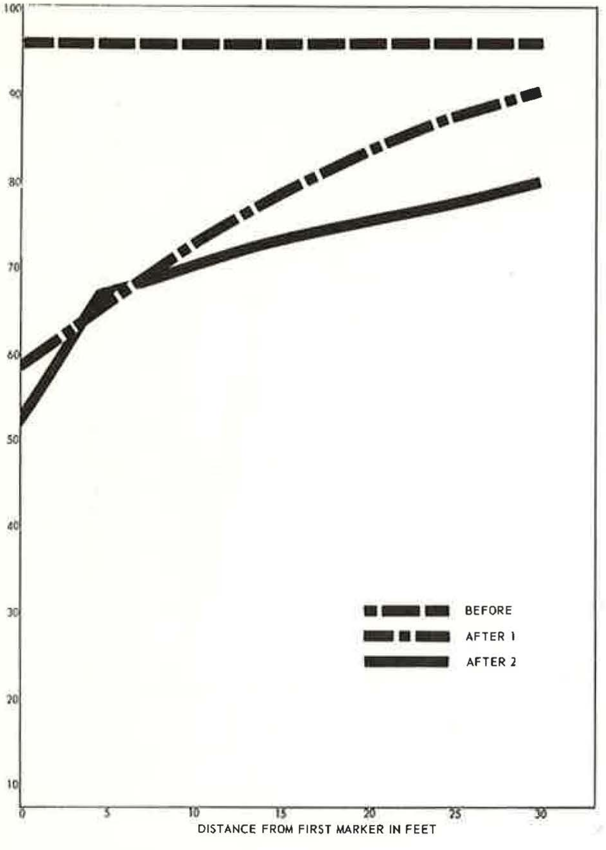

Figure 11. The percentage of left-turn vehicles clearing the through lane at selected locations—northbound traffic at SR 33 and Hayden Run Road.

The curve showing the percentage of vehicles that have cleared the through lane in the "after" period is similar to that found at the other locations. Over 58 percent of the vehicles enter the left-turn lane before the first marker, and 90 percent have entered within the first 30 ft. These percentages change to 52 percent and 90 percent in the "After 2" data. Once again, the tendency is to amplify the original difference.

Traffic Flow Patterns

The preceding two measures provided quantitative data on the effectiveness of adding color to the pavement. The third measure selected is difficult to describe in quantitative terms, and it is illustrated as a qualitative measure of the effectiveness of color.

Time exposures of vehicles making left turns at the subject intersections were taken at night from an elevated position. The camera was operated from the top of a platform lift truck at an elevation of about 18 ft. A total of 15 vehicles was recorded on each picture, and the uniformity of the flow pattern was compared for the "before" and "after" period. While this type of measure is not very definitive, it provides a useful comparison when considered in conjunction with the previous two measures.

The results of the third parameter selected for study are strictly qualitative. No attempt was made to quantify the flow patterns on the "before" and "after" photographs. Visual inspection of the sets of photographic pictures indicates that the flow patterns created by the taillights of vehicles entering the left-turn lane after the colored asphalt was in place is more uniform than that exhibited before the installation.

However, this difference does not appear as significant as that found in the lateral placement study. The reason for this probably lies in the non-reflective property of the colored asphalt. It is very difficult to discern the colored asphalt from the through lane at night. In fact, the painted island area has a considerably greater visibility at night than the colored asphalt.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions reached in this study can be summarized as follows:

1. The introduction of colored asphalt at an intersection does not significantly affect the velocity of vehicles in the through lane.

2. During the day, vehicles exhibit a more uniform pattern of lane changing with colored asphalt than with the painted island.

3. The colored asphalt has little effect on traffic flow patterns at night.

The lateral placement data collected at all five of the left-turn lanes are plotted on the same graph for comparative purposes. The wide differences among the patterns

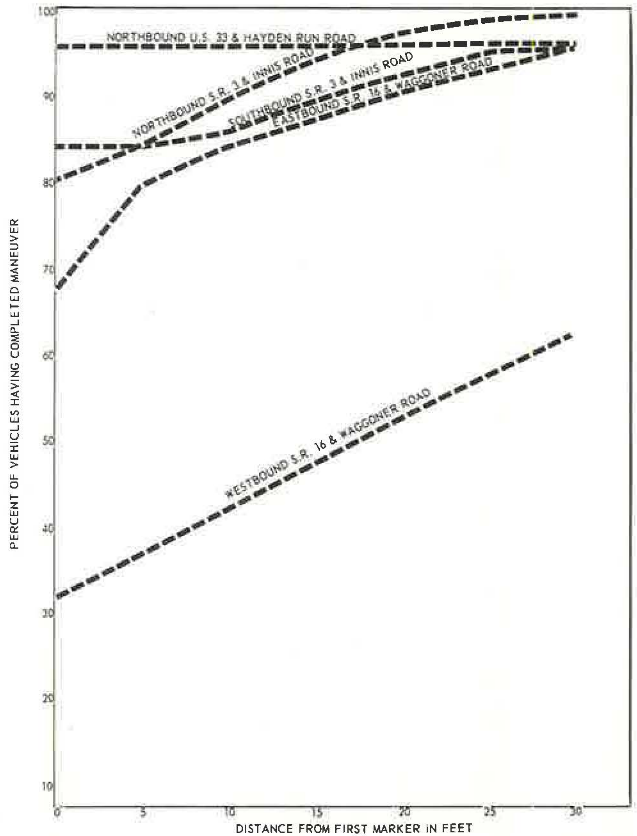

Figure 12. The percentage of left-turn vehicles clearing the through lane at selected locations—"before," all sites.

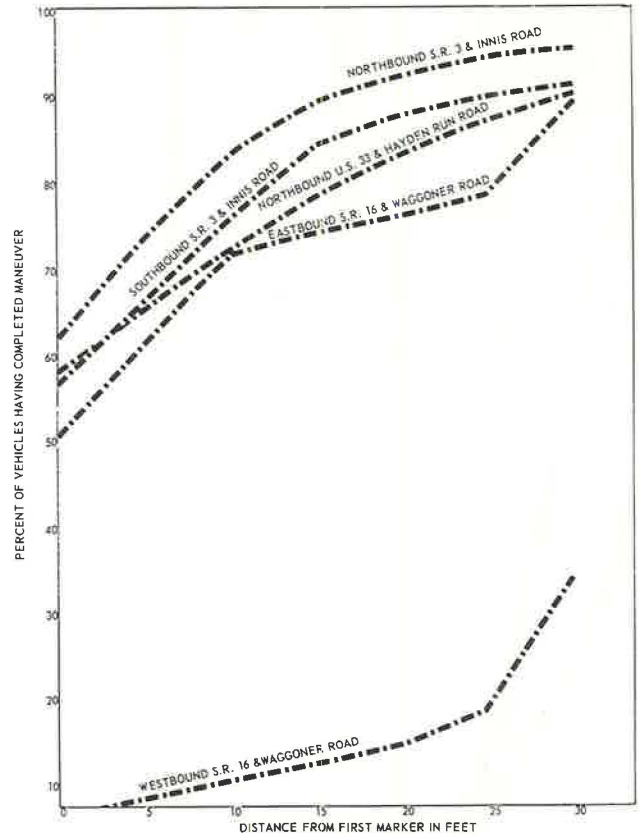

Figure 13. The percentage of left-turn vehicles clearing the through lane at selected locations—"after 1," all sites.

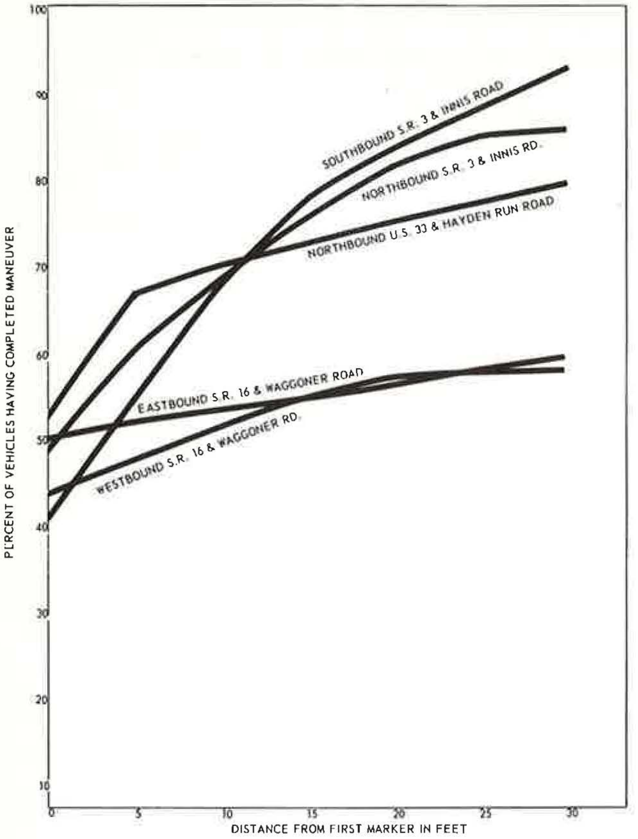

Figure 14. The percentage of left-turn vehicles clearing the through lane at selected locations—"After 2," all sites.

exhibited at the various locations before the installation of the colored asphalt are shown in Figure 12. With the exception of one approach, the patterns exhibited after the colored asphalt was placed are fairly consistent. These data are shown in Figures 13 and 14.

Based on these data, it appears that the application of the colored asphalt is effective in channelizing the left-turning vehicles entering the turning lane. This is accomplished primarily by directing the vehicles around the island area rather than by inducing the drivers to enter the left-turn lane earlier. In fact, the percent of vehicles completing their maneuver at selected distances is consistently higher for the "before" case than the "after" case.

One interpretation of this change in the pattern is that the yellow asphalt directs vehicles around the island area better than the painted cross-hatching, but that the use of a green left-turn lane is not significant in directing the turning vehicles into the proper lane.

Recommendations

Based on the results of the data collected in this study, the recommendation for use of colored pavement would be limited to one problem area. The problem, as defined by accident history or engineering judgment, must involve vehicles moving in opposite directions. The most pronounced effect of the colored pavement was to discourage vehicles from crossing the channelizing island when entering a left-turn lane.

The use of the green pavement in the turning lane apparently did not affect the drivers' patterns of entry into the lane. If this use had been effective, a steeper slope of the lines shown in Figures 13 and 14 would be expected. The slope of these lines, however, was nearly the same as the "before" case shown in Figure 12.

This study and the other studies reported on the use of colored pavements provide evidence that the use of color in pavements may well be an effective traffic control device. It appears that this effectiveness will be more pronounced when used to indicate areas of "no trespassing" rather than to indicate paths or corridors to be followed.

The limited scope of the project was not sufficient to provide data upon which generalized conclusions can be stated with certainty, but it does provide a starting point. Further research into possible uses of colored pavement is warranted by the results obtained in this study. In Ohio, we propose to conduct additional experiments on possible uses.

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Red Colored Pavement

DAVID W. GWYNN and JOSEPH SEIFERT, New Jersey Department of Transportation

A before-and-after study was performed on a ramp ending with a stop sign and at the ramp's intersection with a one-way roadway. Speeds and lags for both day and night conditions were measured before and after the ramp was paved red. Four pneumatic tubes were placed on the ramp at various distances from the stop line. Ramp traffic was registered on a 20-pen recorder. Speeds were computed using the difference in time and the distances between tubes. Average speeds were determined for points 100, 200, and 318 ft from the stop line.

Ramp traffic stopping and ramp and highway traffic crossing the intersection were manually recorded using a 20-pen recorder. Average accepted and rejected lags were computed. Before and after measurements were compared: the daytime speeds were significantly lower after the ramp was paved red, but for nighttime speeds and daytime and nighttime lags, there were no significant differences.

•THE motorists on today's highways are called upon to assimilate vast amounts of information about the ever-changing traffic stream around them. They must digest and react to a multitude of traffic-control devices within a relatively short period of time.

Over the past 25 to 30 years, various experimental sections of colored pavement have been examined in an effort to determine their potential as an aid to better driver performance and highway safety. However, very little controlled research has been reported, and that which has, did not lead to conclusive results.

Initial attempts at producing colored pavement resulted in some unsatisfactory results. A principal source of complaint was color fading, and early applications suffered from an insufficient amount of pigments.

Although colored pavement studies to date have not proven or disproven, for the matter, its effectiveness as an aid to better driving performance and safety, other studies dealing with the use of color as a means of conveying information indicate that color coding is a significant means of conveying information to the motorist (1, 2).

In order to evaluate on a limited basis the potential of colored pavement as an aid to driver performance and safety, the New Jersey Department of Transportation has installed a section of red colored pavement to be used in conjunction with stop signs. The study will evaluate (a) the performance of materials used to produce a colored surface, and (b) various driver characteristics related to the use of colored pavement. The first portion of this study, the materials evaluation, will not be discussed in this report as the test section of highway has not been in operation a sufficient period of time to adequately conduct such an evaluation. It is anticipated that the materials evaluation will evaluate such physical characteristics as (a) color uniformity, saturation, and retention; (b) visibility and reflectance; (c) skid resistance properties; and (d) reaction to chemicals, among other factors. The Prismo Safety Corporation applied red-coated spheres to this section of colored pavement, and the results of their tests of reflectivity and sphere retention will be covered in the materials evaluation.

Paper sponsored by Committee on Traffic Control Devices and presented at the 47th Annual Meeting.

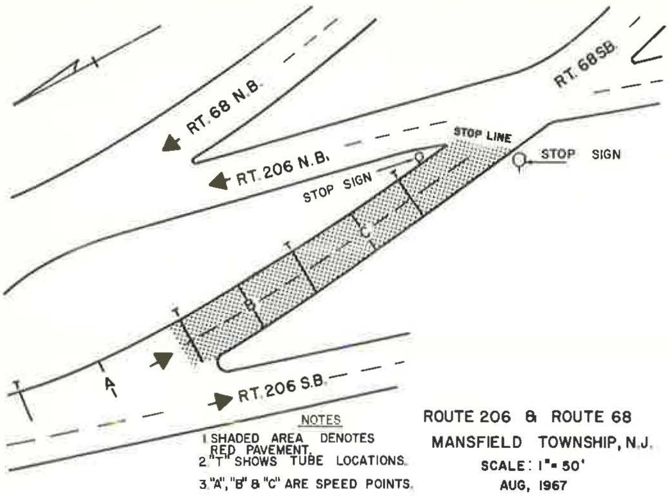

Figure 1. Red pavement site plan.

As previously stated, the color selected for evaluation was red and was used in conjunction with a stop condition. This is in accordance with a recommended use of the color red by the special committee on color of the National Joint Committee on Uniform Traffic Control Devices.

The color red in highway practices is accepted to mean "stop" or "do not" (2). There was some apprehension as to what the motorist's reaction would be to this red colored pavement. Would he think there was a stop condition ahead and react accordingly, or would he feel that it meant "do not enter" and cause him to react accordingly? This was an important question at this location because it was at a ramp-type location where the color red could have conveyed either meaning. However, since to our knowledge it has not stopped anyone from entering the ramp, it did not convey the meaning of "do not enter."

The purpose of this portion of the study is to evaluate the effects of red colored pavement on various aspects of driver behavior at a ramp location ending in a stop condition.

DESCRIPTION OF SITE

The section of highway selected for the study was the intersection of US 206 and NJ 68 in Mansfield Township, Burlington County, New Jersey (Figs. 1 and 2). A 280-ft section of the Route 206 southbound ramp (between US 206 southbound lanes and US 206 northbound lanes) was paved with ¾-in. red asphalt (see shading) on June 22, 1967. Prior to this, the ramp was concrete. This ramp leads to a stop condition at the northbound lanes of US 206 and connects into NJ 68 eastbound (on the east side of the northbound lanes of US 206). The terrain is flat and open, with no sight-distance restrictions. The speed limit on US 206 is 55 mph in both directions. There are two oversize (44 by 44 in.) internally illuminated (neon tubing) flashing stop signs at the end of the ramp (Fig. 3). The ramp is illuminated by two 4,000 lumen incandescent overhead lights spaced

Figure 2. Study site.

170 ft apart on the right-hand side of the ramp. Two 4,000 lumen incandescent overhead lights located on US 206 also contribute illumination to the ramp.

The AADT on US 206 south, at the entrance to the ramp, is 7,500 veh/day, with approximately 54 percent, or 4,050 veh/day using the ramp. The AADT on US 206 north is 3,500 veh/day at the ramp crossing with US 206 north. The ramp consists of two 16-ft lanes bordered by mountable concrete curbs. White solid edge-marking lines were in place next to the concrete curb and a white skip-line separated the two lanes. A white stop line was at the location of the stop signs at the east end of the ramp. The same markings were replaced after the red pavement was installed.

Figure 3. Stop condition.

Although accidents are not reported in this report due to insufficient time to evaluate them after the pavement was laid, they did play a role in the selection of the location as a study section. A before-and-after accident analysis will be conducted after sufficient time has elapsed to adequately conduct such a study.

From January 1, 1963, to December 31, 1965, 31 accidents occurred—three fatal, killing five persons, and eleven injury accidents, injuring 44 persons. Twenty-two of the accidents were right angle and seven were rear-end. All rear-end accidents occurred on the ramp.

STUDY PROCEDURE

One of the major reasons for adding color to the road surface is that the color should alert the motorist to an impending special condition. In this case, the red color was to alert the motorist to an impending stop condition. In an attempt to evaluate this, two characteristics were measured: (a) the speeds of vehicles (passenger cars or vehicles with four tires) approaching the stop signs before and after the installation of the red pavement at various points to be discussed later, and (b) the number of vehicles disregarding the stop signs before and after the installation of the red pavement. A third characteristic was also measured that was felt could be affected by the red pavement. This was the acceptance of lags by vehicles crossing US 206 north from the ramp. It was felt that the red pavement might alert the motorist to a hazardous condition and cause him to be more cautious to conditions at the intersection, thereby perhaps choosing to accept longer lags. A lag, for the purpose of this study, is the time interval between the arrival of a vehicle at the stop sign on the ramp and the arrival of the first vehicle on US 206 at the center of the intersection of the ramp and US 206 north. A lag will also be referred to as a time interval between the arrival of a vehicle at the center of the intersection of US 206 north and the arrival of the next vehicle on US 206 north at the same location when a vehicle is waiting on the ramp to cross US 206 north. To insure an adequate, consistent sample, only lags for cars that are first in line on the ramp will be tabulated. (Lags for queued vehicles will not be tabulated.)

Speed Measurements

Speeds were measured by use of pneumatic tubes stretched across the ramp at distances of 50, 150, 250, and 386 ft from the stop line and connected to a 20-pen recorder. The speed data were collected during both day and night conditions; only vehicles which stopped for the stop sign were recorded. Because speeds recorded are average speeds over a measured distance, they are plotted at the midpoints between the tubes at which the speeds were determined; i.e., the speeds are plotted at distances of 100,

TABLE 1

SPEED DATA COLLECTION TIMES

Date	Day or Night	Hours	Sample Size
		(a) Before	
5/4/67	Day	4:00 p.m. - 6:30 p.m.	286
5/4/67	Night	8:30 p.m. - 10:30 p.m.	297
		(b) After	
7/27/67	Day	3:45 p.m. - 7:00 p.m.	603
7/27/67	Night	8:45 p.m. - 10:15 p.m.	402

200, and 318 ft from the stop line (Fig. 1). Before and after speed data were taken as given in Table 1. The weather was clear and dry during all data collection periods.

Lag Measurements

The measurements of lag acceptance and rejection were made by utilizing a telegraph key connected to a 20-pen recorder. The recorded data were (a) a vehicle arriving at the center of the intersection of US 206 and the ramp on US 206, (b) a vehicle arriving at the stop line on the ramp, and (c) a ramp vehicle crossing the intersection. The sample data recorded on the recorder are shown in Figure 4. Referring to this Figure, line 1 shows a blip for the arrival of a vehicle on US 206, line 2 shows a blip for the arrival of a vehicle on the ramp, and line 3 shows the crossing of a vehicle on US 206. Distance R in Figure 4 is a time lag rejected, while distance A is the time lag accepted by the vehicle on the ramp to cross US 206 north. Lag data were taken before and after the installation of the red pavement (Table 2).

Vehicles Disregarding Stop Sign

The number of vehicles which passed through the stop sign, making no attempt to slow or stop, was recorded during the same time periods as the collection of lag data.

ANALYSIS AND RESULTS

Speeds Before and After

The speed data were analyzed statistically by utilizing the standard test for testing significant differences in means. The formula utilized for this test is

$$t = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\frac{S_1^2}{N_1} + \frac{S_2^2}{N_2}}}$$

where

$\overline{X}_1$ and $\overline{X}_2$ = the mean speeds for before and after;
N_1 and N_2 = the sample sizes for before and after; and
S_1 and S_2 = the standard deviations of the speeds before and after.

The level of significance used in the test was the 90 percent level of significance. If the t-value obtained in the test

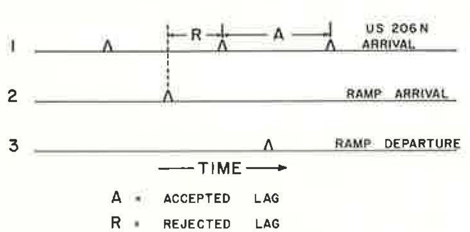

Figure 4. Recorder tape.

TABLE 2

LAG DATA COLLECTION TIMES

Date	Day or Night	Hours	Sample Size*
		(a) Before	
11/4/66	Day	1:30 p.m. - 3:00 p.m.	86
11/9/66	Night	6:30 p.m. - 8:00 p.m.	44
		(b) After	
7/31/67	Day	1:40 p.m. - 3:10 p.m.	124
8/1/67	Night	8:45 p.m. - 10:15 p.m.	47

*For cars which are first in line only.

is greater than the t-value for the 90 percent level (1.65) of significance, then there is assumed to be a significant difference in the results at this level of significance.

As stated previously, the speeds were obtained at distances of 100, 200, and 318 ft from the stop line. Table 3 shows the speeds before and after at each point and for day and night conditions. This is also shown in Figures 5 and 6.

It can be seen (Fig. 5) that at all three points where the speeds were recorded for the day conditions, the after average speeds were less than the corresponding before speeds. Although the average speeds after were less by only approximately 1 mph, they were significantly different at the 90 percent level of confidence at points A and B. At point C there was no significant difference in the average speeds before and after.

TABLE 3

BEFORE AND AFTER SPEED DATA

Site	Distance From Stop Sign (ft)	Mean Speed (mph)		Significant Difference
		Before	After	
(a) Day				
A	318	38.7	37.6	Yes
B	200	32.1	31.5	Yes
C	100	24.1	23.9	No
(b) Night				
A	318	37.4	37.1	No
B	200	31.5	31.2	No
C	100	23.9	23.6	No

At the same three points (Fig. 6) during night conditions, the average speed during the after portion was also less than the before average speed. However, at none of the three points were the average speeds significantly different at the 90 percent level for before and after.

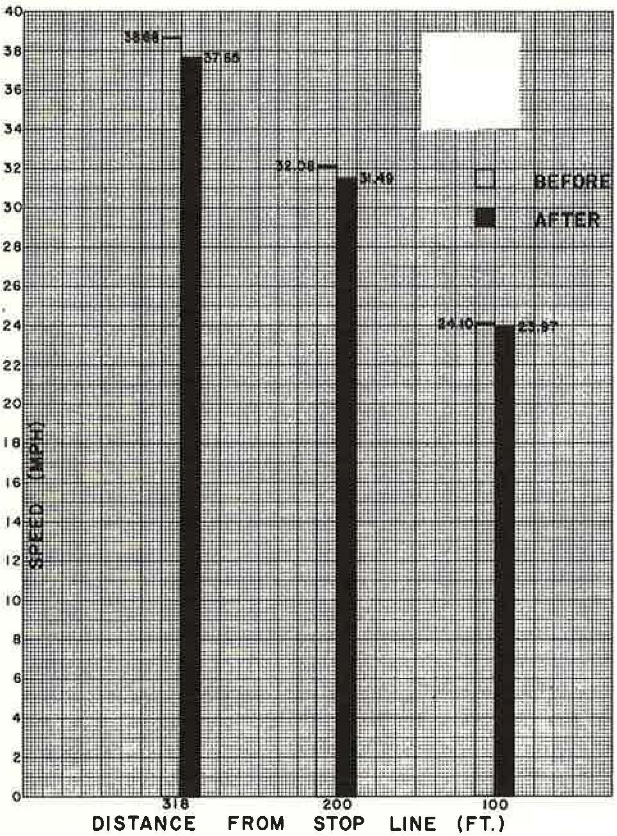

Figure 5. Speeds, day.

A second test, the "F" test was used to check for a significant difference in the variance of the before and after speeds. The formula utilized for this test is

$$F = \frac{S_1^2}{S_2^2}$$

where

S_1^2 = variance of before speed data, and

S_2^2 = variance of after speed data.

The 90 percent level of significance was used in this test also.

At points A and C there is a significant difference in the variability of the before and after speeds for day conditions. At point A, the variance before is greater than the after variance, whereas at point C, the reverse is true. At point B there is no difference at the 90 percent level of significance. The colored pavement may have the effect of causing speeds to be grouped closer around the mean at point A, whereas it appears that the reverse occurred at point C. It would appear more desirable to have the speeds grouped closer around the mean, as at point A, than to have a larger variation, as at point C

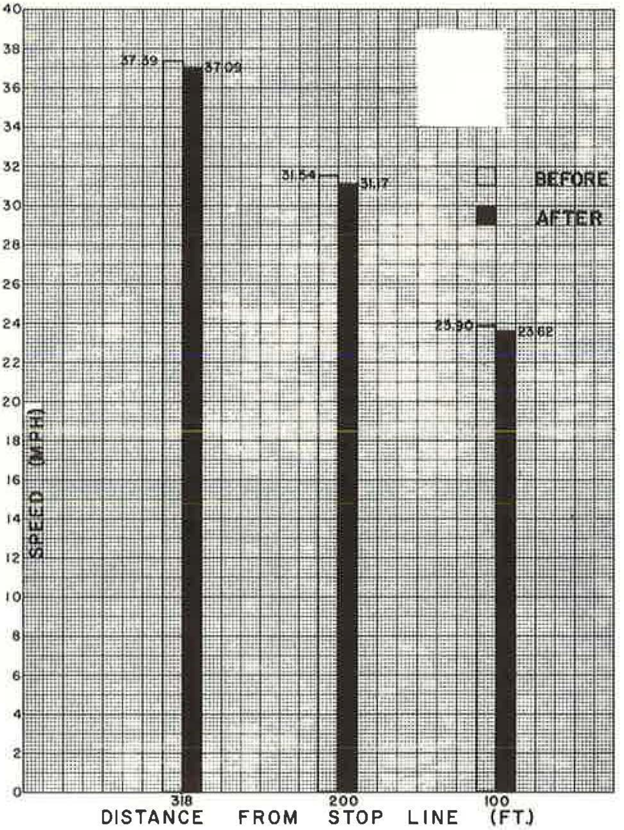

Figure 6. Speeds, night.

For night conditions, there was a significant difference at the 90 percent level at point A, as there was for day conditions. However, there was no significant difference at points B and C. The after variance was again less than the before variance.

The fact that the mean speeds are not significantly different for before and after conditions during hours of darkness appears to support the opinions of our engineers who have driven the section during both day and night conditions. The red pavement is certainly visible during day conditions, but appears to black out at night. The red color does not appear to be discernible, thereby logically indicating that the red pavement should have no effect on changing speeds in the area.

Although the average speeds were only slightly less during the after period for day conditions, it is possible that the red pavement alerted the motorist to the impending stop-hazardous condition, which thereby contributed to the reduction in the average approach speeds. For night conditions, since it appears at least visually that the red pavement was not distinguishable, the average speeds should not have been affected by the red pavement and, statistically, they were not.

Lag Acceptance and Rejection

Vehicles accepting or rejecting lags of from 0.5 to 15.0 seconds were analyzed to determine the mean lag accepted and the mean lag rejected for both before and after conditions. The 15-second maximum lag was used because no vehicle rejected a lag over 15 seconds.

During the before and after period for day conditions, the average lag accepted was 10.52 and 10.72 seconds respectively. A test for significant differences in means at the 90 percent level of confidence indicated that there was no significant difference between the average lag accepted before and after the installation of the red pavement. The average lags rejected for the same conditions were 3.70 seconds before and 3.97 seconds after. Here, also, there was no significant difference in the two average values before and after (Table 4). It does not appear that the red colored pavement has any effect on causing the driver to be more cautious and perhaps accept a longer lag.

For night conditions before and after, the average lags accepted were 11.28 and 11.05 seconds respectively (Table 5). Here again, the test for significant differences in means indicates that there was no significant difference between the average lag accepted at night before or after the installation of the red pavement. The average lag rejected at night before was 4.49 seconds, and after it was 4.51 seconds. There was no significant difference at night in the average lag rejected before and after the instal-

TABLE 4

LAG DISTRIBUTIONS, DAY

Midpoint Value (sec)	Accepted		Rejected	
	Before	After	Before	After
0.25	0	0	8	11
0.75	0	0	13	21
1.25	0	0	9	28
1.75	0	0	13	16
2.25	0	0	18	15
2.75	0	0	17	15
3.25	0	0	9	29
3.75	0	0	14	12
4.25	1	0	7	12
4.75	0	0	6	6
5.25	0	0	4	18
5.75	1	4	8	11
6.25	1	2	4	10
6.75	4	4	6	11
7.25	4	7	11	5
7.75	9	3	3	2
8.25	5	7	3	4
8.75	5	11	2	2
9.25	6	3	0	5
9.75	2	9	1	2
10.25	2	9	0	2
10.75	7	8	1	3
11.25	3	9	0	3
11.75	7	8	0	3
12.25	6	5	0	0
12.75	4	2	0	1
13.25	6	8	0	1
13.75	2	9	0	0
14.25	5	6	1	0
14.75	6	10	0	0
Total	86	124	158	248
Over 15.00	157	190	0	2
Mean	10.52	10.72	3.70	3.97
Std. Dev.	2.65	2.57	2.53	2.90

Note: Means are computed using values of 15.00 seconds or less.

TABLE 5

LAG DISTRIBUTIONS, NIGHT

Midpoint Value (sec)	Accepted		Rejected	
	Before	After	Before	After
0.25	0	0	4	10
0.75	0	0	8	12
1.25	0	0	9	8
1.75	0	0	6	20
2.25	0	0	6	8
2.75	0	0	4	9
3.25	0	0	4	7
3.75	0	1	7	14
4.25	0	0	8	12
4.75	0	1	5	6
5.25	0	0	5	5
5.75	0	0	3	13
6.25	0	0	3	7
6.75	0	1	3	7
7.25	0	2	5	7
7.75	1	0	4	4
8.25	0	3	2	2
8.75	3	3	4	7
9.25	3	1	4	2
9.75	5	4	0	1
10.25	6	3	1	1
10.75	3	3	2	4
11.25	2	3	0	2
11.75	4	0	0	2
12.25	6	6	2	0
12.75	3	3	0	1
13.25	4	3	0	3
13.75	1	4	0	0
14.25	0	3	0	0
14.75	3	3	0	0
Total	44	47	99	174
Over 15.00	164	216	1	0
Mean	11.28	11.05	4.49	4.51
Std. Dev.	1.77	2.65	3.05	3.19

Note: Means are computed using values of 15.00 seconds or less.

lation of the red pavement. Here again, it does not appear that the red colored pavement had any effect on causing the driver to be more cautious and perhaps accept a longer lag.

Disregard of Stop Sign

Under day conditions before the pavement was installed, eight out of 282 vehicles observed passed through the stop sign without making any attempt to slow or stop. After the installation, three vehicles out of 351 observed passed without stopping. For night conditions, two out of 252 observed passed before as compared with one out of the 311 observed after. A test for significant difference in proportions, also at the 90 percent level, indicates that there is no significant difference in the passing before and after for day or night conditions.

CONCLUSIONS

No definite conclusions can be drawn from this study concerning the overall effectiveness of the use of red colored pavement, particularly since it deals with only one location. However, it is felt that this study gives some insight into the effect of the red colored pavement on approach speeds and lag acceptance and rejection. At this site, it appears that the red pavement may have a significant effect on causing the average approach speed at two of the three study points to be significantly less for day conditions. It also may have an effect on the variation of speeds at two of the study points, although the variation was less at one point after and greater at the other. It could be argued

that the difference in average speeds was too small to be appreciable logically; however, it was appreciable statistically at the 90 percent level of significance. For night conditions, there was no change in the average before and after speeds statistically; since the red color was not discernible at night, this appears logical.

The red pavement apparently had no effect on the average lag accepted or rejected after the installation of the pavement as compared with before the installation for both day and night conditions.

Generally, it appears that the red colored pavement caused little, if any, change in the parameters studied. However, two major categories are yet to be studied, namely, the materials evaluation and the accident analysis.

Also, it must be kept in mind that the use of colored pavement to convey a definite meaning to the motoring public is new and little, if any, attempt has been made to educate the public as to its use or meaning. Although this location was given wide newspaper and TV coverage, it is doubtful that the majority of motorists passing through the location accept the red color to mean that a "stop" or "hazardous" condition is impending. Perhaps the better measure will be the accident evaluation.

Although the results of this study show little, if any, change in the parameters studied after the installation of the red pavement, it is not meant to infer that colored pavement cannot be an effective traffic control device. It is felt that colored pavement has the same potential to convey messages through its use of color as traffic signs. However, some means of educating the motoring public to the meanings of the various colors when used in pavements must be devised. Then the results of studies to evaluate its effectiveness will perhaps be more meaningful and give a better insight into its actual potential as a traffic control device.

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Evaluation of Pavement Marking To Designate Direction of Travel and Degree of Safety

JAMES STEPHEN HUBBELL and WILLIAM C. TAYLOR, Ohio Department of Highways

This study was designed to investigate the effectiveness of pavement marking as related to three items—driver perception, driver understanding, and driver performance. Three marking systems were studied using white, yellow, broken and solid lines, singly and in combination.

Since there appeared to be no single measure that fully described the effectiveness of pavement marking, five phases of study were developed. Each phase was directed toward a different measure of effectiveness. The analysis of the results of these phases provided sufficient data from which conclusions were drawn regarding the effectiveness of various marking types. Significant results were found in many of the studies which indicated that pavement marking systems could be devised to convey meaningful information to the driver. However, this would require some period of education and adjustment on the driver's part. Also, the use of color appears to have greater potential in the long run than the use of line shape.

•A TRUE evaluation of the effectiveness of a traffic control device is dependent on a determination of three factors—driver perception, driver understanding, and driver performance. A traffic control device may be ineffective as a result of a deficiency in any of these three factors. Thus, any attempt to establish or alter standards for traffic control devices must consider each item. In many cases, the functions cannot be separated and studied exclusively, but must be studied in combination.

Pavement marking, in conjunction with the appropriate signing, is one such traffic control device. This project was designed to study the effectiveness of pavement marking as related to each of the three factors. The project consisted of five separate studies, each designed to evaluate these factors either singly or in combination.

Under the driver perception factor, tests were conducted to determine the driver's ability to discriminate between line pattern and color. These elements were used to form a two-factor analysis. The color difference was limited to white and yellow, and the line was either continuous or broken.

The driver understanding item was divided into three separate elements: (a) the definition of a corridor within which the driver should maintain his position; (b) a supplement to signing in informing the driver that the sight distance is inadequate to complete a passing maneuver; and (c) the designation of conditions existing beyond the defined corridor. Specifically, the marking should indicate whether the space immediately adjacent to this corridor is reserved for vehicles traveling in the same direction, in the opposite direction, or is not designated as a corridor for moving vehicles.

Driver performance was separated into three basic elements. Each element is a combination of driver visibility and driver understanding, as well as driver acceptance. These elements are roughly defined as ability to maintain lateral position, observance of passing restrictions, and lane utilization.

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The study consisted of five phases, each directed toward a different measure of effectiveness, yet each a measure of one of the three factors listed previously. This approach was selected because there did not appear to be any single measure that fully described the effectiveness of a pavement marking system. The analysis of the results of these five phases provided information on which decisions can be made regarding the effectiveness of various pavement marking systems.

Phase I consisted of a slide presentation, depicting various pavement markings and pavement marking systems, which was presented to various groups of drivers. Each subject was questioned regarding his interpretation of the meaning conveyed by the markings. The two elements of driver understanding involved information transfer regarding the direction of travel and passing safety. Results were used to measure driver understanding on these two questions. This test was also used to evaluate the ability of the observer to adapt to selected pavement marking systems. This was accomplished by questioning the subject with the slides arranged in random order, and then repeating the test with the slides systematized.

Phase II was conducted as a direct measure of driver performances under various markings. The unit of measure was the ability of the driver to maintain lateral position while negotiating a series of curves. This phase was designed to provide further insight into driver perception. The purpose of the pavement marking system was corridor definition, because driver understanding was not a variable in this phase.

Phase III, directed toward driver understanding and acceptability, was conducted to determine whether pavement marking is successful in deterring the driver from crossing the centerline to initiate a passing maneuver. The unit of measure was the number of drivers passing a slow-moving vehicle introduced into the traffic stream. Comparing these numbers provided information concerning the relative value of the markings as a deterrent to the passing maneuver. These data provided additional information on the driver's understanding as well as his performance.

Phase IV also dealt with driver performance as a function of driver understanding. Various pavement markings were placed on a section of highway including a transition from 2-lane to 4-lane highway, and lane utilization observances were made. The objective was the determination of differences in lane usage occasioned by pavement marking. This test provided information on which judgments were made regarding the deterrent effect of pavement marking in a situation where lane changing is appropriate. This differed from the preceding phase where the deterrent effect was measured in a section where lane changing was not allowed.

The final phase of the project was designed primarily to measure driver perception. A questionnaire was designed to test driver perception of pavement marking on a section of highway over which he had just driven. The questions concerned: (a) what the driver had seen; (b) his interpretation of the marking; and (c) his opinion of the marking. In this way, results could be used to evaluate understanding and acceptance as well as perception. However, these two measurements could only be obtained from those drivers who had correctly identified the markings. The pavement marking question depicted a transition from a 4- to 2-lane highway. This particular type of pavement marking accents differences between the various proposed marking systems. The results of all five phases of the study were used to form conclusions regarding the factors which defined effectiveness. Recommendations regarding pavement marking are based on a composite of all five phases.

PROCEDURE AND RESULTS

Phase I

Procedure—A series of slides illustrating two vehicles on a simulated highway was prepared. The vehicles were placed in a car-following position in the right-hand lane in each case, and the pavement markings were varied. These markings included all combinations of white, yellow, dashed and solid lines, singly and in combination.

The first eight slides depicted pavement markings as described by the present Manual on Uniform Traffic Control Devices. The only two lines defined in this system are the dashed white and the solid yellow.

The next 16 slides depicted pavement markings that are described by an alternate pavement marking system proposed by the subcommittee on pavement marking of the National Joint Committee on Uniform Traffic Control Devices. This marking system is designed on the principle of using color to define direction of travel and line shape to define degree of safety. A yellow line is used to separate traffic flowing in opposite directions, whereas a white line separates two traffic streams flowing in the same direction. A solid line is used to designate areas where no passing or lane changing is permitted, with a broken line used at all other locations.

The remaining 12 slides depicted pavement markings as they would appear in a second alternate system proposed by the committee. Line shape is used to designate direction of travel, and color to designate degree of safety. A solid line separates traffic streams moving in opposite directions; a dashed line separates traffic streams moving in the same direction. A yellow line designates locations where it is unsafe to pass or change lanes.

In both alternate systems, a meaning is defined for four possible lines: solid or broken white, solid or broken yellow. All these markings were illustrated on the 36 slides that comprised the problem set for Phase I. The slides were presented first in random order and with very little explanation. The subjects were asked to identify with the car following in each slide and to answer the following two questions:

1. Is the lane immediately to your left for vehicles moving in the same direction? (yes or no)
2. Is passing permitted at this location? (yes or no)

The purpose was to obtain data from which conclusions regarding driver understanding and interpretation of line pattern and color could be drawn.

The subjects were then told the purpose of the interview and the rationale for each of the three marking systems. Principal differences among the three systems were emphasized and all questions from the subjects were answered.

Finally, slides were grouped with respect to system (8 slides for the present marking system, 16 for alternative 1, and 12 for alternative 2), and each group was presented sequentially after a short review of the rationale. For each system presentation the subject was required to answer the preceding pair of questions, but in the context of the system being presented.

Analysis and Results—The analysis was conducted to answer the following questions:

1. How does the driver presently interpret each pavement marking configuration?
2. Can the driver easily understand a marking system?
3. Which elements of the marking system are most easily understood?
4. Are these answers influenced by the subject's geographical location (state), sex, or driving experience?

The answer to the first question was derived from the random presentation. There is only one of the four possible combinations of line and color that is uniquely defined in the present manual. This element is a solid yellow line. The dashed white line is used in the present system, but it carries two meanings. The results of the response are given in Table 1.

It is apparent that the line type closest to conveying a universal message is the solid yellow. Ninety percent of the subjects interpret this line as a separation between opposite direction flows, and 88 percent feel that it indicates no passing. The broken white line is also interpreted the same by nearly all interviewees on the question of passing. Ninety-one percent of the responses indicated that this line could be crossed to execute a passing maneuver. However, this line does not convey a unique message regarding the direction of travel because approximately two-thirds of the subjects interpreted the line to mean same direction and one-third to mean opposite direction flow.

Color alone does not convey a consistent message. Only 77 percent interpreted the broken yellow as separating opposite direction flow, and 63 percent indicated they felt it was safe to cross this line type when passing. The solid white line was more often considered a barrier to passing (66 percent) and to indicate opposite direction of flow (79 percent). This might suggest that the shape of the line is the more critical of the two parameters.

TABLE 1

LINE INTERPRETATION FROM RANDOM PRESENTATION

Response	Type of Line			
	Solid Yellow	Broken Yellow	Solid White	Broken White
Indicates opposite direction (%)	90	77	79	34
Indicates unsafe to pass (%)	88	37	66	9

The analysis of the ability of drivers to understand and respond correctly to different marking systems was based on the results of the systematized presentation. Each of the three systems was analyzed and the percentage of correct responses was recorded. The present system rated highest, with 96 percent correctly identifying the direction of travel and 97 percent the degree of safety. This is due to the fact that there are only two line types involved, and one of these has two correct responses. Thus, the only errors involved would be either in response to the solid yellow line or in the passing response to the dashed white.

Alternate 2 was the most easily understood of the two alternatives, perhaps because this system is more closely related to the present system. Both systems use all four line types to describe the direction of travel and degree of safety; thus, they are somewhat more complex than the present system. However, 92 percent of the subjects correctly identified the direction of travel and the same number identified the degree of safety following the discussion.

Alternate 1 showed a greater number of errors. Both the question concerning the direction of travel and that concerning passing had a lower percentage of correct answers (89 and 89 %). This system is the most different from the present marking system.

Table 2 gives the number of incorrect responses given in the random order presentation that were corrected when the slides were systematized and the system rationale explained. These figures give some indication of the ease with which each of the systems could be learned, and the ability of the driver to adapt to new systems.

Over the past 25 years there has been no change in the present marking system, and it should be readily identifiable to the subjects answering the questionnaire. In other words, the percent correct for the present system given in random order presentation should be an indication of the ease with which this system could be learned. However, since alternate systems 1 and 2 are new to the driver, an explanation of these systems is necessary to arrive at a comparable measure of learning and adaptation.

Using the present system, 87 percent correctly identified the direction of travel and 89 percent the degree of safety. The total corrected percentage using alternates 1 and 2 combined was 90 percent for correctly identifying the direction of travel and the same for the degree of safety. This indicates that the driver could learn and adapt to a new system as easily as to the present system.

An analysis was next made of the responses to line type to isolate the elements of the various systems that proved to be most difficult to the driver. It may be assumed that a "difficult" marking element is less readily learned after the system rationale has

TABLE 2

LINE INTERPRETATION FROM SYSTEM PRESENTATION

System	Sample Size	Random (no. correct)	Percent Correct	Random (no. incorrect)	System (no. corrected in presentation)	Percent Corrected	Total Corrected (%)
(a) Identification of Direction of Travel							
Present	3376	2957	87	419	298	72	96
Alt. No. 1	9035	6019	67	3016	1988	66	89
Alt. No. 2	6822	5562	81	1260	701	56	92
(b) Identification of Passing Safety							
Present	3376	2991	89	385	270	70	97
Alt. No. 1	9035	6327	70	2708	1727	64	89
Alt. No. 2	6822	5755	84	1067	506	47	92

TABLE 3

LINE INTERPRETATION BY LINE TYPE

Line Type	Sample Size	Random (no. correct)	Percent Correct	Random (no. correct)	System (no. corrected in presentation)	Percent Corrected	Total Corrected (%)
(a) Identification of Direction of Travel							
Broken white	6244	4638	74	1606	1117	70	92
Solid white	4538	2644	58	1894	1227	61	85
Broken yellow	2232	1718	77	514	312	65	91
Solid yellow	6219	5538	89	681	411	63	96
(a) Identification of Passing Safety							
Broken white	6244	5662	91	582	384	74	97
Solid white	4538	2487	55	2051	1123	56	80
Broken yellow	2232	1491	67	741	468	65	88
Solid yellow	6219	5433	87	786	528	69	96

been explained than an "easy" one. If the responses from the random presentation are scored right or wrong in the context of the system represented by each slide, the subject's improvement in the systematized presentation can be computed as the portion of incorrect responses that were corrected after the rationale for the system was explained. This improvement is indicated in Table 3.

The results were consistent with the analysis of the random presentation. The solid yellow line was most often correctly defined on both questions (96 and 96 %). The type of line next most often correctly identified was the broken white, particularly when the slides were arranged by system and an explanation given for each system. The question of a dual meaning was removed in this context, and the responses reflect this simplification.

The meaning of color is apparently easier to convey than that of line shape. The percent of responses correctly identifying the meaning of the dashed yellow line was greater than that for the solid white. Of the responses to the broken yellow line, 91 percent correctly identified the direction; the respective percent for the solid white was 85 percent. The question of safety was correctly identified 88 and 80 percent, respectively.

It is apparent from Table 3 that the type of line that comes closest to conveying a universal message is the solid yellow. As in Table 1, 90 percent of the subjects interpret this line as a separation between opposite direction flows, and 88 percent feel that it indicates no passing. The broken white line was also interpreted the same by nearly all interviewees on the question of passing. Ninety-one percent of the responses indicated that this line could be crossed to execute a passing maneuver. However, this line does not convey information regarding the direction of travel because approximately two-thirds of the subjects interpreted the line to mean same direction and one-third to mean opposite direction flow.

Color alone does not convey a consistent message. Only 77 percent of the subjects interpreted the broken yellow as separating opposite direction flow, and 63 percent

TABLE 4

PERCENT OF CORRECT RESPONSES

Marking System	Years of Driving Experience										
	Drivers Ed.	<1	1-3	3-5	5-10	10-15	15-20	20-25	25-30	30-35	>35
Present (without explanation)	88	80	93	89	86	90	88	90	86	88	85
Present (with explanation)	96	92	99	99	98	98	96	98	95	94	96
Alternate 1 (with explanation)	89	80	94	86	92	93	89	90	87	86	86
Alternate 2 (with explanation)	92	84	94	91	92	94	92	94	90	90	90

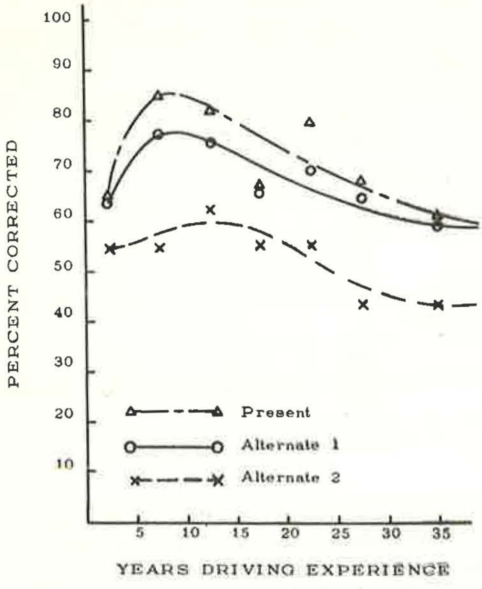


Figure 1. Effect of driving experience on learning power.

indicated they felt it was safe to cross this line type when passing. The solid white line was more often considered a barrier to passing (66 %) and to indicate opposite direction of flow (79 %). This might suggest that the shape of the line is the more critical of the two parameters.

The final question analyzed was the influence of location, sex, and driver experience on the results. Questionnaire results were received from 7 states, and the results showed that there were no significant differences in the responses. None of these states differ markedly with respect to its use of pavement markings.

The subjects were classified by sex, and an analysis was made to determine if this had any effect on the responses to the questions. The male group had a slightly higher percent of total corrected for each system than the female group. However, there was found to be no significant difference between the groups.

Next an analysis was made to determine the influence driver experience had on the results (Table 4). The 1 to 3-year group ranked highest in percent of correct responses for all systems. The next highest ranking for all systems was the 25 to 30-year group, and beyond this the percentages dropped. For all groups, the present system with an explanation given ranked highest, with alternate 2 second and alternate 1 last. However, it should be reemphasized that alternate 2 is the more closely related to the present system; the only variation being the solid white line.

Therefore, an analysis was made of those subjects who incorrectly answered a question in random order presentation, and correctly answered the same question after an explanation of each system. It is evident (Fig. 1) that alternate 1 could be learned easier than alternate 2 for all driving experience groups.

Phase II

Procedure—The second phase was an analysis of the lateral position of vehicles on curves. The effect of various markings on the positioning and the variance in this positioning provide information on the adequacy of these lines in the definition of a corridor. The specific aims of the lateral placement research phase were (a) to relate lateral placement to variations in pavement markings (solid or broken lines and edge lines) and illumination (day vs night); and (b) to determine if the changes caused by the various markings are different for various locations on a curve.

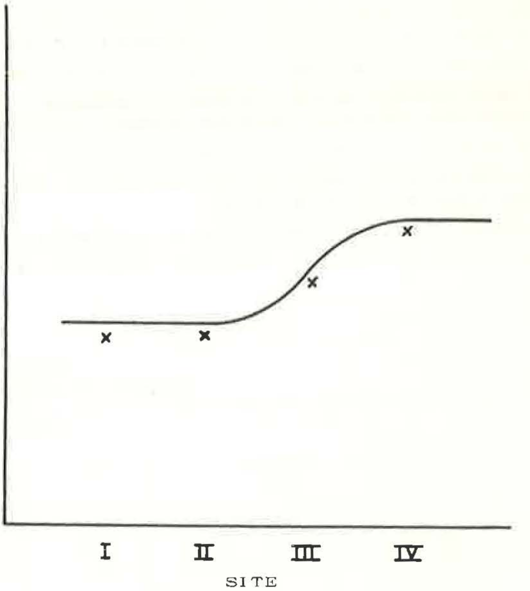


Figure 2. Identification of lateral placement data points relative to the curve.

The research was done at two Ohio locations—on State Route 16 about 16 miles east of Newark, and on State Route 62 about 10 miles south of Washington Court House.

The first location was a section of newly paved roadway that was unmarked at the start of the program. Four data collection sites were chosen at an S-curve within a total road distance of one-third mile. Site 1 was along the straight portion of the highway about 150 yd before the start of a left curve. Site 2 was about 30 yd beyond the start of the curve. Site 3 was in the middle of the curve (approximately 150 yd beyond site 2). Site 4 was at the end of the left curve where the reverse curve began (it was more than 200 yd beyond site 3). The second location studied was set up similarly, but it was a more gentle curve.

The lateral placement of vehicles was measured as the distance between the right rear tire and the edge of the highway. This was determined by photographs. The location of the data collection points in relation to the curve is shown in Figure 2.

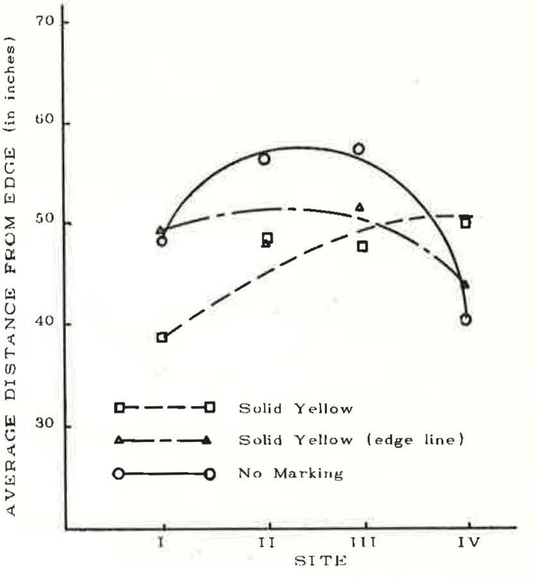


Figure 3. Distance from outside rear tire to edge of road.

Since the Ohio Department of Highways did not have the photographic capabilities required for data collection, a contract for collection and reduction of data was awarded to H. R. B. Singer, Inc., of State College, Pa.

A motor-driven camera, triggered by an infrared detector unit, was used to photograph the vehicles at the specific point where measurements were to be made. The camera was mounted on a sign stake driven into the ground at the same distance from the road edge as a normal route sign. One stake was driven at each data collection site and left in place until the entire data collection program was completed. The camera and film magazine were slender enough to be almost concealed by the stake. Since both the camera and magazine were painted flat black, they could not be seen until the vehicle was within one or two car lengths of the camera, and at night, normally they were not visible.

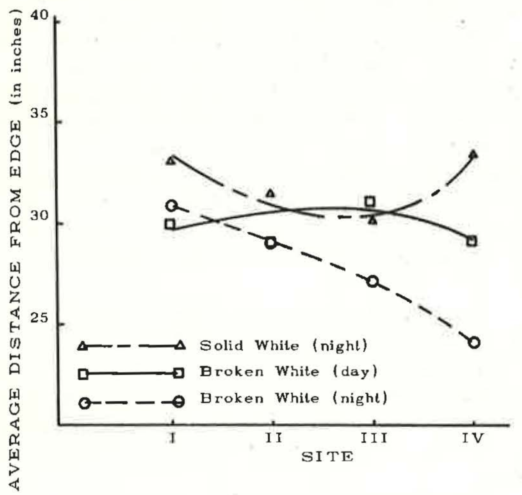


Figure 4. Distance from outside rear tire to edge of road.

The infrared triggering unit was concealed in the weeds and grass by the road edge. When a vehicle drove through the detector's field of view, the difference in infrared radiation between the vehicle and the background was detected. The circuitry was designed so that the camera trigger pulse was generated when the detected signal returned to background level. This occurred as the rear of the vehicle passed the detector so that the rear tire was photographed.

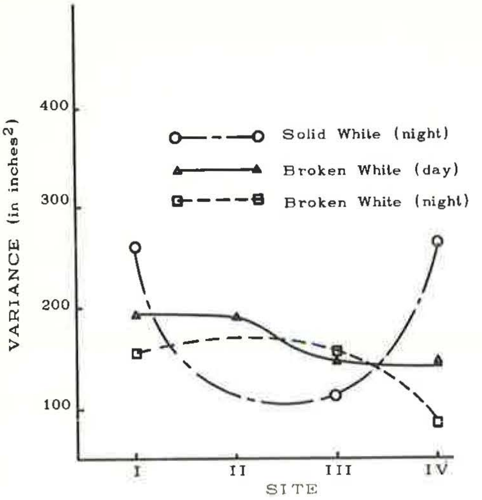


Figure 5. Variance of mean distance from the edge of the road to outside rear tire.

Photographs were obtained at night by using a standard electronic flashgun to illuminate the scene. However, to keep the data recording unobtrusive, it was necessary to mask out all visible light from the flash. This was accomplished by placing a Wratten type 87 filter over the flashgun and using a high-speed infrared film. With this technique, the light flash could be seen by looking directly into the flash gun; however, the light was not visible to the driver of the vehicle because the unit was below and behind him when it flashed.

At each site, marker stripes were laid on the road for one or two photographs to provide reference marks for later data reduction. These strips were then removed so they would not interfere with the vehicle placement while data were being taken.

To obtain distance measurements, the developed films were projected on the screen of a film reader. The marker stripe positions were scribed on the screen, and the pictures were projected in register with the scribe marks. The marks were used to obtain the spacing between the right rear tire and the road edge. The uncertainty in these measurements was <3 in. even when the exact point of contact between tire and road was masked by shadows.

At the first location five different markings were tested: (a) small, reflectorized buttons in the center of the roadway; (b) double solid yellow line in the center of the roadway; (c) double solid yellow line in the center, with solid white edge line; (d) broken yellow line in the center of the roadway; and (e) broken yellow line in the center, with solid white edge line.

The second location tested two other markings—solid and broken white centerlines, with solid white edge lines in both cases.

Analysis and Results—The mean distance from the edge of the road and the variance around the mean were recorded for each sample. Data were taken on each marking system at the four locations. The results of this study are given in Figures 3, 4, 5, and 6.

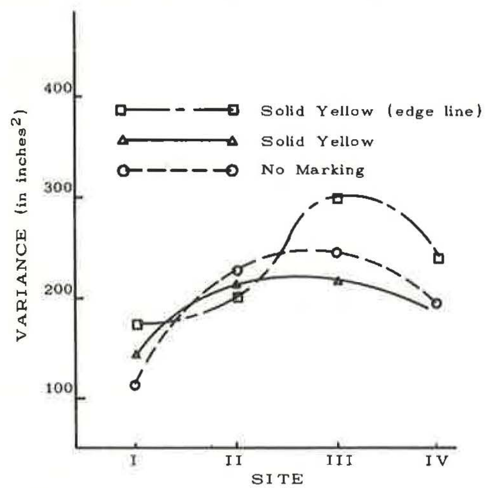


Figure 6. Variance of mean distance from the edge of the road to outside rear tire.

An analysis of variance was conducted on the mean value of the distance from tire to road edge with respect to data collection site. The results indicate that vehicles negotiating the sharper curve (first location) exhibited a significant change in position, whereas those at the gentle curve (second location) did not. For this reason, the analyses of differences by site were limited to the data collected at the first location.

The relative ability to define a corridor, as measured by the mean and variance, is most pronounced between the highway with no marking and the one with solid yellow centerlines plus edge lines. These are the extremes among the schemes of delineation considered here.

Edge delineation was the only element that proved significant in all cases. The addition of an edge line invariably produced a change

TABLE 5

RESULTS OF SIGNIFICANCE TESTS ON LATERAL DISTANCE AND VARIANCE

Item	Element Being Tested	Mean Value	Variance	Difference in Mean	Difference in Variance
Day	Solid yellow vs	45.4	227	S.[a]	N. S.[b]
	no marking	50.6	198		
	Solid yellow vs	45.4	227	N. S.	S.[c]
	broken yellow	44.3	171		
	Solid white vs	28.1	213	N. S.	N. S.
	broken white	29.8	175		
Night	Solid yellow vs	47.8	257	N. S.	S.[c]
	broken yellow	46.0	117		
	Solid white vs	32.8	225	S.	S.[c]
	broken white	27.4	131		
Edge line	Day:				
	Solid yellow vs (with)	48.0	187	S.	N. S.
	Solid yellow (without)	45.4	227		
	Night:				
	Solid yellow vs (with)	54.1	240	S.	S.[c]
	Solid yellow (without)	47.8	257		
	Broken yellow (with) vs	36.5	234	S.	S.[c]
	Broken yellow (without)	44.3	171		
Day	(D) Solid white	27.9	213	S.	N. S.
Night	(N)	32.8	225		
	(D) Broken white	30.0	175	N. S.	N. S.
	(N)	27.4	131		
	(D) Solid yellow	44.4	227	S.	S.[c]
	(N)	47.8	257		
	(D) Broken yellow	42.4	171	S.	S.[c]
	(N)	46.0	117		

[a]Significant.
[b]Not significant.
[c]Significant at one or more locations.

in the mean value of the lateral position. Changes in the color or configuration of the centerline had a less pronounced effect on the lateral placement.

In two of the five comparisons of solid vs dashed lines, the mean distance between the vehicle and the edge of the highway was, by an analysis of variance test, significantly different. In each case, the distance was greater with the solid line. The other three cases exhibited no significant difference. A ratio test of variances of 18 sites showed that the solid line produced a wider dispersion of means in five cases and a narrower dispersion in one. The remaining 12 sites showed no significant difference.

An analysis of variance of means for day data showed no significant difference between markings (solid vs broken white lines) with respect to the distance from the road edge. A ratio comparison of the variances also showed no significant differences. Thus, there is no measurable difference between their ability to define a corridor in daylight. The night data showed a significant difference. The broken white line produced a corridor closer to the road edge. Table 5 gives the results of the significance tests for all the data.

Phase III

Procedure—The third phase of this project was conducted on State Route 98 in Marion County, Ohio. Its purpose was to determine the merit of various markings as a deterrent to passing. Variations in the effectiveness were obtained for two geometric conditions and for day and night.

Data records indicated the percentage of drivers who crossed the various markings to pass another vehicle. To assure the comparability of data, the same experimental setup was used for all line types. The experiment involved the introduction of a constant velocity vehicle into the traffic flow and the recording of the number of drivers that passed the test vehicle.

The test-car velocity was 40 mph, although the speed limit is 60 mph in daytime and 50 mph at night. The driver of the test vehicle was stationed along the road approximately one-fourth mile from the test section. As a vehicle approached, the test car

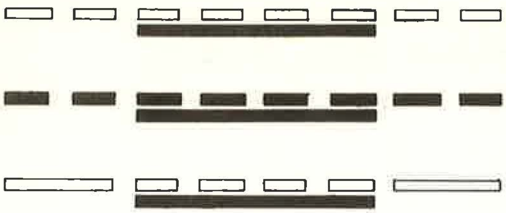


Figure 7. Various pavement markings tested.

entered the highway and paced himself so that the observed vehicle was immediately behind him as he entered the test section. The test vehicle then maintained a velocity of 40 mph and the driver noted the action of the following vehicle. The experimental section was 1 mi long, and the observations were noted as "pass" or "no pass." We did not attempt to record the location within this 1-mi section where a passing maneuver took place.

No data were recorded for the situation where oncoming vehicles prevented a passing maneuver. In all cases, the data indicated the driver's willingness to cross the line to pass when there is no apparent reason to follow the slow-moving vehicle.

Analysis and Results—Two 1-mi long sections of highway were selected. The first was a very straight, level section of road with no sight restrictions of any kind. The second site was also a straight section of road, but it was in rolling terrain. There were no severe sight restrictions, but this would not be obvious to the driver. The undulations were severe enough to obscure sections of the road ahead. Thus, the depth of the dips between the rises could not be determined by a driver unfamiliar with the road.

A complete set of data was obtained for these two sections immediately following a repaving contract before the standard marking had been replaced. The purpose was to establish base data against which valid comparisons could be made. The results showed that 100 percent of the subject drivers passed in the flat section and 95 percent passed in the hilly section. All of the seven no-passing observations occurred at night.

A solid yellow line was then placed in the test sections and the centerline marking was changed according to the various systems. First, a broken white line was used approaching the test section and continuing adjacent to the solid yellow. Next, a broken yellow line replaced the broken white line. Finally, a solid white line was used for the centerline approach, and a broken white line used adjacent to the solid yellow (Fig. 7).

The data for each of these three markings were similar; χ-square tests indicated that the shape or color of the approach line did not change the percentage of the observed passes significantly. This could be expected because the predominant line in all cases was the solid yellow line adjacent to a broken line.

Significant differences were noted when the percentage of the observed daytime passes was compared with nighttime passes. Table 6 summarizes the data for the three centerline types used with the solid yellow line. It is apparent that there is no difference between the flat and hilly sections either during the day or at night.

The solid yellow line was replaced by a solid white line to mark the flat test section. A dashed white line was placed on the approach to the solid white, but no markings were used adjacent to the solid white. This marking was found to be much less a deterrent than the solid yellow. In fact, the percentage of drivers crossing this line to pass increased from 46 to 93 percent during the day and from 26 to 87 percent at night. It is obvious that a solid line does not deter many drivers from executing a passing maneuver.

The white line was then removed and a series of reflective markers placed on the test highway. White reflectors were used as the centerline, and yellow reflectors were used through the two test sites. This afforded an opportunity to test the effect of a broken yellow line (simulated by the yellow reflectors) on the number of drivers who pass. As with the solid white line, this proved to have little deterrent effect on the drivers. During the day, 87 percent of the vehicles crossed the broken yellow line and 46 percent crossed the solid yellow line. At night, the percentages were 70 and 26 percent, respectively.

TABLE 6

VEHICLES CROSSING YELLOW LINE TO PASS

Site	Percent of Vehicles Passing		
	Day	Night	Total
Flat	46	28	36
Hilly	47	27	37
Total	46	27	36

The yellow line, even when broken, was a slightly greater deterrent than the solid white line. This difference was more pronounced at night (70 to 87 percent) than during the day (87 to 93 percent). This may be partially due to the increased visibility of the markers at night.

It was recognized that the deterrent effect the solid yellow, broken yellow, or solid white line had on the driver could be biased towards those familiar with the test section. Therefore, the license plate of the following vehicle was recorded enabling local vs nonlocal classification of vehicles. χ-square tests indicated driver knowledge of the section did not significantly affect the percent of vehicles passing on each of these marking systems (Table 7).

TABLE 7

CHI-SQUARE TEST—ALL MARKING SYSTEMS

Classification	Pass		No-Pass		Total
Local	215	(220)	196	(187)	407
Nonlocal	265	(256)	208	(217)	473
Total	476		404		880

χ^2 = 1.49.
Not significant at 0.05 level.

Phase IV

Procedure—This phase was conducted to determine the driver's interpretation of pavement marking. In Phase III we tested driver perception and interpretation in a relatively critical situation. That is, the driver was subjected to a decision involving his safety; his reaction to the various centerlines was recorded. In Phase IV we were interested in the driver's interpretation under conditions involving no particular stress. The test site selected included a 2-lane to 4-lane transition. In this phase, the data were collected on drivers going to the 4-lane from the 2-lane section. The test site was on Hamilton Road in Columbus.

The measure of the effect of the various pavement markings was the relative amount of use of the two southbound lanes of the 4-lane roadway. Here the 4-lane pavement reached its full width and also began the lane-dividing lines in the southbound lanes. Beginning at this point and continuing at 100-ft intervals, the number of vehicles in each lane was recorded until the relative number of vehicles in the two lanes reached a constant value.

Analysis and Results—Three types of pavement markings were studied: broken white, solid white, and broken yellow. For all marking types, the number of vehicles in each lane reached a relatively constant value between 300 and 700 ft from the initial data collection point. Thus, the number of vehicles was recorded at 0, 100, 200, 300, and 700 ft. The sample size for each data point consisted of 400 vehicles. Figure 8 sketches the study site.

Beyond the last data point was an approach ramp to West Fifth Avenue. The number of vehicles using this ramp remained constant for all marking types. This indicated the desire to cross into or remain in the outer lane was the same for this turning maneuver. This proved to be an aid in measuring the deterrent effect of the different markings as a desire to cross into the outer lane was exhibited. Also, in the 2-lane section, the single southbound lane fed directly into the inner lane of the 4-lane section. Any influence the line type would have in restricting the driver's desire to cross into the outer lane could be measured. Table 8 gives the percent vehicles in both lanes for each marking type.

It is apparent that for the existing broken white lane line use remained relatively constant for each lane. χ-square tests indicated there were no significant differences between any two data points for the inside lane.

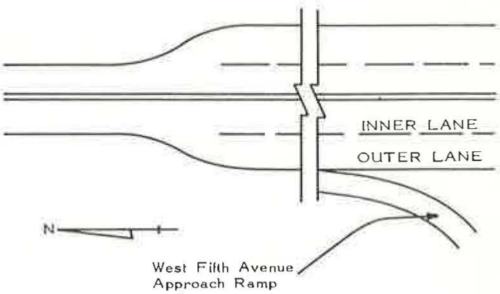


Figure 8. Phase IV test section.

TABLE 8

LANE USAGE OF VEHICLES

Lane	Percent Vehicles at Each Data Point				
	0 Ft	100 Ft	200 Ft	300 Ft	700 Ft
Inside:					
Present	42	39	42	44	41
Solid white	53	50	42	38	38
Broken yellow	62	50	44	42	42
Outside:					
Present	58	61	58	56	59
Solid white	47	50	58	62	62
Broken yellow	38	50	56	58	58

The marking was changed to a solid white line and a second data set was collected. This alteration significantly increased driver reaction for the first two data points. For this data set, 53 percent of the vehicles remained in the inside lane, whereas 42 percent remained in this lane for the broken white line at the initial data point. Fifty percent stayed in this lane for the solid white line as compared to 39 percent for the broken white line at the second data point. Beyond the 200-ft point the percent of vehicles in the inner lane reached a constant value for both markings. This would indicate that the solid white line influenced the driver's immediate desire. In other words, there was an initial hesitancy on the part of the driver to cross the solid white line.

The marking was next changed to a broken yellow line and a third data set collected. The broken yellow line significantly increased driver performance for the first two data points. Sixty-two percent of the drivers did not cross this line, whereas 42 percent did not cross the broken white line at the first data point. The data at the 100-ft point and thereafter were similar to the solid white line.

This indicates that the broken yellow line had an initial deterrent effect on driver desires. This line type had an even more deterrent effect than the other two types for the initial data point. For this point, 62 percent of the vehicles stayed in the inside lane, whereas 53 percent for the solid white and 42 percent for the broken white remained in this lane. However, after the initial data point, the solid white and broken yellow resulted in a similar percent of lane use.

The broken white showed a constant lane use throughout the study area. The broken yellow and solid white resulted in cars merging into the outer lane for the first 100 to 200 ft. After this point, lane use became constant and the number of vehicles in each of the lanes did not vary significantly from the broken white.

Phase V

Procedure—The previous two phases of the project tested driver perception and interpretation under under stress and driver interpretation under a "no-stress" condition. Phase V was conducted to determine the driver perception and interpretation under this "no-stress" condition.

Again, the test site selected included a transition from a 4-lane to a 2-lane highway. The site was located at Olentangy River Road in Columbus. The pavement marking was changed on the 2-lane section of highway, and driver interviews were conducted. The interview took place approximately one-half mile beyond the point of transition.

The interview consisted of an explanation of the purpose of the interview, followed by a series of questions. The wording of the interview was as follows:

> Good morning, sir. We are from the Ohio Department of Highways and are conducting a survey on pavement marking. Accordingly, we would like to ask you a few questions.
>
> 1. Would you please describe the pavement marking on the 4-lane section?
>
>   For example: Was it yellow or white? Solid or broken?
>
> 2. Would you please describe the pavement marking on the 2-lane section?
>
>   For example: Was it yellow or white? Solid or broken?

If the subject driver accurately described the pavement marking, the interview proceeded to the third question:

3. What is the major difference in the meaning of these two markings you described?
4. Do you feel that this type of marking is helpful and should be standard treatment?

Thank you for your cooperation.

Analysis and Results—The first set of interviews was conducted with the existing pavement marking, consisting of a broken white lane line on the 4-lane section and a broken white centerline on the 2-lane section. The transition area was marked with a double yellow line. However, if the subject described this transition section, the interviewer indicated that we were interested in the 2-lane highway beyond the point of transition. In this particular case, questions 3 and 4 were not applicable since the correct answer to questions 1 and 2 would be identical.

The results of the first interview indicated that drivers were not consciously aware of the pavement marking. In fact, only 16 percent of the 94 drivers interviewed could properly identify the pavement marking on both the 2-lane and the 4-lane sections. Twenty-two percent of the drivers offered no description of the pavement marking in either section.

The number of drivers who properly described the pavement marking for the 4-lane section was higher than for the 2-lane section. Exactly 50 percent of the drivers correctly identified the broken white line on the 4-lane section, but only 19 percent could identify the broken white on the 2-lane section. Seventeen percent of them erroneously described both the 4-lane and the 2-lane sections.

The pavement marking was then changed to a solid white line on the 2-lane section. The broken white line remained on the 4-lane section. A second set of interview data was then obtained. In this case, questions 3 and 4 were applicalbe and were summarized.

The alteration of the pavement marking did not increase driver awareness by a significant amount. For this set of interviews, 21 percent of the drivers correctly identified the dashed white lane line and the solid white centerline. Twelve percent of the drivers erroneously identified both sections.

The changed section, the solid white centerline, increased driver perception somewhat. The number of drivers correctly identifying the centerline increased from 19 to 28 percent. The percentage of drivers correctly identifying the lane line dropped from 50 to 44 percent. The number of drivers offering no description increased from 22 to 28 percent.

The marking tape used to produce the solid white line was brighter than the existing painted lines. The existing lines had not been repainted since the previous summer. We felt that this might influence the results by calling special attention to these lines. Consequently, alternate sections of the solid white line were removed leaving a broken line which matched the painted line. Another set of data was obtained which duplicated the first set.

The results of the third set of interviews indicated that the brightness of the line does have an effect on driver perception. The percentage of drivers correctly identifying both sections was 22 percent. The percentage correctly identifying the 4-lane section dropped from 50 to 42 percent, whereas the percentage identifying the 2-lane section increased from 19 to 30 percent.

The remainder of the interviews were conducted on locations with new tape for the markings. All further comparisons are based on the results of this second set of interviews on the present system.

TABLE 9

DRIVER AWARENESS—BY LINE TYPE

Marking Type on 2-Lane	Percent Corrected			No Opinion (%)
	2-Lane	4-Lane	Both Sections	2-Lane
Broken white	30	42	22	31
Solid white	28	44	21	33
Broken yellow	34	41	15	16

The pavement marking was next changed to a broken yellow centerline on the 2-lane section. This alteration did not significantly affect driver awareness. Of the 91 drivers interviewed, only 15 percent correctly identified the dashed white lane line and broken yellow centerline. Twenty-one percent could not correctly identify the 2-lane and 4-lane sections.

The broken yellow centerline was correctly described by 34 percent of the drivers. The dashed white lane line was correctly identified by 41 percent. Fourteen percent of drivers interviewed could not recall the pavement marking in either section and offered no description.

Table 9 summarizes these percentages for all markings. It appears that the broken yellow has a greater influence on driver perception than the other markings. χ-square tests indicated there was no significant difference. Also, it should be emphasized that the broken yellow is completely new to the driver. Thus, the driver would be more aware of this alteration than the other marking types. This is evident from the last column.

SUMMARY AND CONCLUSIONS

The study was conducted to determine some facts about the effectiveness of various pavement markings in terms of driver perception, understanding, and performance. The results are discussed separately for all but the perception factor, which could not be isolated. However, perception may be considered basic to both understanding and performance.

Driver Perception and Understanding

Measures of driver perception and understanding were tested in 4 of the 5 phases. In two cases, slide presentation and passing study, the solid yellow line proved to be the best understood line type. In the lane use and driver interview phases, the solid yellow line was not used.

The slide presentation results showed that 96 percent of the subjects correctly identified the meaning of the line both in relation to direction of travel and degree of safety. The broken white line proved to be nearly as effective, with 92 percent correctly identifying the direction of travel and 98 percent the degree of safety. The solid white and broken yellow lines were less often identified correctly.

When categorized by sex and driver experience, the results are the same. The 5 to 10-year driver experience group has the best perception and understanding for each type of line. The same is true in correctly identifying the direction of travel and degree of safety. For all line types, males had a higher but not significant total corrected than females in identification of direction of travel and degree of safety.

In the passing phase, the solid yellow line also proved to be more uniformly understood. Only 46 percent of the drivers crossed this line during the day and 26 percent at night. When this marking was replaced with a solid white line, the percentage of drivers passing increased to 93 and 87 percent, respectively. The broken yellow line produced nearly the same effect as a solid white line with 87 and 70 percent, respectively.

The driver interview phase indicated no significant difference in driver perception of the different line types. However, 34 percent correctly identified the broken yellow line, 30 percent the broken white, and 28 percent the solid white.

A solid yellow line was not used in the lane use phase. Drivers appeared to be more aware of the broken yellow line, and there was a higher percentage of vehicles that were deterred by this line type. The solid white was less of a deterrent while the broken white line had little or no deterrent effect.

It is apparent that driver interpretation is conditioned to the present marking system. The use of a solid yellow line as a barrier has come to be generally used and accepted. It is also apparent that neither color nor solid line (alone) conveys this message. The solid white line did not prove to be an acceptable substitute; neither did the broken yellow. Any proposed revision of the pavement marking that uses line type and line color to convey separate meanings would involve a driver education period.

Driver Performance

Three phases of the project involve measures of driver performance associated with different types of lines. The lateral placement studies were intended to illustrate differences in driver dependence on the line to convey the corridor description. The passing study was conducted to determine the driver's willingness to cross different lines in a critical situation, and the lane use study was for this determination in a "no-stress" condition.

The lateral placement study indicated that drivers interpret various line types differently when negotiating a curve. When lateral positions of vehicles are compared for the conditions with no lines and with the solid yellow, the driver tends to move away from the center of the road. When an edge line is added, the driver tends to move back toward the center. It can be hypothesized that the driver is shifting his reference guide from the edge of the road when there is no marking, to the center of the road when a solid yellow line is supplied, and back again to the edge of the road when an edge line is added. The use of a solid white line appears to influence the driver in much the same manner as a broken white line. It does not cause the driver to avoid the centerline as much as the solid yellow line.

The results of the passing study illustrate driver performance in regard to crossing various type lines. Again, the solid yellow line is the only line to which drivers react. The results of the lane use study give an indication of the deterrent effect of line types. The broken yellow line had more influence on driver performance in this respect.

In summary, it appears that the results of these tests were influenced by driver conditioning. However, significant differences were found in many of the tests that indicate that systems could be devised to convey meaningful information to the driver following some period of education and adjustment. The use of color appears to have a greater potential than the use of line shape. No observations can be made of the long-range effects because of the limited duration of the study.

Discussion

R. J. ISRAEL, California Division of Highways—I congratulate the authors on their ingenious approach methods to a problem that is very difficult to research. Particularly, I would like to cite the slide presentation study, duplicates of which were forwarded to engineers throughout the nation for comparable tests in their specific areas; therefore, this phase represents subjects from many geographical areas, substantially increasing the validity of this part of the study.

The authors state that the establishment of a new system of longitudinal markings would require a period of education and adjustment on the driver's part. I agree that there would be some such period because there are some segments of the driving population that react slowly to new signs or markings. However, the establishment of a new system would be heavily publicized and I believe the changeover would not be a major problem as evidenced by the far greater changeover, from driving on the left to driving on the right, recently accomplished in Sweden. Hopeful evidence in this regard is given by Phase I. With only a few minutes' explanation and a more orderly arrangement of the slides, from 56 percent to 72 percent of the subjects were able to correct previous errors concerning the direction of travel.

The problem of confusion with a single marking to indicate both two-way and one-way travel has been one of long standing. The problem is being intensified by the building of Interstate and other freeways and multi-lane divided highways. Serious accidents due to this confusion are on the increase, particularly in areas where a high percentage of travel is on freeways.

It is fortunate that this research indicates color as being the more effective. The dashed yellow centerline has the greater possibility as a line for two-directional roads. It presents the greatest contrast to the lane line on one-directional roads and should not

be confused with other current uses. The solid white line is being used in other concepts such as a channelizing line through tunnels and interchanges to discourage unnecessary lane changes. The dashed yellow line on the great mileage of two-lane roads can also be attained at considerably less cost. The dashed line requiring only 40 percent of the paint will be far more acceptable than a solid line by most states and particularly by counties and other local jurisdictions.

This research gives firm backing to the recent actions of the National Joint Committee, which has voted to establish the yellow dashed line on two-directional roads as a basic change in their forthcoming revised manual. I personally believe that the elimination of the present duplication will provide a substantial contribution to traffic safety on a national basis.

JAMES L. FOLEY, JR., Commissioner, Department of Transit and Traffic, City of Baltimore—The research conducted by Taylor and Hubbell was greatly needed. For several years the markings subcommittee of NJCUTCD has been struggling to eliminate the conflicts in the existing manual. These conflicts are of two kinds: (a) differing meanings for the same marking, and (b) differing markings for the same meaning.

Examples of (a) are: dashed white line (urban) means lane line, and dashed white line (rural) means centerline or lane line. Examples of (b) are: dashed white line (rural) means centerline, and solid white line (urban) means centerline. The problem becomes more acute when we add solid white edge lines, solid white reversible lane lines, dashed white lane lines in rural areas, and many others.

Fortunately for the subcommittee, a draft of this report became available in July 1967. The data and interpretations proved valuable in developing the basic concepts on which to base improvements to this part of the Manual. The public understanding of markings, as indicated by the random slide presentation, shows the effect of the conflicts noted previously. Those markings that have had consistent meanings rated high in general understanding. The best example is that solid yellow indicated traffic from the opposite direction, and that passing was unsafe. Another, dashed white indicated that passing was allowed.

On the other hand, those markings with multiple meanings in the present system resulted in confused interpretations when new systems were presented. For example, two-thirds of those tested felt a solid white line indicated no passing zone. Some of this may be due to older drivers remembering 15 to 20 years ago when that was its meaning. Another example, two-thirds do not expect opposing traffic on the other side of a broken white line.

The only obvious rationale is the likelihood that a significant number of the viewers were city oriented, where this symbol is used for lane lines. It is an unfortunate interpretation because literally hundreds of thousands of miles of rural highways use this symbol as the centerline of two-way, 2-lane roads. It seems that the broken yellow marking—not used in present system—indicates to most viewers that opposing traffic will be in other lane, but that sight distances are not restrictive.

Table 10 compares the meanings of the present system and the two alternates with the interpretations from the random slide presentation.

Of significance in the Committee effort to revise and improve the pavement marking system are the problems related to the transition period. During the changeover, the conflicting or alternative meanings for any given marking should not differ drastically from the present meaning; potential misinterpretations should be "fail safe."

Also important is a high probability of rapid comprehension of the new system. The "percent corrected" column of Table 2 suggests that alternate 1 is more readily "learned" than alternate 2. The values being 66 vs 56 percent and 64 vs 47 percent, respectively, for "Identification of Direction of Travel" and "Identification of Passing Safety."

It is quite likely that in the present system, the percent corrected is more likely the result of a memory jogger when the system rationale is explained. Figure 1 seems to corroborate the greater "learnability" of alternate 1 over all ranges of driving experience.

TABLE 10

Based on System Response Should Be	Type of Line			
	Solid Yellow	Broken Yellow	Solid White	Broken White
Indicates opposite direction:				
Present rural	Yes	—*	No	Yes & No
Present urban	Yes	—*	Yes	No
Alternate 1	Yes	Yes	No	No
Alternate 2	Yes	No	Yes	No
Random response	90%	77%	79%	34%
Indicates unsafe to pass:				
Present rural	Yes	—*	Yes	No
Present urban	Yes	—*	Yes	No
Alternate 1	Yes	No	Yes	No
Alternate 2	Yes	Yes	No	No
Random response	88%	37%	66%	9%

*No meaning assigned.

It appears from the field tests conducted in Phase III that two lines in combination are required to form a barrier line or no passing zone. It is probable that the low response to single solid white centerline as a barrier is due to the use of this marking as a centerline in urban areas where crossing is permitted. The greater deterrent value of the broken yellow over solid white is probably the combination of a new or novel technique with the carryover effect from present use of yellow only at no passing zones.

Table 8 indicates that both solid white and broken yellow lane lines tend to discourage crossing initially, but with passage of time (distance traveled) this reluctance is overcome. The similarity of the solid white and edge lines could account for this action by drivers. The novelty or caution meaning of the broken yellow line probably accounts for the initial hesitancy to cross this marking. The novelty characteristic of the broken yellow seems to be borne out by the interviews in Phase V.

The studies, especially the slide tests and the passing tests, indicate an undesirable amount of misunderstanding in the present markings system. However, the authors point out that understanding of any new systems is conditioned by driver experience with the present system. Thus, any proposed system must be deeply rooted in the existing system of markings. The system of marking being proposed by the NJCUTCD endeavors to eliminate the existing conflicts while retaining the most firmly rooted elements of the present system.

Essentially the proposed system is alternate No. 1, modified to use double lines to form the barrier or no passing line. Thus the solid yellow barrier line element of the two-line no passing marking is retained. The new symbol—broken yellow line—is used in the context most often understood (Table 1). The basic concepts on which the system is based are: (a) yellow lines delineate the separation of traffic flows in opposing directions; (b) white lines delineate the separation of traffic flows in the same direction; (c) broken lines are permissive in character; (d) solid lines are restrictive in character; and (e) width of line indicates the degree of restriction—a narrow line indicates less, and a wide line indicates greater restriction than a single normal width line.

ALAN T. GONSETH, Supervisor, Test and Applications, Port of New York Authority—Although safety and efficiency of highway operation depend to a considerable degree on the geometric design of the highway, the physical layout must also be supplemented by effective lane markings as a means of informing, warning, and controlling drivers. As with signs, respect for lane markings grows mainly from proper use because of the

natural tendency for lane markings to guide the road user along whatever path the striping outlines.

This report truly investigates the effectiveness of various combinations of line striping as related to three items: driver perception, driver understanding, and driver performance. The five-phase study fully evaluated various combinations of white, yellow, broken, and solid lines, and the conclusions of the authors are well founded and supported with substantial research data.

Since the findings are well documented and conservatively stated, I took the liberty of trying to find other relationships in their presentation that may lack solid back-up, but which might be indicative of some particular trend.

Although the authors never lose sight of their goal of determining which pavement marking is better for designating direction of travel and degree of safety, I feel an underlying point is suggested; that is, drivers who are properly taught can learn any striping system, but in practice will only consciously follow the "rules" if the rules really affect their personal safety or perhaps fit their convenience.

To illustrate, in the authors' random slide presentation people were basically unsure of meaning, but when the various striping systems were described and the slides presented orderly then all subjects "scored" much better. There was a slight edge for relearning the existing system rather than learning, for the first time, the two alternate schemes. In essence, Phase I indicates that people either do not remember the meanings of various pavement markings or unconsciously elect to disregard them and thereby use their own judgment rather than rely on markings.

The lane placement study indicates that people will tend to follow a clearly identified corridor and will position themselves with the most obvious striping, regardless of the striping's meaning or intent.

Similarly, the passing study showed that people used their own judgment in passing when the striping indicated it was unsafe. Did they consciously consider the no-crossing lines, or rather, did they just drive as they felt safe? This latter attitude appears particularly true on the level test section.

The 2- to 4-lane transition study again shows that people may temporarily react to lane striping, but tend to disregard it if they feel it is not applicable to their needs or convenience.

The 4- to 2-lane transition study indicates that only 16 percent of the drivers interviewed could properly identify the pavement marking on both the 2- and 4-lane sections. If you apply the results of Phase I to the 16 percent you find that even less people knew what the pavement marking meant.

My intent is not to be critical of the existing or any proposed alternate pavement marking scheme. I believe the authors have truly shown "... the results of these tests were influenced by driver conditioning" and, "The use of color appears to have a greater potential than the use of line shape." What the study does suggest to me is that we have been lax in promoting the existing uniform standard, and therefore, whatever standard is adopted should not be used properly and consistently throughout the nation as reinforcement to warning or regulatory signs, but most of all an increased public education program on the meaning of the lane markings should be initiated and frequently followed up.

Clearance Interval at Traffic Signals

ADOLF D. MAY, JR., ITTE, University of California, Berkeley

This research was a pilot study to identify promising modifications of amber period duration, transverse pavement markings, and supplemental advanced signing that gave evidence of improvements of safe operations at signalized intersections. The dilemma zone problem for minimum amber periods was extensively researched. It was found that increasing the amber phase at an urban location from 3 to 5 seconds increased the percentage of motorists operating in an unsafe or unexpected manner. Increasing the amber phase at the rural location from 5 to 7 seconds decreased the percentage of motorists operating in an unsafe or unexpected manner. However, it was found that the installation of experimental transverse pavement markings at the urban location slightly decreased the percentages of motorists operating in an unsafe or unexpected manner. Also, the installation of experimental transverse pavement markings at the rural location increased the percentage of motorists operating in an unsafe or unexpected manner.

•THE purpose of this study was to conduct a pilot investigation of traffic behavior as related to the amber period at traffic signals, and of possible modifications in amber period duration, advance signing, and additional pavement markings. The first part reviews current practice and discusses theoretical analysis. The second part is concerned with experimental field studies and includes the design of experiment and field work, film analysis, data processing and data reduction, and experimental field study results.[1]

CURRENT PRACTICE

Amber Law

To determine the prevalent practice regarding the amber clearance interval, a questionnaire (Fig. 1) was sent to 50 state highway departments, 32 major cities outside California, and 17 California cities. Of the 49 other states responding, 14 have laws similar to California's law that permits vehicles to legally enter the intersection during the amber period. On the other hand, 27 states have adopted laws requiring vehicular traffic to have completely cleared the intersection before the end of the amber phase. The remaining 8 states follow a law that falls in between the two extremes. A typical wording is: "Traffic facing the yellow signal shall stop before entering the nearest crosswalk at the intersection, but if such stop cannot be made in safety, a vehicle may be driven cautiously through the intersection." Table 1 outlines the three groups.

The applicable section of the Uniform Vehicle Code of the National Committee on Uniform Traffic Laws and Ordinances has been changed recently to allow vehicles to enter an intersection at any time prior to the termination of the amber phase. To compare, the 1956 edition states in Sec. 11-202-b-1: "Vehicular traffic facing the (yellow)

[1]The original manuscript contains a literature search, a list of references and an annotated bibliography which are not reproduced herein. This information is available from the Highway Research Board at cost of handling and reproduction. Refer to XS-12, Highway Research Record 221, 33 pp.

Paper sponsored by Committee on Traffic Control Devices and presented at the 47th Annual Meeting.

CLEARANCE INTERVAL

CURRENT PRACTICE QUESTIONNAIRE

Please complete both sides, using space on other side for explanations, and return to: University of California, Institute of Transportation and Traffic Engineering, 1301 South 46th Street, Richmond, California 94804.

I. WHAT IS THE INTERPRETATION OF YOUR APPLICABLE LAW REGARDING THE AMBER PHASE?

A vehicle must either stop before entering the intersection or must clear the intersection before the end of the amber phase.

A vehicle must either stop before entering the intersection or must have entered the intersection before the end of the amber phase.

Other. (please specify on other side)

II. DO YOU BELIEVE THE CLEARANCE INTERVAL (LENGTH OF AMBER) TO BE A PROBLEM?

No.

Yes. If so, why?

Safety

Capacity

Other. (please specify)

III. WHAT IS THE PREVALENT PRACTICE, REGARDING THE AMBER PHASE, IN YOUR CITY OR STATE?

Fixed amber time as a function of traffic conditions and/or location. e.g., 5 seconds for rural conditions and 3 seconds for urban conditions. Please specify.

secs. (urban)

secs. (rural)

Amber time based on approach speed. Please include graphs, equations, or explanation.

Additional phasing such as:

Green-amber phase

All-red phase.

Flashing green phase.

Other. (please specify)

Other. (please explain)

IV. AT PRESENT DO YOU HAVE ANY SIGNALIZED INTERSECTIONS WITH SPECIAL PHASING SUCH AS: (PLEASE INDICATE APPROXIMATE NUMBER AFTER DESCRIPTION)

Flashing green phase preceding amber phase. ________(approx. number)

Green-amber phase preceding amber phase. ________(approx. number)

All-red phase. ________(approx. number)

Numerical countdown or other display device. ____________(number) (please explain)

Other. (please explain)

V. HAVE YOU MADE ANY STUDIES OF THE CLEARANCE INTERVAL PROBLEM?

Yes.

Study now in progress.

No.

VI. IF THE ANSWER TO V IS YES, IS INFORMATION FROM THE STUDY AVAILABLE?

Information enclosed.

Information will be forwarded by separate mail.

Information may be found in ________________(name of article and date of publication).

No.

VII. WOULD YOU BE INTERESTED IN OBTAINING A COPY OF THE FINAL REPORT OF THIS PILOT STUDY?

Yes. No.

Please use the space below for explanations or further comments. Please attach an extra sheet if necessary. Thank you for your cooperation.

Figure 1. Current practice questionnaire.

TABLE 1

CURRENT PRACTICE CRITERIA

City-County	Law	Problems	Prevalent Practice	Special Phasing—No. of Intersections	Studies Made
			(a) Cities and Counties		
Outside California					
Albuquerque	—[a]	Safety	T = 3 to 4.1 sec, $T = f(V_0, W)$; all-red phase used infrequently	All-red phase, 4 to 5 intersections	No
Amarillo	—[a]	None	Urban T = 2.7 to 3.6 sec, Rural T = 3.2 to 4.0 sec; $T = f(V_0$, W, traffic type, turbulence). Some all-red	All-red phase, 4 intersections	Individual studies
Atlanta	—[a]	"Confusion"	Urban T = 3 sec, Rural T = 5 sec	All-red phase, 15-20 intersections	No
Baltimore	—[a]	Safety	Urban T = 3 sec, Rural T = 5 sec	All-red phase, 250 out of 1,038	No
Dallas	—[a]	Safety	$T = 0.8 + 0.04\,V + \frac{0.7\,W}{V}$; Urban T = 3.5 to 4 sec; Rural T = 4 to 5 sec. A few all-red	All-red phase; 15 intersections	No
Fort Worth	—[a]	None	$T = f(V_0 + W)$, all-red phase	All-red phase, 5 intersections	No
Minneapolis	—[a]	Safety and capacity	Urban T = 3 sec, Rural $T = f(V_0, W)$; all-red phase	All-red phase, 6 intersections	Yes
New Orleans	—[a]	Other	$T = f(V_0$, W, sight distance)	All-red phase, 6 intersections	Yes
New York	—[a]	None	$T = \Delta T + f(V_0) + 1.5$ to 2 sec of all-red "period." T = 4 to 5 sec	All-red "phase" (for pedestrians) 20 intersections	No
Phoenix	—[a]	None	$T = f(t, V, W)$; $T \leq 5$ sec	All-red phase, 249 out of 360 (being removed)	Yes
Pittsburgh	—[a]	Safety and capacity	T = 3 sec (minimum) of green-amber + 2 sec of amber	Green-amber, 575 intersections	In progress
San Antonio	—[a]	None	T = 3 sec for $V_0 \leq 35$ to 5 sec for $V_0 > 50$ mph	All-red phase, 10 intersections	No
In California					
Berkeley	—[a]	Safety and capacity	T = 3 sec min + $f(V_0, W)$	All-red phase, 3 intersections—pedestrain signals	Yes
Redding	—[a]	Safety and capacity	T = 3 to 5 sec	None	No
Outside California					
Akron	—[b]	Safety	Urban T = 3 sec; Suburban T = 4 sec for V = 35 mph; all-red for problem intersections	All-red phase, 15 intersections	No
Chicago	—[b]	None	Urban T = 3.25 sec, Suburban T up to 5 sec; all-red phase	All-red 80, a few green-yellow and red-yellow	No
Cincinnati	—[b]	None	T = 3 seconds generally	All-red phase, 290 intersections	No
Cleveland	—[b]	None	Urban T = 3 sec, Rural T = 5 sec if speed limit ≥ 50 mph	All-red phase at wide intersection; 35 intersections	No
Denver	—[b]	None	$T = 3 + \frac{V - 30}{10} + \frac{W - 50}{10}$; V = 30, 40, 50 . . . mph; W = 50, 60, 70 . . . ft; all red	All-red phase, 24 out of 937 intersections	In progress
Kansas City, Mo.	—[b]	Safety if T is short	$T = f(V_0, W, t)$; T = 3.5 to 5 sec	All-red phase, 10 intersections	No
Norfolk	—[b]	Safety and capacity	T = 3 to 5 sec, $T = f(V_0, W)$	All-red phase, 6 intersections	—
Rochester, N.Y.	—[b]	None	T = 3 to 4 sec	All-red phase, 10 intersections	No
St. Louis	—[b]	Safety	$T = f(V_0, W)$; T = 2.5 to 5.3 sec	All-red phase, 5 intersections	—
St. Paul	—[b]	Other	T = 3.5 to 4 sec	All-red phase, 6 intersections	No
Tempe, Ariz.	—[b]	Safety	Urban T = 4 sec, Rural T = 6 sec; $T = f(V_0)$	All-red phase, 7 intersections	Yes
In California					
Anaheim	—[b]	Safety and capacity	T = 3 to 4.5 sec for V = 20 to 60 mph	None	Yes
Burbank	—[b]	Safety and capacity	T = 3 sec (longer T for two locations); all-red phase	All-red phase, 10 intersections	Yes
Hayward	—[b]	Safety	Urban T = 3 sec, Suburban T = 4 sec	All-red phase, 1 intersection	No
Long Beach	—[b]	Safety and capacity	T = 3 to 4 sec; flashing green phase	All-red phase, 1 intersection	No
Los Angeles	—[b]	None	$T = f(V_0, W)$; T = 3 to 4.2 sec	All-red 20; "slot clearance" 100	No
Modesto	—[b]	None	T = 4 to 4.5 sec	None	No
Oakland	—[b]	None	Urban T = 3 sec; all-red and delayed red	All-red phase, 55 intersections	No
Riverside	—[b]	Safety	T = 3 to 4.5 sec (1 sec/10 mph); all-red phase	All-red phase, 6 intersections	No
Sacramento	—[b]	Safety and capacity	T = 3 sec min, $T = f(V_0)$	All-red period for wide intersections	No
San Diego	—[b]	Safety—citations	$T = f(V_0, W)$; few all-red for wide intersections	All-red phase, 3 intersections	Yes
San Francisco	—[b]	Safety	Usually T = 3 sec; T > 3 for wide intersections	None	No
San Jose	—[b]	Safety	Urban T = 5 sec, Rural T = 3 sec, $T = f(V_0$, W, accidents); all-red	All-red phase, 6 intersections	Yes—signal observance studies
Outside California					
Boston	—[c]	Safety and capacity	Urban T = 3 sec, Rural T = 3 sec; all-red	All-red phase, 70 intersections	No
Detroit	—[c]	Safety	All-red phase	All-red phase, 70 intersections	No
Milwaukee	—[c]	Safety—other	T = 3.5 sec min + all-red period	All-red phase, 575 intersections	In progress
Omaha	—[c]	Safety	$T = f(V_0)$; T = 3.0 sec for $V_0 = 25$ mph to 4.8 sec for $V_0 \geq 40$ mph	All-red phase, 2 intersections	No
Philadelphia	—[c]	Safety	T = 3 to 5 sec, $T = f(V_0, W)$	All-red phase, 50 intersections	No
Portland, Ore.	—[c]	Safety and capacity	Downtown: T = 2.25 sec + 1.35 sec all-red. Other: T = 3 + 1.2 sec all-red. V > 40 mph; T = 4 to 5 sec + 1 to 2 sec all-red	All-red, 550 intersections	Yes
Washington, D.C.	—[b]	Safety	Urban T = 3.6 to 4.2 sec; all-red phase	All-red phase, 120 intersections	In progress
Cook County, Ill.	—[c]	Safety and capacity	T = 3 sec both Urban and Rural		No
Louisville-Jefferson Co., Ky.	—[b]	Safety and capacity	Urban T = 3 to 4 sec, Rural T = 4 to 5 sec; all-red	All-red phase	
Alameda County, Calif.	—[b]	None	Urban T = 3 sec; Rural T = 5 sec	Right arrow during protected portion of other phase	No
Fresno County, Calif.	—[b]	Safety and capacity	Urban T = 3 ± sec, Rural T = 5 ± sec	None	No
Los Angelo County, Calif.	—[b]	None	Urban T = 3 sec, Rural T = 3 to 4.5 sec	All-red phase, 6 intersections	No

TABLE 1 (Continued)

State	Law	Problems	Prevalent Practice	Special Phasing—No. of Intersections	Studies Made
			(b) States		
Alabama	—[a]	None	Urban $T = 3$ sec, Rural $T = 4$ to 6 sec; all-red phase	All-red phase, 150 intersections	No
Alaska	—[a]	Safety	$T = f(V_0)$, $T = 3$ sec minimum up to 35 mph, to 5 sec for 50 mph	Green-amber phase, 1 intersection in Ketchikan	No
Arizona	—[a]	Safety	$T = t + \frac{V}{2d} + \frac{W + L}{V}$; all-red phase	All-red phase, 15 intersections	No
Arkansas	—[a]	Capacity	Urban $T = 3$ to 3 sec		No
Florida	—[a]	Safety and capacity	$T = 2$ sec after green arrow for left turns	All-red phase (N/A), numerical countdown, 1	No
Georgia	—[a]	Safety	Urban $T = 3$ sec, Rural $T = 4$ to 5 sec	All-red phase in 3 cities for high V_0 streets	No
Hawaii	—[a]	None	Urban $T = 4$ sec, Rural $T = 4$ sec		No
Idaho	—[a]	None	$T = 3$ to 5 sec; $T = f(V_0$, W, judgment)		No
Indiana	—[a]	None	Urban $T = 3$ sec, Rural $T = 5$ sec; all-red phase	All-red phase, 100 intersections	No
Iowa	—[a]	None	Urban $T = 3$ to 4 sec, Rural $T = 5$ to 7 sec	All-red phase used temporarily only at e.g., worksites	No
Louisiana	—[a]	Safety	$T = 3$ to 4.5 sec; all-red phase	All-red phase, 10 intersections	No
Michigan	—[a]	Safety	$T = f(V_0$, W, reaction time); all-red phase	All-red phase, 50 intersections out of 1,600	In progress
Mississippi	—[a]	None	Urban $T = 3$ sec, Rural $T = 5$ sec; all-red phase	All-red phase, 8 intersections	No
Missouri	—[a]	Safety and capacity	$T = 0.8 + 0.04\,V + \frac{0.7\,W}{V}$; all-red phase	All-red phase, 10 intersections	No
Montana	—[a]	Safety	$T = 0.8 + 0.04 + \frac{0.7\,W}{V}$; $T = 3$ to 5 sec; all-red phase	All-red phase, 1 intersection	No
Nebraska	—[a]	Safety	$T = 3$ to 5 sec; all-red phase	All-red phase, 15 intersections	No
Nevada	—[a]	None	$T = \frac{0.682}{V}$ (W, reaction and stopping distance), $3 < T < 5$ sec		No
New Hampshire	—[a]	Safety and capacity	Urban $T = 4$ sec, Rural $T = 6$ sec		No
New Mexico	—[a]	None	$T = 3$ to 4 sec for both Urban and Rural; all-red phase	All-red phase, 1 intersection	No
North Dakota	—[a]	None	$T = 0.8 + \frac{V}{22} + \frac{0.7\,W}{V}$; $3 \le T \le 6$ sec	All-red phase, 1 intersection	No
Rhode Island	—[a]	None	Urban $T = 3$ sec, Rural $T = 5$ sec	None	No
South Carolina	—[a]	Safety	$T = f(V_0)$	All-red phase, 20 intersections	Limited on safety
Texas	—[a]	None	$T = f(V, W) \le 5$ sec; all-red phase if $T > 5$ sec	All-red phase, number unknown	No
Utah	—[a]	None	$T = 4$ sec for both Urban and Rural	All-red phase, 50 intersections	No
Vermont	—[a]	Other	$T = 3$ to 5 sec; all-red phase, limited	All-red phase, 3 intersections (Under state jurisdiction)	No
Washington	—[a]	None	$T = 3.5$ sec for $V_0 = 25$ mph up to 5 sec for $V_0 = 60$ mph	All-red phase, 2 intersections	No
West Virginia	—[a]	Safety	$T = \frac{V}{10} \le 5$ sec; all-red phase	All-red phase; number not avaliable	No
Wyoming	—[a]	None	$T = 3 + 1$ sec for each 10 mph above 30. $T = 3$ to 5 sec		No
California	—[b]	Other	$T = f(V_0$, judgment)	All-red phase, 20 on very wide streets	In progress
Colorado	—[b]	Safety	$T = 3$ sec + 1 sec for each 10 mph above 30, $3 \le T \le 5$ sec; all-red phase	All-red, 6 intersections, yellow arrow being tried	Yellow arrow
Delaware	—[b]	Safety and capacity	Urban $T = 5$ sec, Rural $T = 3.5$ sec	All-red phase, 75 intersections	No
Illinois	—[b]	None	$T = f(V$, W, stopping distance)	All-red phase, 15 intersections	Limited on safety
Kansas	—[b]	Safety	$T = f(V_0$, W, traffic conditions); all-red phase (few)	All-red phase, 5 intersections	No
Kentucky	—[b]	None	$T = f(V_0)$; $T = 3$ sec up to 35 mph, to 5 sec at 50 mph	All-red phase, 10 intersections	No
Maine	—[b]	None	$T = f(V_0, W)$, $3 \le T \le 4$ sec; all-red phase if $T > 4$ sec	All-red phase, 10 intersections	No
Minnesota	—[b]	Safety and capacity	$T = f(V_0)$, $3 \le T \le 5$ sec; all-red phase	All-red phase, 20 intersections (rural)	No
New Jersey	—[b]	Other	$T = f(V_0)$; $T = 3$ sec + 1 sec for each 10 mph above 30; all-red phase	All-red phase, 800 intersections	No
North Carolina	—[b]	Safety	Urban $T = 3$ to 4 sec, Rural $T = 3$ to 5 sec; all-red phase	All-red phase, 100 intersections	No
Ohio	—[b]	None	$T = f(V_0)$; $T \le 4$ sec + all-red (1 to 3 sec); all-red phase	All-red phase, No. not available	No
Oklahoma	—[b]	None	$T = f(V_0)$; $T = 3$ sec for $V_0 \le 35$ mph, up to 5 sec for $V_0 \ge 45$ mph. All-red phase	All-red phase, 50 intersections	No
South Dakota	—[b]	Safety	Urban $T = 3$ normally	None	No
Tennessee	—[b]	Safety	Urban $T = 3$ sec, Rural $T = 3$ sec; all-red phase	All-red phase	No
Wisconsin	—[b]	Safety	$T = 3$ sec minimum to 6 sec maximum; all-red phase	All-red phase, 5 intersections	No
Connecticut	—[c]	Safety	$T = f(V_0$, W, accidents); $3 \le T \le 5$ sec	All-red phase, 150 intersections	No
Maryland	—[c]	None	Urban $T = 3$ to 4 sec, Rural $T = 5$ to 6 sec; all-red phase	All-red phase, 40 intersections	No
New York	—[c]	Safety	$T = f($observed $V_0)$, $3 \le T \le 5$ sec; all-red phase	All-red phase, 100 intersections	Under consideration
Oregon	—[c]	Safety	$T = 4$ to 8 sec, $T = f(V_0$, W, judgment)	All-red phase, 200 in Portland only	No
Pennsylvania	—[c]	Safety	$T = 3$ to 5 sec, $T = f(V_0, W)$; all-red phase	All-red 250, green-yellow 575 in Pittsburgh city	No
Virginia	—[c]	None	Urban $T = 3$ to 4 sec, Rural $T = 4$ to 5 sec; all-red phase	All-red phase, 10 intersections	No

Note: T = amber phase duration; t = reaction time; V_0 = approach speed; W = width of intersection.

[a]A vehicle must either stop before entering the intersection or must clear the intersection before the end of the amber phase.

[b]A vehicle must either stop before entering the intersection or must have entered the intersection before the end of the amber phase.

[c]Other.

TABLE 2

SUMMARY OF CURRENT VIEWS OF THE CLEARANCE INTERVAL PROBLEM

Responses	Law	Clearance Interval Problem: No	Safety	Capacity	Safety and Capacity	Other	Subtotal	Total Responses
Cities in Calif.	Permissive	5	4	0	7	1	17	17
Cities outside Calif.	Restrictive	5	3	0	2	2	12	
	Permissive	6	4	0	2	1	13	32
	Other	0	3	0	3	1	7	
States	Restrictive	14	9	1	3	1	28	
	Permissive	6	6	0	2	2	16	49
	Other	1	4	0	0	0	5	

signal is thereby warned that the red or 'stop' signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red or 'stop' signal is exhibited." The 1962 edition states in the same section: "Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection." This change effected by the Uniform Vehicle Code has stimulated a few states to revise their law toward conformity with the recent code.

View of Clearance Interval Problem

Twelve of the 17 California city and county agencies viewed the clearance interval at traffic signals as a problem because of safety implications or the combination of safety and capacity considerations. Twenty-one out of 32 cities and counties outside California had similar views. The prevalent law regarding the amber phase, whether restrictive or permissive, does not seem to have a significant bearing on their view of the problem—61.5 percent and 54 percent respectively (Table 2). This is somewhat different for the states responding. Forty-eight percent of those following a restrictive law viewed the clearance interval as a problem, compared to 62.5 percent of the states adopting a permissive law. This is somewhat contradictory to the fact that an intersection is hazardous from a safety point of view if the law adopted is restrictive and requires the traffic to clear the intersection when operating under a theoretically inadequate clearance interval. The cities appeared to consider the clearance interval more of a problem than did the states.

Prevalent Practice

The prevalent practice in California city and county agencies (as reported by 12 of the 17 agencies) was the fixed amber time (Table 3). The other 5 agencies modified the

TABLE 3

SUMMARY OF CURRENT PRACTICES CRITERIA

Responses	Law	Prevalent Practice: T Fixed	T = $f(V_0)$	T = $f(V_0, W)$	T = $f(V_0, W, \text{Other})$	All-Red	Special Phasing: All-Red	Green-Amber	Other	None	Studies Made: Yes	In Progress	No	Sub-total	Total Responding
Cities in Calif.	P	12	1	4	0	3	10	0	1	6	4	0	13	17	17
Cities outside Calif.	R	3	2	4	3	5	11	1	0	0	3	1	8	12	
	P	7	2	3	1	4	13	0	0	0	1	2	10	13	32
	O	3	2	1	0	5	7	0	0	0	1	1	5	7	
States	R	14	5	6	3	13	21	1	(1)	6	1	1	26	28	
	P	5	6	2	2	10	14	0	(1)	1	1	1	13	15	49
	O	2	1	1	2	4	6	(1)	0	0	0	0	6	6	

Note: P stands for permissive law, R for restrictive law, and O for other type of law. Numbers in parentheses indicate states listed under more than one criterion. (Also see Highway Research In Progress, HRB, 1967.)

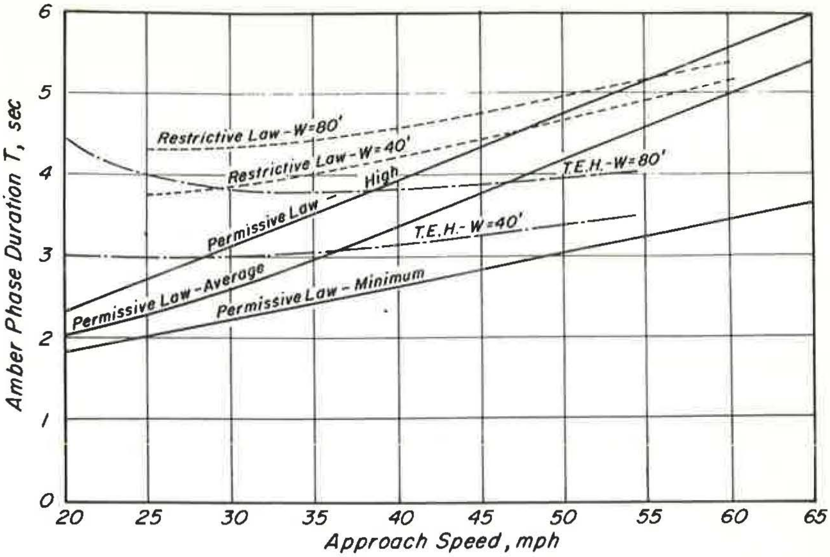


Figure 2. Comparison of amber phase duration for different practices.

amber time by considering approach speed and/or geometry. Three agencies reported the practice of using an all-red phase. Cities outside California having restrictive laws reported to have practices in conformity with that law (9 out of 12 used amber time modified according to speed, geometry and other parameters compared to 6 out of 13 agencies having permissive laws).

Of the 28 state highway departments following restrictive laws, 14 had fixed amber time. Thirteen had the all-red phase as prevalent practice, used mostly in situations where the amber time needed was excessive. The 15 states having permissive laws had a smaller percentage adopting fixed amber time and a considerably larger percentage using approach speed for modification (Table 3). Ten states employed the all-red phase at their intersections.

Most highway departments and city and county agencies limit the amber phase between 3 and 5 sec, which is in conformity with the Manual of Uniform Control Devices recommendations. Very few went below 3 sec or exceeded 6 sec where an all-red overlap was used if warranted by extraordinary geometric conditions, high approach speeds, high accident rate, or heavy turning movements. Authorities requiring vehicles to have cleared the intersection generally require longer clearance intervals and are probably unable to use the amber phase as a partial extension of the green interval and thereby increase intersection capacity. To show this, three types of practice are compared (Fig. 2). The first is the 1950 edition of the Traffic Engineers Handbook based on

$$Y = 0.8 + 0.04V + \frac{0.7D}{V}$$

where Y is the amber phase duration in sec; V, the speed in mph; and D, the intersection width. The second is Michigan's, obtained from a graph and adopted here as an example for the restrictive law practice. The third is that obtained by using

$$V_0 t_2 + \frac{V_0^2}{2d} = V_0 T + \frac{a}{2} (T - t_1)^2$$

with the following parameters (note that this equation deletes the width of intersection and thereby represents the permissive law):

Parameter	Minimum	Average	High
t_1, sec	0.40	0.85	1.20
t_2, sec	0.75	1.00	1.47
a, ft/sec^2	3.00	13.50-0.145 V_0	16-0.145 V_0
d, ft/sec^2	8.00	13.00	16.00

where T is the amber phase duration; V_0, the approach speed; t_1 and t_2 are the reaction times to clearing and stopping; and a, d are the acceleration and deceleration limits, respectively. The expressions (T is in sec and V_0 is in mph) resulting from these parameters are

Minimum, $T = 1.01 + 0.0397\ V_0$
Average, $T = 0.747 + 0.0585\ V_0 + 0.000166\ V_0^2$
High, $T = 0.366 + 0.0928\ V_0 - 0.000121\ V_0^2$

Special Signal Phasing

Among the special signal phasing, the all-red was predominant and is being used in varying degrees by almost all the agencies (Table 3). This phasing—a red overlap after the yellow interval—is favored mostly at accident-prone intersections where accidents of the right-angle collision type are frequent. It is also used by states that limit the length of the amber phase short of the clearance period required. This usually occurs at very wide intersections or at those exhibiting high approach speeds. The all-red interval is also favored for special traffic conditions where heavy turning movements occur, or at intersections with heavy commercial traffic. Some agencies, however, are proposing to discontinue the use of the all-red phase or have already discontinued it. The reasons include the danger of confusion when both a "pure amber" and an amber followed by all-red are being used in the same city. Also, the practice of turning left on red is in violation of certain traffic laws.

On the other hand, from a red overlap study in Portland, it was concluded that the removal of the all-red phase increased the accident rate and was unfavorable, even though intersections in the CBD showed increase in the highest average vehicles per cycle, and averaged less accidents for the test period. The study concludes justifying the red overlap.

Pittsburgh utilizes a green-amber phase of 3 to 5-sec duration before a standard amber interval of 2 sec at its 575 intersections, and has had favorable experience with it. The same phasing is used in Ketchikan, Alaska, for the only signal in the city. Florida and Abilene, Texas, are currently experimenting with numerical countdown devices.

THEORETICAL ANALYSIS

This section has two specific objectives. The first is to determine the required amber duration for a variety of conditions and as influenced by type of law. The second is to determine under what set of conditions dilemma zones would exist.

General Kinematics

A vehicle approaching a signalized intersection, when faced with the amber indication, will either have to stop or proceed to clear the intersection. In the latter case, a motorist can only accelerate when approaching the intersection at lower than the approach speed limit.

Certain relations of kinematics are important in conjunction with stopping and clearing. Curves should be smooth and with limited peaks. Speed profiles (Fig. 3) are good indications for patterns of drivers' reactions to the amber phase. Deceleration curves should have a limited peak, depending on comfort and convenience. A. D. May outlines

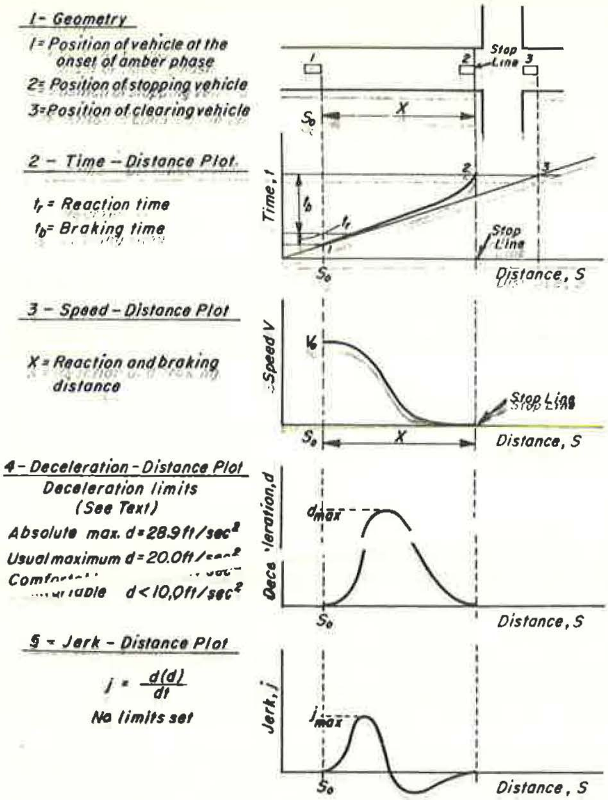


Figure 3. Intersection kinematics.

in the "Traffic Engineering Handbook" various values for acceleration and deceleration characteristics. Up to 1963, the absolute maximum deceleration under ideal conditions was 28.9 ft/sec^2 and the maximum under usual conditions was about 20 ft/sec^2. Practical values for deceleration used in everyday traffic conditions rarely exceed 8 to 9 ft/sec^2. Acceleration on the other hand has a more variant characteristic because it depends on the vehicle's power plant and the speed of travel. D. Gazis used a value of 16 - 0.145 V_0 for Detroit models, V_0 being the speed in ft/sec. No limits or values have yet been ascertained relative to the effect of time change of acceleration or deceleration (jerk in ft/sec^3).

Influence of Type of Law on Required Amber Duration

The portions of the two extreme laws pertinent to this analysis are (a) a vehicle must either stop before entering the intersection or must clear the intersection before the end of the amber phase; and (b) a vehicle must either stop before entering the intersection or must have entered the intersection before the end of the amber phase. Figure 4 shows the geometry of intersections for cases (a) and (b). Note that for the first case, both the effective width of intersection and the length of the vehicle are considered in W, whereas in the second case only the latter dimension L (or part of it) is included.

From the geometry of intersections, the following equations can be deduced as outlined by Gazis, relating stopping and clearing distances, reaction times, and approach speeds:

Case (a)

Stopping distance,

$$X_S \geq \frac{V_0^2}{2d} + V_0 t_2 \qquad \text{(a-1)}$$

Clearing distance,

$$X_C \leq V_0 T + \frac{a}{2}(T - t_1)^2 - W \qquad \text{(a-2)}$$

Case (b)

Stopping distance,

$$X_S \geq \frac{V_0^2}{2d} + V_0 t_2 \qquad \text{(b-1)}$$

Clearing distance,

$$X_C \leq V_0 T + \frac{a}{2}(T - t_1)^2 - L \qquad \text{(b-2)}$$

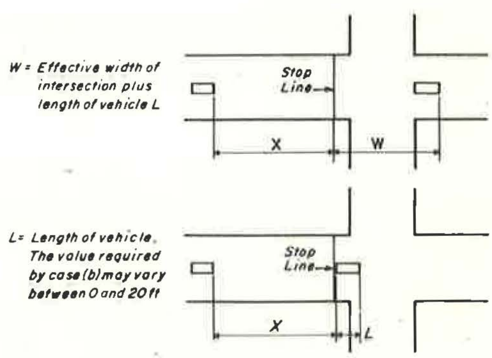


Figure 4. Geometric layouts of intersections for cases (a) and (b).

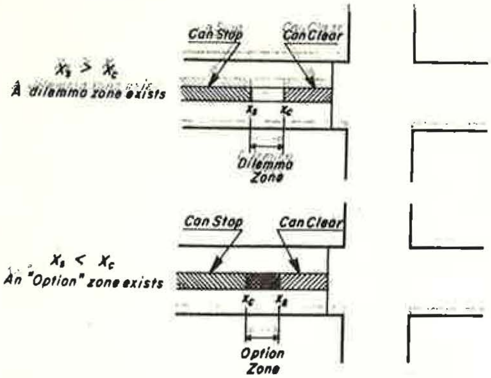


Figure 5. Schematic drawing for dilemma and option zones.

where

X_s, X_c = distance from stop line when the amber phase commences, ft;
V_0 = approach speed, ft/sec;
T = amber phase duration, sec;
L = length of vehicle, ft;
W = gross width of intersection (effective width plus length of vehicle), ft;
a = acceleration rate, ft/sec^2;
d = deceleration rate, ft/sec^2; and
t_1, t_2 = reaction times to accelerate and decelerate respectively, sec.

Three conditions can, however, exist relative to the dimensions of the stopping distance X_S and the clearing distance X_C:

Condition I

$X_S > X_C$: A dilemma zone exists within which the driver could neither stop safely nor clear the intersection (Fig. 5). This condition is more pertinent to case (a).

Condition II

$X_S = X_C$: The dilemma zone in this case is deleted.

Condition III

$X_S < X_C$: An option zone exists within which a driver can choose between stopping and clearing the intersection (Fig. 5).

In order to have a dilemma zone-free situation, the length of the amber phase can be obtained for two basic conditions.

Condition I—A vehicle is approaching the intersection at the approach speed limit. Clearance of the intersection is not accompanied by acceleration. Thus, assuming a = 0, and equating the expressions for stopping and clearing distances, the following equations result:

$$T_a = t_2 + \frac{V_0}{2d} + \frac{W}{V_0} \qquad \text{(a-3)}$$

$$T_b = t_2 + \frac{V_0}{2d} + \frac{L}{V_0} \qquad \text{(b-3)}$$

Since L can be expressed as a function of W, Eq. b-3 may be considered as a special case of Eq. a-3. Figure 6 shows a plot of $T_{min} - t_2$ (amber phase duration less the reaction time to stopping) vs approach speed V_0 for various intersection widths, W.

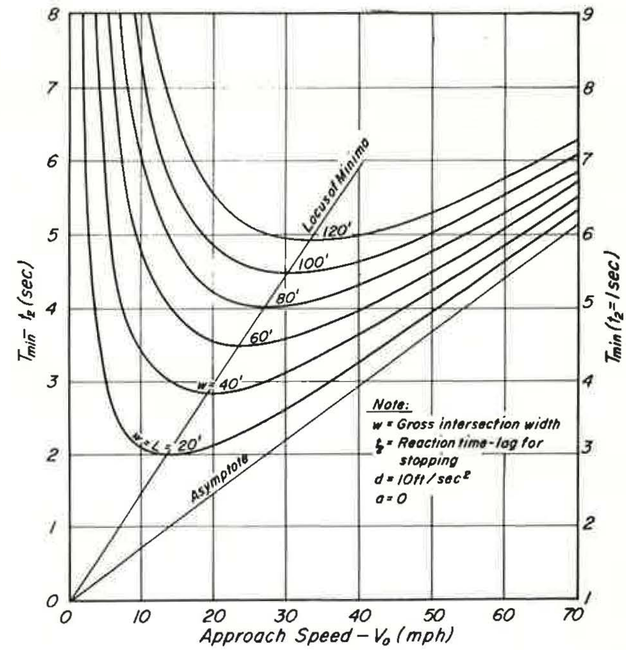


Figure 6. Variation of the minimum amber period T_{min} vs constant approach speed for dilemma zone-proof operation.

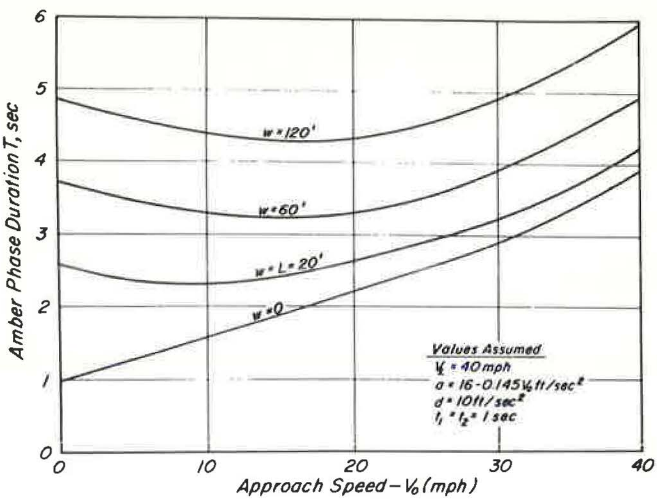


Figure 7. Length of amber phase vs approach speed for Condition II (a driver can accelerate when clearing the intersection).

The amber period T is subscripted as minimum because the value of deceleration is taken to be a "maximum practical" value of 10 ft/sec^2. Case (b) is represented by the lowest curve of the series having L = 20 ft (could be less), whereas case (a) is represented by curves W = 40 ft through 120 ft.

Condition II—A vehicle is approaching the intersection at less than the approach speed limit, and can accelerate to clear the intersection. In this case the approach speed limit must be specified. Acceleration is dependent on the approach speed, and a value $a = 16 - 0.145\ V_0$ of Gazis is adopted here. The stopping distance remains the same:

$$X_S \geq V_0 t_2 + \frac{V_0^2}{2d} \tag{4}$$

The clearing distance is dependent on whether the speed limit V_ℓ is attained within or outside the intersection.

In the first case

$$T \geq t_1 + \frac{V_\ell - V_0}{a}$$

and

$$X_C \leq V_0 t_1 - W + \frac{V_\ell^2 - V_0^2}{2a} + V_\ell\left(T - t_1 - \frac{V_\ell - V_0}{a}\right) \tag{5}$$

In the second case

$$T \leq t_1 + \frac{V_\ell - V_0}{a}$$

and

$$X_C \leq V_0 T - W + \frac{a}{2}\,(T - t_1)^2 \tag{6}$$

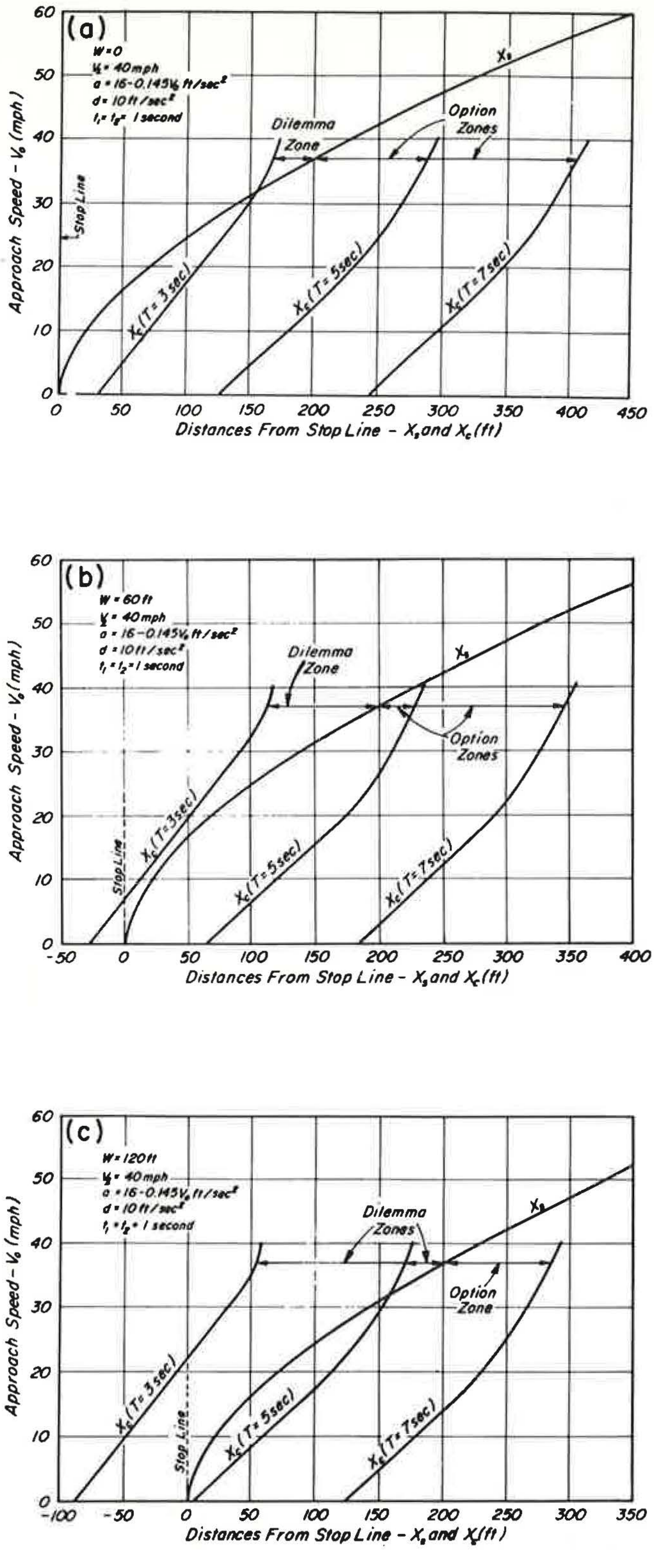


Figure 8. Stopping and clearing distances vs approach speed: (a) W = 0 ft; (b) W = 60 ft; (c) W = 120 ft.

The same relations for case (b), which requires a vehicle to just enter the intersection, can be arrived at with W either equal to zero or having a small value L. Variation of T vs V_0 is shown in Figure 7 for the values V_{ℓ} = 40 mph, a = 16 - 0.145 V_0 ft/sec^2, d = 10 ft/sec^2, and $t_1 = t_2$ = 1 sec. W is given values between zero, and L = 20 ft for case (b) and up to 120 ft for case (a). In this condition where a motorist can accelerate in order to clear the intersection, the curves of T vs V_0 become no longer asymptotic to the line $V_0 = 0$ as in the first condition where V_0 is assumed constant. Instead, a finite value for T at V = 0 results.

Dilemma and Option Zones

A direct way of showing the existence of a dilemma zone is to plot the stopping and clearing distances vs the approach speed. Such plots are shown in Figure 8 for various values of amber phase duration T, and gross intersection width W. Eqs. 4, 5, and 6 are used with a = 16 - 0.145 V_0 (V_0 in ft/sec), d = 10 ft/sec^2, $t_1 = t_2$ = 1 sec. It is assumed here that the speed limit V_{ℓ} is 40 mph, and that vehicles approaching the intersection at lower than the speed limit can only accelerate up to that limit.

DESIGN OF EXPERIMENT AND FIELD WORK

Controlled Conditions

Single approaches to a typical urban and rural signalized intersection were to be studied under four controlled conditions:

1. Clearance interval set according to existing state practice;
2. Clearance interval of longer duration to eliminate any possible dilemma zone;
3. Signalization supplemented by advance signing; and
4. Signalization supplemented by additional pavement markings.

Site Characteristics

To make the study as general as possible and to facilitate photographic work, a number of site characteristics were desired. They included:

1. Approach with two through lanes,
2. No left-turn lane,
3. No left-turn phasing,
4. Four-approach intersection,
5. Level slope of approach,
6. On state highway,
7. Good photo approach and camera location,
8. No advanced pavement sensors,
9. Urban approach speed 20 to 35 mph,
10. Rural approach speed 40 to 60 mph, and
11. Electric power available.

LOCATION / CONTROLLED CONDITIONS	URBAN-SAN PABLO AT VALE		RURAL-RT. 4 AT SUMMERVILLE		JUNIPERO SERRA BLVD AT CLAY AVENUE
	NORMAL CONDITIONS	CONTROLLED CONDITIONS	NORMAL CONDITIONS	CONTROLLED CONDITIONS	NORMAL CONDITIONS
Clearance Interval Existing State Practice	1 29 Mar. 1967 T=3 sec	3 27 May 1967 T=3 sec	5 22 Mar. 1967 T=5 sec	7 27 May 1967 T=5 sec	9 23 May 1967 T=4 sec
Clearance Interval Increased Duration	2 3 and 16 May 1967 T=5 sec		6 4 May 1967 T=7 sec		
Supplemental Advanced Signing					10 19 July 1967 T=4 sec
Supplemental Pavement Markings		4 3 June 1967 T=3 sec		8 3 June 1967 T=5 sec	

Figure 9. Modified design of experiment (shown in various cells are dates of filming and amber periods involved).

Modified Design of Experiment

The original design of experiment was a 2 × 4 matrix for studying two locations under four controlled conditions. To take advantage of an already scheduled installation of a "prepare to stop" sign and in order to use "controlled devices" in the field tests involving pavement markings, the design of experiment was modified to include five pairs of controlled conditions. This modified design is shown in Figure 9.

Site Selections

All state highway intersections under district 04 jurisdiction were studied. After careful investigation, two

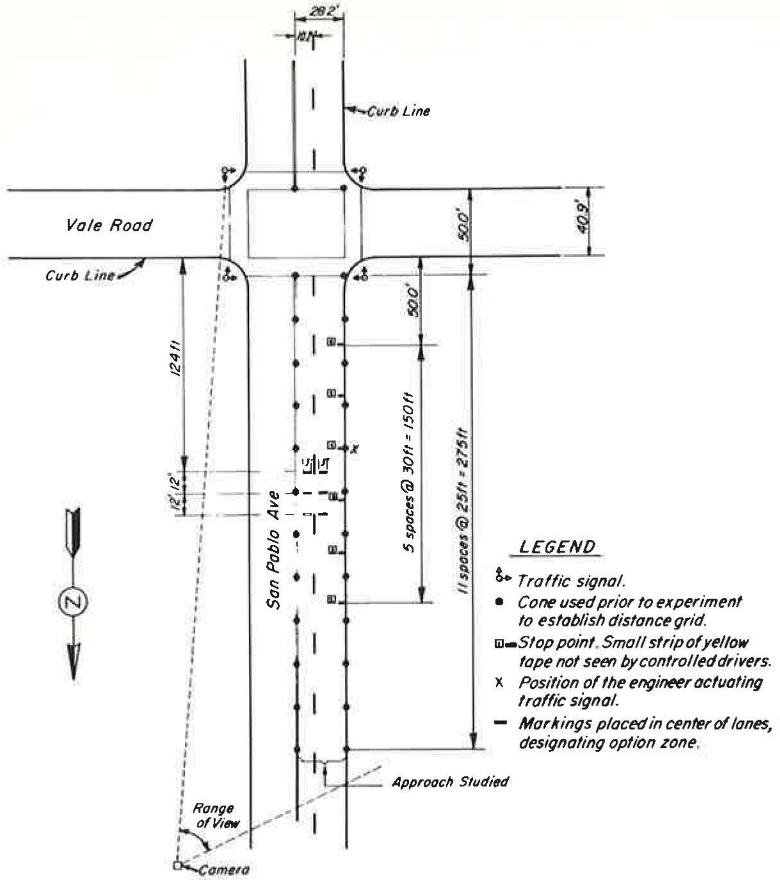


Figure 10. Urban intersection: San Pablo Ave. at Vale Rd., experiment layout for lanes studied.

intersections were selected. The urban site was located at the intersection of San Pablo Avenue (US 40) and Vale Road in San Pablo (Fig. 10), and the rural site was located at the intersection of Route 4 and Summerville in Antioch (Fig. 11). The urban location met all of the site characteristics fairly adequately, whereas the rural location had both left-turn lane and left-turn signal phasing, but it was the best location observed. It was decided that these locations were to be used for normal amber, lengthened amber, and advanced pavement marking phases of the study. A third location was afterwards selected to be used for advanced display phase under normal conditions. This site was located at Junipero Serra Blvd. and Clay Ave. in South San Francisco (Fig. 12).

Camera Setup

A 16-mm pulse camera (MK 100 ES) was used at a speed of two frames per second. The film was 7241-EF Ektachrome high-speed 100 color-type film. There were 400 ft per roll, which at 2 frames per second lasted approximately 2 hours. For the normal conditions, the standard amber and the lengthened amber phases, filming was done either Tuesday, Wednesday, or Thursday; Monday and Friday being thought of as rather extraordinary traffic days for all locations. For each phase of the study, it was decided to take two reels of film. The filming was carried out between 1:00 p.m. and 6:00 p.m. It was assumed and also noted from preliminary observations that the first reel taken from 1:00 p.m. to 3:30 p.m. was light traffic, and from 3:30 p.m. to 6:00 p.m. was heavy traffic.

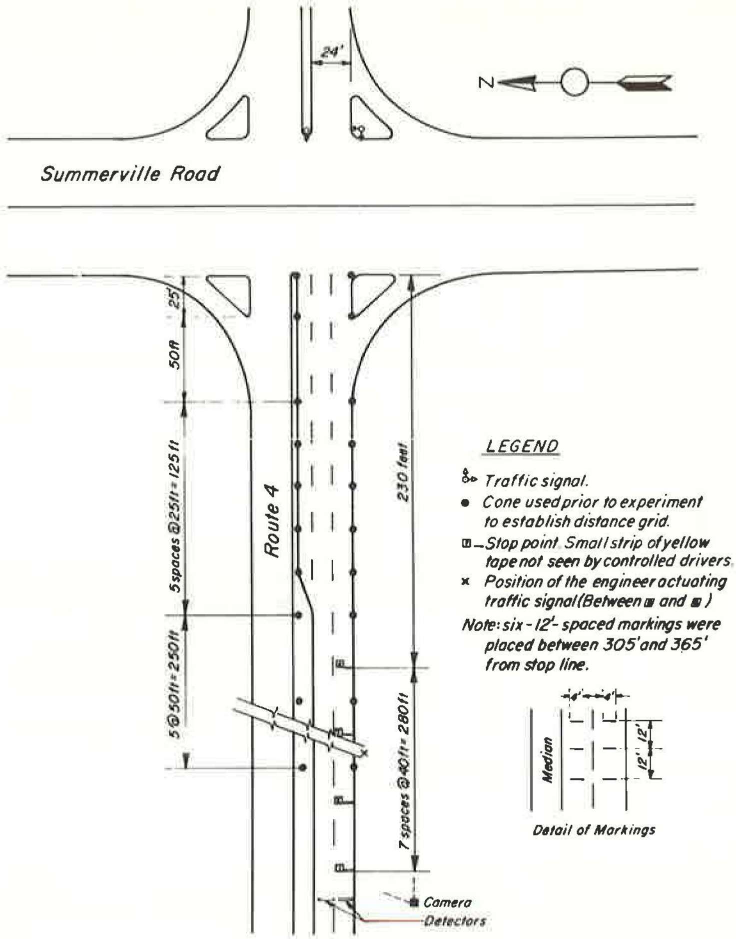


Figure 11. Rural intersection: Route 4 at Summerville Rd., experiment layout.

Two consecutive Saturdays were chosen for filming the advanced pavement marking phase. Since the only vehicles of interest were the control-driver vehicles, Saturday was a convenient time, the traffic conditions being fairly light. Filming was done only when the control vehicles were in range of the camera (the camera was fixed in all cases). Eight controlled drivers were required, and each driver made approximately 15 runs for each location and controlled condition (see Fig. 9 for date of each filming).

The camera at San Pablo and Vale was positioned on top of a ladder placed on top of a panel truck in an adjacent vacant lot where electricity was available. The camera at Route 4 and Summerville was on the roof of an adjacent building. The camera at Junipero Serra Blvd. and Clay Ave. was on a hillside adjacent to Junipero Serra Blvd. At San Pablo and Clay Ave. locations, the signal could be seen well enough for analysis purposes. However, at Summerville, the signal was not easily definable. Therefore, a light bulb was connected in the camera range to the signal so that it would be on only when the signal was in the amber phase.

Photographic Processes and Controlled Conditions

Traffic cones were placed at intervals on the sides of the lanes to use as a reference. These cones were temporarily placed at the beginning of each setup, photographed for 3 min, and then taken up so they would not influence traffic (see Figures 10, 11, and 12 for cone layouts).

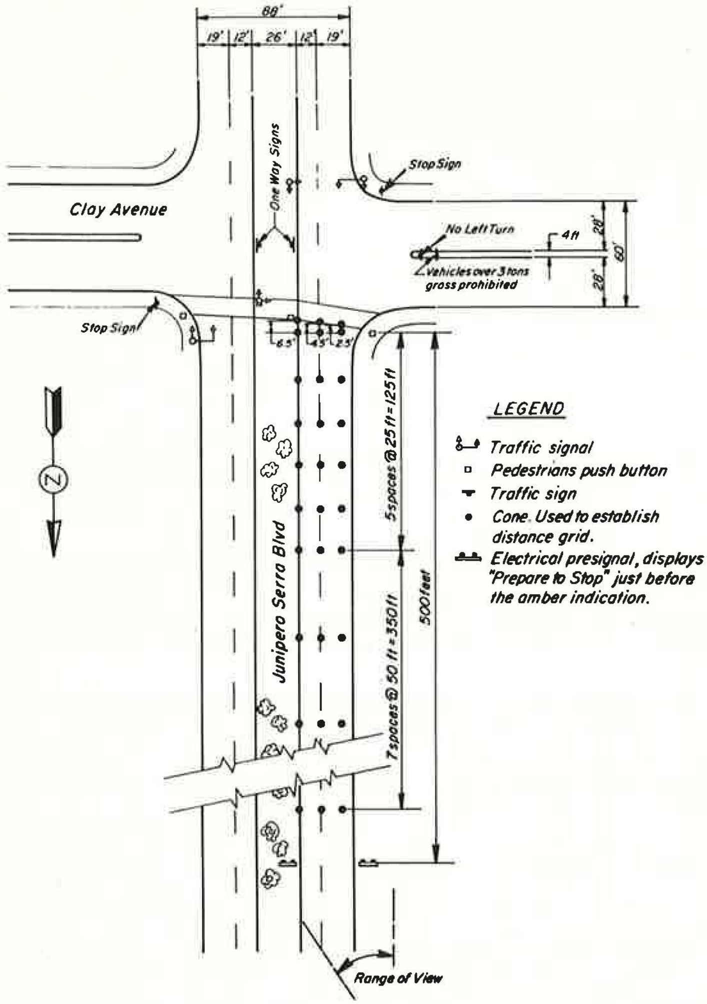


Figure 12. Junipero Serra Blvd. at Clay Ave., experiment layout.

To establish the feasibility of the different sites and camera locations, several preliminary films were made. Cone layouts were included to determine the accuracy of our measurements and reasonable cone positions. The cones could be more easily seen if painted bright orange instead of the standard yellow. All three sites were photographed with existing conditions with the state standard length of amber: at San Pablo and Vale Road, amber = 3.0 sec; at Route 4 and Summerville Road, amber = 5.0 sec; and at Junipero Serra Blvd. and Clay Ave., amber = 4.0 sec.

The intersection at South San Francisco was peculiar in that it is actuated only by pedestrians. During slack pedestrian periods, it was necessary to push the actuator to obtain a fairly standard cycle.

Later, San Pablo and Route 4 were photographed with the lengthened amber period: 5.0-sec amber at San Pablo, Vale; and 7.0-sec amber at Route 4, Summerville. Special precautions were taken at Summerville Road in lengthening the amber. It was stepped up in two stages before being photographed: one week at 6.0 sec and one week at 7.0 sec.

San Pablo and Route 4 were next photographed under the existing conditions without and with pavement markings using controlled drivers. For both with and without pavement marking phases, the drivers were instructed to drive at the approach

TABLE 4

MATRIX OF VEHICLES CAUGHT IN VARIOUS APPROACH SPEED-STOP POINT COMBINATIONS

Condition	Speed	Stop Point							
		1	2	3	4	5	6	7	8
(a) San Pablo at Vale (urban)									
Before markings	25	10	9	7	6	6	6		
	30	9	6	6	6	6	11		
	35	6	9	9	8	9	6		
With markings	25	7	6	6	6	6	6		
	30	6	6	6	7	6	6		
	35	6	7	6	6	7	6		
(b) Route 4 at Summerville (rural)									
Before markings	40	13	6	6	7	6	7	6	6
	50	6	6	6	8	10	6	6	-
	60	6	13	10	6	6	6	-	-
With markings	40	-	-	6	6	6	6	6	6
	50	4	6	6	6	6	6	6	6
	60	6	6	6	6	6	6	6	-

speed specified in their log; at Route 4 these speeds were 40, 50, and 60 mph, and at San Pablo, were 25, 30, and 35 mph. In the without-markings phase, the drivers were told to react as normally as possible to the signal. In the with-markings phase, the drivers were told that the markings were designed for 50 mph. The drivers were told if they did not reach the markings when the light turned amber they were advised to stop. If they were in the zone, the drivers had the option to stop or go through. However, they were told to still rely on their own judgment, and the markings were to be only an aid. These markings were made of strips of special epoxy bonded tape, 8 in. wide (Figs. 10 and 11).

To get a spread of stopping distances, a number of stop points were created and the signals were wired for semimanual operation. The points were uniformly spaced on the pavement (unseen by the drivers) and randomly selected so that for each point there would be a minimum of 6 control vehicles for each approach speed-stop point combination (see Table 4).

The signals were operated by a traffic engineer stationed some distance back from the intersections. When the control vehicles passed a specified stop point, the engineer started the cycle. At San Pablo when the cycle was started, the signal would turn instantaneously yellow if at least 10 sec of green had elapsed; the average cycle was approximately 65 sec. Six stop points were required and laid out (Fig. 10). They were placed so the cars at higher speeds would almost always stop for the points farthest from the intersection, and cars at lower speeds for points nearest the intersection would almost always pass through. At Route 4 eight stop points were required. When the cycle was started, there was a 1.8-sec delay before the light would turn amber if at least 20 sec of green had elapsed; the average cycle was approximately 85 sec. This was due to the behavior of the signal mechanism. For this reason, the stop points were placed approximately 100 ft farther back than if the signal mechanism had been instantaneous.

The last filming operation was carried out for Junipero Serra Blvd. at Clay Ave., under the controlled condition of an advanced sign placed 500 ft from the intersection with Clay Ave. The sign displays a "prepare to stop" message just prior to the beginning of the amber phase and returns to a "blank-out" state just before the end of the red phase.

FILM ANALYSIS, DATA PROCESSING AND DATA REDUCTION

Film Analysis

The purpose of the film analysis was to determine the time-distance trace of vehicles passing through the intersections. Specifically, this time-distance function was needed to determine driver reaction to the amber phase of the signal.

To obtain the time-distance trace, two requirements had to be fulfilled: a time relationship had to exist between different pictures taken of one particular car, and a distance grid had to be established on the road. The first requirement was satisfied by using time-lapse photography. Pictures were taken of the intersection at ½-sec intervals, establishing a time relationship between frames. Also, continuous movies could be made of the intersection because of the relatively small film requirements: 100 ft of film could record more than 30 min of events.

The second requirement was satisfied by establishing a grid system on the road. Before filming began, highly visible traffic markers were placed at equal distance intervals on both sides of the road. Next, the first few frames of film were used to record their location. Later, these frames were used to reestablish the grid. Specifically, what was done was to project a frame on which the cones were recorded on a sheet of paper. Then the position of the cones was recorded on the paper, together with a number of reference marks (for example, the centerline of the road, road signs, or telephone poles). Using the positions of the cones, a grid could be drawn. The distances between the cones were known, and thus a calibrated "master" could be made from which time-distance traces of individual vehicles could be obtained. Since in this study the needed results were driver reaction to the amber phase of the signal, screening had to be done in choosing the cars which were to be traced through the intersection. Several criteria were used in the screening procedure.

Driver' reaction should be almost exclusively due to the amber phase of the signal. This type of reaction is opposed to the reaction due to heavy traffic, an intended left or right turn, or police interference. Furthermore, the vehicle had to be sufficiently close to the intersection when the light turned amber so that the driver had to make a real decision regarding whether he should go through.

To determine the trace of the vehicle, the consecutive frames of a particular car were projected on a sheet of paper. A distinctive feature on the car was then plotted on the paper for each frame. At the rural location the front tire was plotted, in the urban location the left-front tire was plotted, while at the Clay Ave. location the left-rear bumper was plotted. In addition to these points, a number of reference marks were recorded so that later the trace of the vehicle could be superimposed on the master.

Once the vehicle trace and reference marks were plotted, the trace was superimposed on the master, overlaying the identical reference marks. The position of the car in the distance grid could then be read off the master. To determine the position of the car relative to the stop line of the intersection, linear interpolation was used for those distances that fell between the grid lines determined by the traffic cones. The distances were then recorded on the specially designed data sheets.

Data Sheets and Data Cards

The format of the data sheets (Fig. 13) was designed primarily so that later the data could be easily transferred to IBM cards for computer reduction. For each trace, a set of data was made: the first card contained certain parameters; the second card, identification and data of 12 position points of the car; and, if more position points were

CARD I

Identif. No. 1 2 3 4	5	No./Frames 6 7	
9052		14	

CARD II

Ident. No. 1 2 3 4	Location & Control 5	Catch No. 6 7 8	Vehicle No. 9 10	Veh. Type 11	Movement 12	Extraordinary Movement 13	Frame No. of 1st Amber frame 14 15 16 17 18	No. frames when intersection is clear 19 20
9052	6	005	02	5	6	4	11627	0 4

Location & Control Code:
1 = Rural-Existing
2 = Rural-Increased Amber
3 = Rural-Marking
4 = Rural-Controlled
5 = Urban-Existing
6 = Urban-Increased Amber
7 = Urban-Markings
8 = Clay Ave.-Existing
9 = Clay Ave. Signs
0 = Urban-Controlled

Vehicle Type Code:
1 = Passenger Car
2 = Bus
3 = Motorcycle
4 = Truck
5 = Pick-up truck

Movement Code:
1 = Straight thru
2 = Left turn
3 = right turn
4 = changes lane
5 = U-Turn
6 = Stops, will go straight
7 = Stops, will make left turn
8 = Stops, will make right turn
9 = Queueing

Extraordinary Movement Code:
1 = Accident
2 = Pedestrian interference
3 = Skid
4 = Nothing
5 = Ambulance, police, fire
6 = Violations

1 (21–25)	2 (26–30)	3 (31–35)	4 (36–40)	5 (41–45)	6 (46–50)	7 (51–55)	8 (56–60)	9 (61–65)	10 (66–70)	11 (71–75)	12 (76–80)
12229	22204	22888	22156	22888	22115	22099	22083	22070	22888	32050	32040

CARD III

Ident. No. 1 2 3 4	13 (5–9)	14 (10–14)	15 (15–19)	16 (20–24)	17 (25–29)	18 (30–34)	19 (35–39)	20 (40–44)	21 (45–49)	22 (50)
9052	32034	32028								

Where in each frame:

Column 1 - Signal Code:
1 = green
2 = amber
3 = red

Column 2 - Lane Code:
1 = lane nearest curb
2 = lane second nearest curb
3 = lane third nearest curb

Columns 3-5 — Location:
Distance from intersection in feet

Figure 13. Format of film analysis—sample.

available on the car, a third card would contain up to a maximum of 24 points. The exact data contained on the data sheets and data cards are as follows:

First Card

Columns 1-4: Identification number. The first digit of this number is composed of the right digit of the reel number on which the car movement is recorded. The middle two digits contain the last two digits of the catch number. The last digit consists of the vehicle number.

Second Card

Columns 1-4: Identification number. Same as for the first card.

Column 5: Location and control number. The coding for this number is as follows:

1 Rural location, existing condition
2 Rural location, with increased amber phase length
3 Rural location, with markings
4 Rural location, controlled experiment
5 Urban location, existing condition
6 Urban location, with increased amber phase length
7 Urban location, with markings
8 Clay Avenue location, existing condition
9 Clay Avenue location, with markings
0 Urban location, controlled experiment

Columns 6-8: Catch number. The catch number is assigned consecutively, starting with 001 for the first catch at the beginning of a reel of film. A catch exists when, at the time the light turns amber, a vehicle is located within the grid as determined by the traffic cones.

Columns 9-10: Vehicle number. Vehicle numbers are assigned consecutively within a catch, starting with 01 for the vehicle located closest to the intersection.

Column 11: The vehicle type code. This code was assigned as follows:

1 Passenger car
2 Bus
3 Motorcycle
4 Truck
5 Pickup truck

Column 12: Movement code. Code assignment:

1 Straight
2 Left turn
3 Right turn
4 Changes lane
5 U-Turn
6 Stops, will go straight
7 Stops, will make left turn
8 Stops, will make right turn
9 Queueing

Column 13: This column contains the extraordinary movement code. The following extraordinary movements were considered and coded:

1 Accident
2 Pedestrian interference
3 Skid
4 Nothing
5 Ambulance interference
6 Violation

Columns 14-18: The frame number of the first amber frame for this particular vehicle. In all cases the projector frame counter was set to the number 11,111 at the first amber frame of the first catch of vehicles.

Columns 19-20: Contain the number of frames, starting with the first red frame, during which the intersection was clear of cars interfering with the direction of traffic under study. In case no traffic crossed, the number 99 was recorded.

Columns 21-80: These 60 columns were filled with 12 five-digit numbers. Within the five-digit number the coding was as follows:

Column 1: Signal code. This code was assigned:

1 Green
2 Amber
3 Red

Column 2: Lane code. Coding was assigned as follows:

1 Lane nearest the curb
2 Lane second nearest curb
3 Lane third nearest curb

Columns 3-5: The distance that the car was located from the stop line, as determined from the trace and the master. The distance is given in feet. In case this distance could not be determined due to an obstruction, the number 888 was recorded instead.

On this second card a maximum of 12 frames could be recorded. In case this was not sufficient, a third card was used.

Third Card

Columns 1-4: Identification number. Same as for first card.

Columns 5-80: These columns contained five digit numbers coded in the same manner as was done for the second card in the last 60 columns.

Data Reduction

The data contained on the coding sheets were punched on IBM cards. The processing of the data cards was accomplished in the following steps:

1. Preliminary checking of punched cards: a computer program was prepared to check distances and signal codes. This procedure eliminated most of the punching errors.

2. Polynomial curve fit: curve fits of different degrees were applied to the raw data, the best of which was found to be the fourth degree polynomial. This step was necessary to infer unknown data points as well as to smooth the time-distance trace of vehicles.

3. Summary of results: the output of the fourth degree polynomial curve fit was used as an input for the summary program. The output of this program included: (i) identification number defining a vehicle in one of the 10 cells studied; (ii) number of frames used to trace the vehicle's position; (iii) movement code, acceptance or rejection; (iv) distance and speed at the beginning of the amber phase; (v) distance and speed at the last recorded frame of amber phase; (vi) maximum speed; (vii) maximum acceleration; (viii) maximum deceleration; (ix) extraordinary movement code; and (x) lane change and entry. Speed (in ft/sec) is obtained by doubling the distance traveled during each ½ sec, divided by 1.467. Speed change (in ft/sec^2) is found by doubling the difference between speeds of two consecutive half seconds.

4. Final proofing: this step was necessary to correct any errors not screened by the first two steps. Indications for such errors were obtained mostly from excessive or unrealistic acceleration or deceleration rates displayed by the summary program. The previous steps were repeated for such errors.

Among the inaccuracies involved were those due to equipment limitations, human difficulties in reading a vehicle's position from the film and the texture of the films used.

PILOT STUDY RESULTS

The results from the pilot study are presented in five parts. The number of vehicles observed in each cell is shown on the design of experiment (Fig. 14).

Effect of Increased Amber Phase at Urban Location

The measurements obtained for the urban location using the standard length of amber phase of 3 sec (cell one) were compared with the measurements obtained for the

LOCATION / CONTROLLED CONDITIONS	URBAN-SAN PABLO AT VALE		RURAL-RT. 4 AT SUMMERVILLE		JUNIPERO SERRA BLVD AT CLAY AVENUE
	NORMAL CONDITIONS	CONTROLLED CONDITIONS	NORMAL CONDITIONS	CONTROLLED CONDITIONS	NORMAL CONDITIONS
Clearance Interval Existing State Practice	1 108/112/220	3 63/51/114	5 25/19/44	7 52/78/130	9 45/46/91
Clearance Interval Increased Duration	2 71/103/174		6 32/19/51		
Supplemental Advanced Signing					10 20/78/98
Supplemental Pavement Markings		4 33/44/77		8 64/56/120	

Figure 14. Number of observations for cells studied (108/112/220 = accepting vehicles/rejecting vehicles/total vehicles).

same urban location using a longer amber period of 5 sec (cell two) in order to ascertain the effect of increased amber phase duration. Two sets of analyses were undertaken: (a) acceptance-rejection characteristics, and (b) risk measurements.

The acceptance-rejection characteristics are shown in Figure 15 for cell one (existing amber duration) and in Figure 16 for cell two (increased amber duration). A solid dot symbol is used to denote vehicles passing through the intersection on the amber (accepting) and an X symbol is used to denote vehicles stopping (rejecting). The solid curves denote the minimum distances for the different approach speeds that vehicles may safely stop before entering the intersection, assuming a maximum deceleration rate of 10 ft/sec^2. Therefore, any X symbol to the left of the solid curve indicates a vehicle that will exceed an average deceleration rate of 10 ft/sec^2 if the vehicle is to halt at the stop line. The dashed curves denote the maximum distance for the different approach speeds that vehicles may enter the intersection before the end of the amber phase. These dashed curves are based on indicated amber phase durations and for an acceleration rate of 5 ft/sec^2, which was

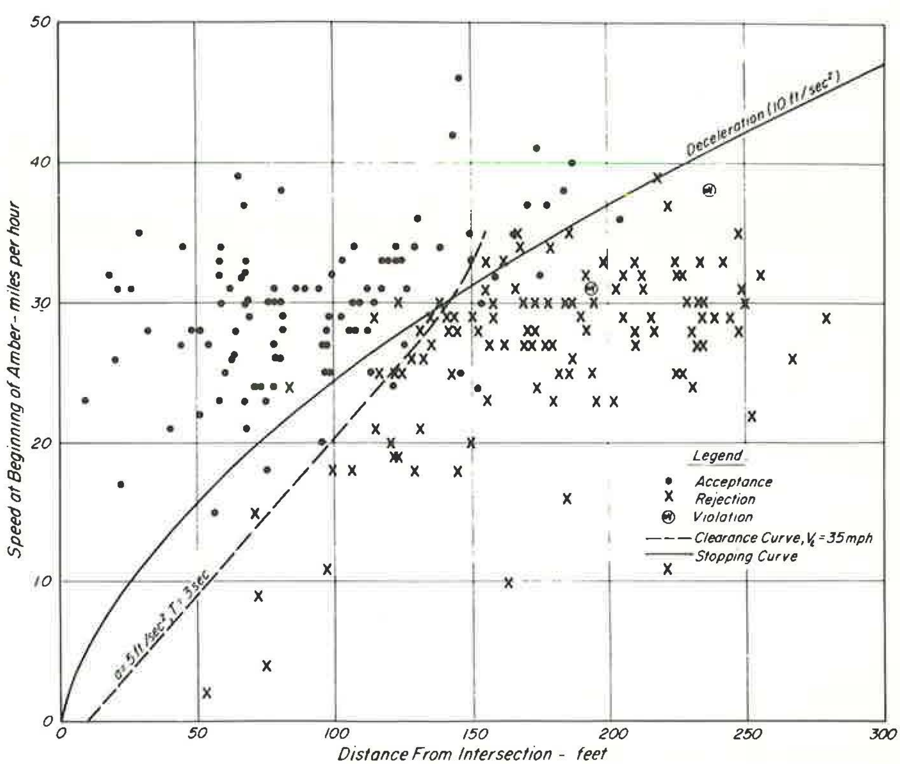


Figure 15. Cell one acceptance-rejection characteristics.

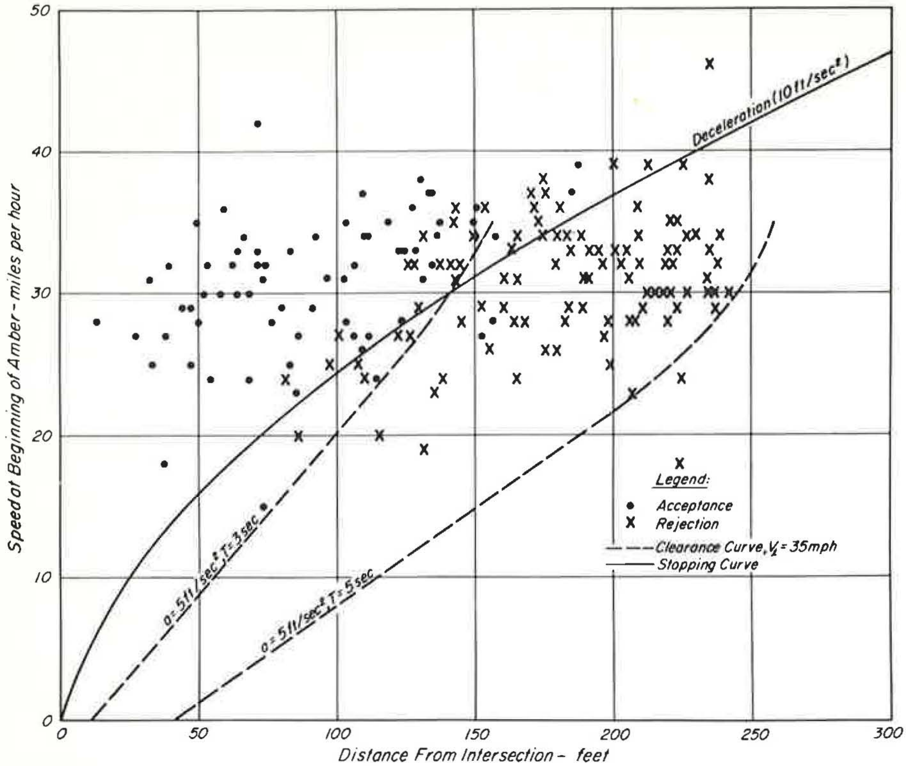


Figure 16. Cell two acceptance-rejection characteristics.

assumed to be safe and reasonable for the conditions studied. Therefore, any dot symbol to the right of the dashed curves indicates a vehicle that will exceed the allowable acceleration rate or legal speed limit or both if the vehicle is to enter the intersection before the end of the amber phase. The solid and dashed curves partition the graph into four regions: acceptance (left of solid and dashed curves); rejection (right of solid and dashed curves); option (right of solid curve and left of dashed curve); and dilemma zone (left of solid curve and right of dashed curve).

A summary of measures of safe and unsafe operations for cells one and two is given in Table 5. Generally, there was only a slight detrimental effect on safe operations due to increasing amber duration; in individual measures there were significant changes. A high percentage of rejecting vehicles was transferred from the rejection region to the option region with increased amber duration. There was a rather high percentage of rejecting vehicles in the acceptance region, but increasing amber duration eliminated the dilemma region and also the vehicles accepting in the rejection region. The most significant observation is that although increasing the amber duration changed the boundary locations between regions, and therefore the percent of vehicles observed in each region, the behavior of traffic in accepting and rejecting the amber phase at various distances from the intersection remained

TABLE 5

CELLS ONE AND TWO MEASURES OF SAFE AND UNSAFE OPERATIONS

Measures of Safe and Unsafe Operations	Cell One Existing Amber Duration (%)	Cell Two Increased Amber Duration (%)
(a) Safe or Expected Operations		
Accepting in acceptance region	39	39
Rejecting in rejection region	45	1
Option region	4	45
Total	88	85
(b) Unsafe or Unexpected Operations		
Rejecting in acceptance region	2	15
Accepting in rejection region	5	0
Dilemma region	5	0
Total	12	15

TABLE 6

RISK MEASUREMENTS OF CELLS ONE AND TWO

Risk Measurements	Cell One Existing Amber Duration (%)	Cell Two Increased Amber Duration (%)
(a) Accepting Vehicles		
Exceeded speed limit after beginning of amber	8	20
Exceeded deceleration rate of 15 ft/sec²	1	1
Exceeded acceleration rate of 8 ft/sec²	3	10
Exceeded deceleration rate of 15/sec² and acceleration rate of 8 ft/sec²	0	0
Entered intersection on red phase	2	0
Changed lanes during amber phase	0	0
Percent of accepting vehicles involved in one or more risks	13	28
(b) Rejecting Vehicles		
Exceeding speed limit after beginning of amber	0	1
Exceeded deceleration rate of 15 ft/sec²	5	12
Exceeded acceleration rate of 8 ft/sec²	3	5
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	0	0
Changed lanes during amber phase	0	0
Percent of rejecting vehicles involved in one or more risks	8	20
Percent of all vehicles involved in one or more risks	11	24

essentially unchanged. It appears that traffic is unaffected by increasing amber duration either because the drivers are unaware of the increase or the drivers are aware but not affected by it. One-fourth of the vehicles classified as performing in an unsafe or unexpected manner in cell one were exceeding the speed limit when the signal changed to amber. One-third of the vehicles classified as performing in an unsafe or unexpected manner in cell two were exceeding the speed limit when the signal changed to amber.

The solid curves denoting the deceleration rate limit for stopping divided the sets of data points in such a manner that the number of rejecting vehicles to the left of the curve is approximately equal to the number of accepting vehicles to the right. Eighty-seven percent of all observed vehicles in cells one and two were on the expected side of the deceleration curve. Fifteen percent of the rejecting vehicles were unexpectedly on the left of the deceleration curve and 11 percent of the accepting vehicles were unexpectedly on the right of the deceleration curve. The drivers observed during the shorter amber period appeared to be slightly more aggressive than drivers observed during the longer amber period.

The second set of analyses was directed toward evaluating measures of risk for the two cells in question. A summary comparing the risk measurements for cells one and two is given in Table 6. There was a higher percentage of vehicles in the various risk measurements with the increased amber duration, with the single exception of vehicles entering the intersection on the red phase. Two out of 108 accepting vehicles in cell one entered the intersection on the red phases; there were no such events with the increased amber duration. There were 2 vehicles (3%) in cell two that would have been classified as entering the intersection on the red phase if a 3-second amber had been in operation. All other risk measurements, while perhaps not as critical, gave no indication that safer operations resulted from increased amber duration. In fact, if any change was noted, the increased amber phase gave a higher percentage of risk measurements.

Effect of Increased Amber Phase at Rural Location

The measurements obtained for the rural location using the 5-sec standard length of amber phase (cell five) were compared with the measurements obtained for the same rural location using a 7-sec period (cell six) to ascertain the effect of the increase. The same two sets of analyses were undertaken.

The acceptance-rejection characteristics are shown in Figure 17 for cell five and in Figure 18 for cell six. These figures were constructed in a manner similar to Figures 15 and 16.

A summary of measures of safe and unsafe operations for cells five and six are given in Table 7. There was a higher percentage of vehicles that operated in a safe or expected manner with the increased duration. There appeared to be two specific changes: (a) by increasing the amber duration, 8 percent of the vehicles which would have been in the group marked "rejecting in rejection region" were transferred to "option region" group; (b) percentage of vehicles in the group, "rejecting in acceptance" was reduced. This specific change is attributable to the increased amber phase, but the relatively small sample size should be noted.

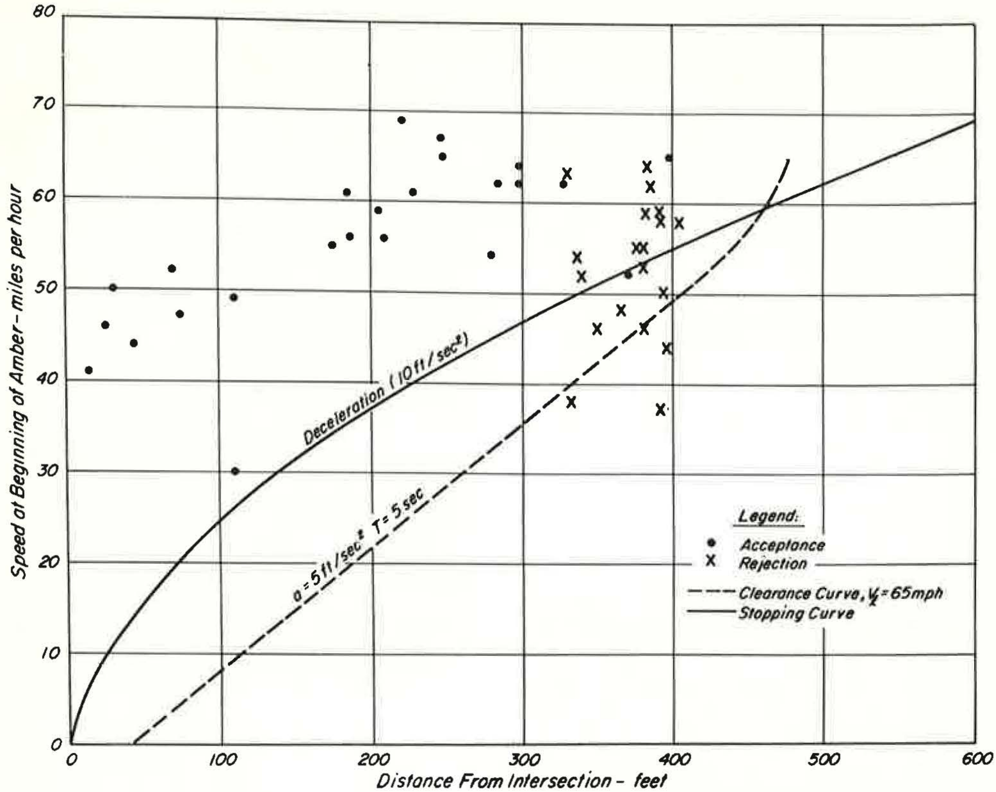


Figure 17. Cell five acceptance-rejection characteristics.

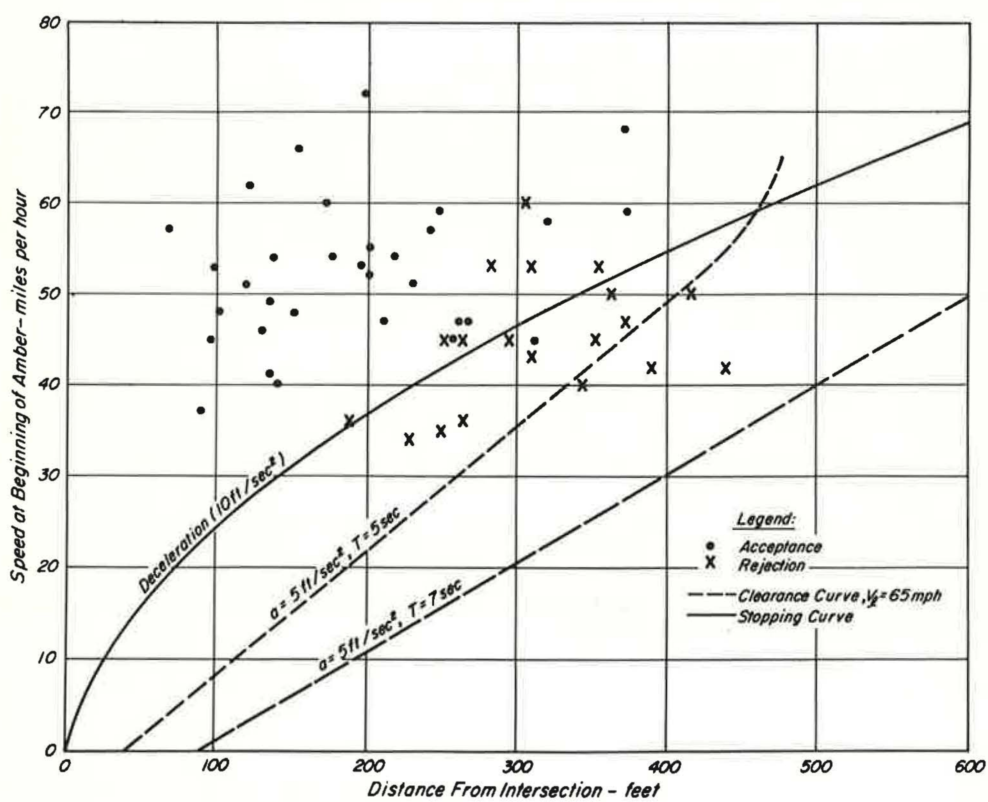


Figure 18. Cell six acceptance-rejection characteristics.

TABLE 7

CELLS FIVE AND SIX MEASURES OF SAFE AND UNSAFE OPERATIONS

Measures of Safe and Unsafe Operations	Cell Five Existing Amber Duration (%)	Cell Six Increased Amber Duration (%)
(a) Safe or Expected Operations		
Accepting in acceptance region	55	61
Rejecting in rejection region	9	0
Option region	11	25
Total	75	86
(b) Unsafe or Unexpected Operations		
Rejecting in acceptance region	25	14
Accepting in rejection region	0	0
Dilemma region	0	0
Total	25	14

TABLE 8

RISK MEASUREMENTS OF CELLS FIVE AND SIX

Risk Measurements	Cell Five Existing Amber Duration (%)	Cell Six Increased Amber Duration (%)
(a) Accepting Vehicles		
Exceeded speed limit after beginning of amber	12	0
Exceeded deceleration rate of 15 ft/sec^2	0	0
Exceeded acceleration rate of 8 ft/sec^2	52	25
Exceeded deceleration rate of 15 ft/sec^2 and acceleration rate of 8 ft/sec^2	0	0
Entered intersection on red phase	4	0
Changed lanes during amber phase	4	0
Percent of accepting vehicles involved in one or more risks	60	25
(b) Rejecting Vehicles		
Exceeded speed limit after beginning of amber	0	0
Exceeded deceleration rate of 15 ft/sec^2	5	26
Exceeded acceleration rate of 8 ft/sec^2	5	0
Exceeded deceleration rate of 15 ft/sec^2 and acceleration rate of 8 ft/sec^2	0	0
Changed lanes during amber phase	0	0
Percent of rejecting vehicles involved in one or more risks	10	26
Percent of all vehicles involved in one or more risks	39	26

The second set of analyses was directed toward evaluating measures of risk for the two cells in question. A summary comparing the risk measurements for cells five and six is given in Table 8. The percentage of accepting vehicles in cell six involved in risk was less than those in cell five. On the other hand, the percentage of rejecting vehicles in cell six involved in risk was greater than those in cell five. There was an overall decrease in the percentage of all vehicles involved in risk when the amber duration was increased.

Effect of Pavement Markings at Urban Location

The measurements obtained for the urban location using the standard length of amber phase of 3 sec without pavement markings and with controlled drivers (cell three) were compared with the measurements obtained for the same urban location, the same duration with pavement markings and with the same controlled drivers (cell four). This permitted the evaluation of the effect of transverse pavement markings on improving safe operations. Again, acceptance-rejection characteristics and risk measurements were analyzed.

The acceptance-rejection characteristics are shown in Figure 19 for cell three and in Figure 20 for cell four. These figures were constructed in a manner similar to that used previously.

Table 9 summarizes measures of safe and unsafe operations for cells three and four. There was a higher percentage of vehicles which operated in a safe or expected manner with the pavement markings than under similar conditions without the pavement markings. The major improvement was the reduced percentage of vehicles in the "rejecting in the acceptance region" group.

Table 10 summarizes the risk measurements for cells three and four. The difference in risk between the two conditions was inconsistent. Generally, the percentage of accepting vehicles involved in a risk was greater in cell four, whereas the percentage of rejecting vehicles involved in a risk was less. Overall, there was little change in risks with pavement markings present.

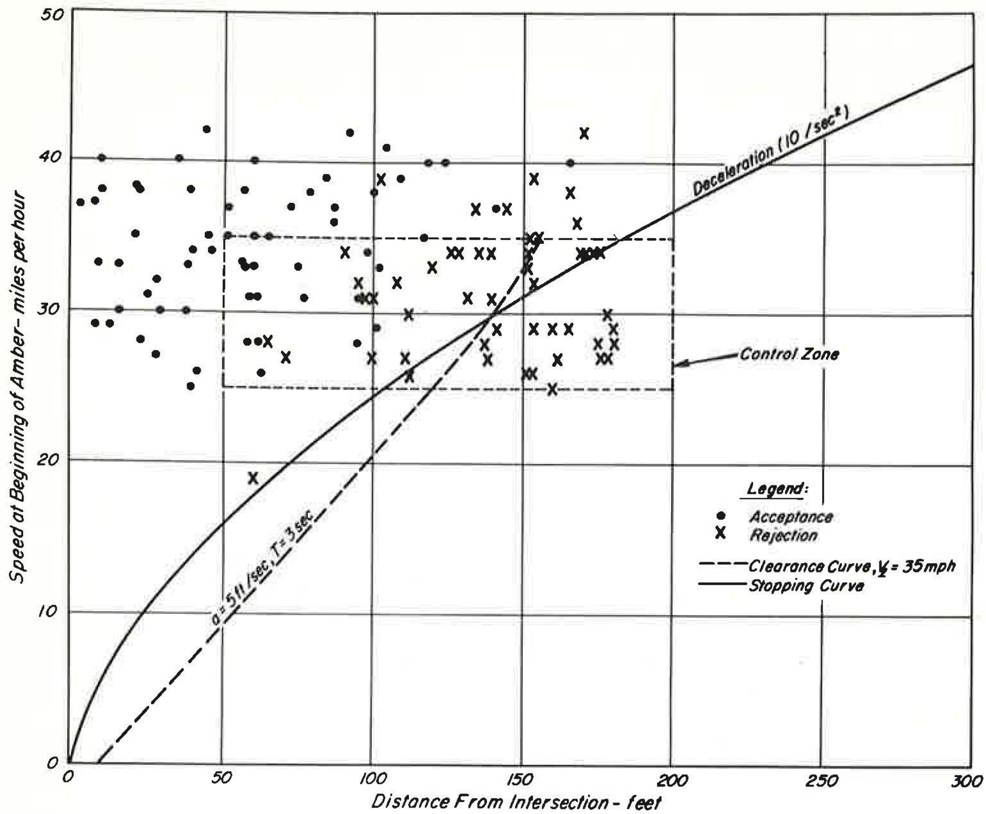


Figure 19. Cell three acceptance-rejection characteristics.

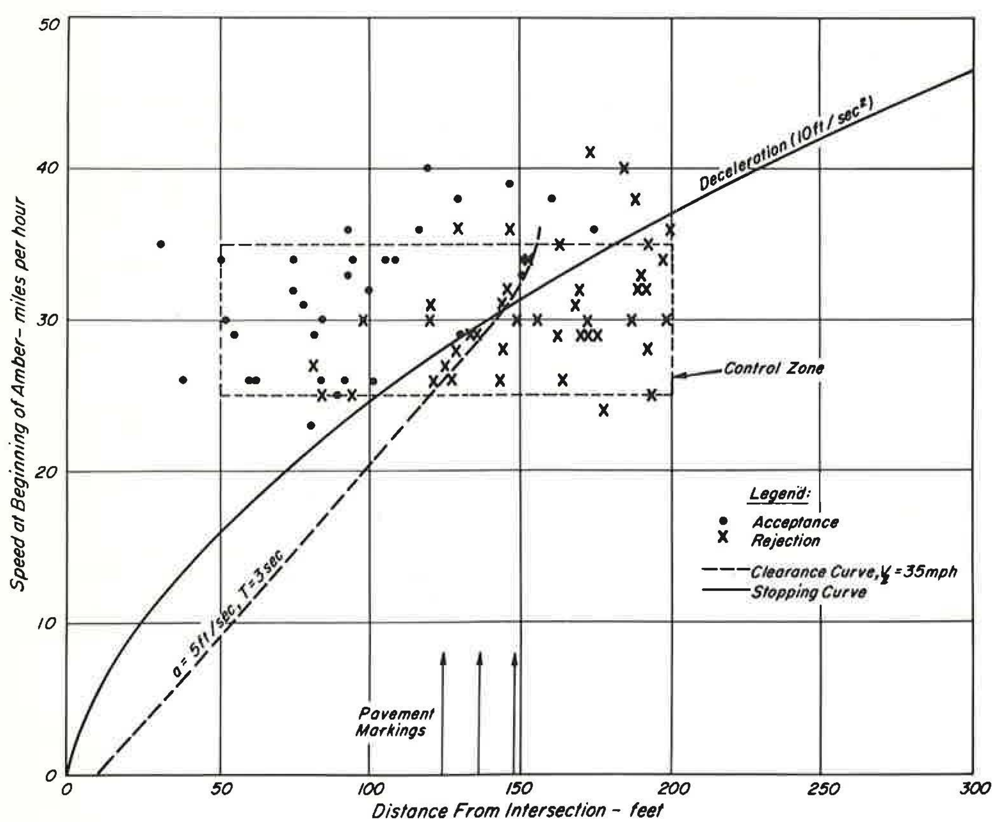


Figure 20. Cell four acceptance-rejection characteristics.

TABLE 9

CELLS THREE AND FOUR MEASURES OF SAFE AND UNSAFE OPERATIONS

Measures of Safe and Unsafe Operations	Cell Three Without Pavement Markings (%)	Cell Four With Pavement Markings (%)
(a) Safe or Expected Operations		
Accepting in acceptance region	54	40
Rejecting in rejection region	14	31
Option region	1	7
Total	69	78
(b) Unsafe or Unexpected Operations		
Rejecting in acceptance region	22	14
Accepting in rejection region	0	0
Dilemma region	9	8
Total	31	22

TABLE 10

RISK MEASUREMENTS OF CELLS THREE AND FOUR

Risk Measurements	Cell Three Without Pavement Markings (%)	Cell Four With Pavement Markings (%)
(a) Accepting Vehicles		
Exceeded speed limit after beginning of amber	5	15
Exceeded deceleration rate of 15 ft/sec²	6	0
Exceeded acceleration rate of 8 ft/sec²	21	33
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	2	0
Entered intersection on red phase	0	3
Changed lanes during amber phase	0	0
Percent of accepting vehicles involved in one or more risks	29	42
(b) Rejecting Vehicles		
Exceeded speed limit after beginning of amber	4	0
Exceeded deceleration rate of 15 ft/sec²	16	11
Exceeded acceleration rate of 8 ft/sec²	8	4
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	2	0
Changed lanes during amber phase	0	0
Percent of rejecting vehicles involved in one or more risks	24	15
Percent of all vehicles involved in one or more risks	26	27

Effect of Pavement Markings at Rural Location

The measurements obtained for the rural location using the standard length of amber phase of 5 sec without pavement markings and with controlled drivers (cell seven) were compared with the measurements obtained for the same rural location using the same standard length with pavement markings and with the same controlled drivers (cell eight). This permitted the evaluation of the effect of transverse pavement markings on improving safe operations.

The acceptance-rejection characteristics are shown in Figure 21 for cell seven and in Figure 22 for cell eight.

Table 11 summarizes measures of safe and unsafe operations for cells seven and eight. There was a slightly lower percentage of vehicles that operated in a safe or expected manner with the pavement markings than under similar conditions without pavement markings. The major individual change was the increase of the percentage of vehicles in the group "rejecting in acceptance region" and the decrease of the percentage of vehicles in the group "option region." Overall, there was a slight detrimental effect of pavement markings on safe operations.

Table 12 summarizes the risk measurements for cells seven and eight. The pavement markings had an adverse effect on rejecting vehicles and a slight adverse effect on accepting vehicles. Generally, the experiments evaluating the effect of pavement markings on safe operations indicated that safe operations did not improve with pavement markings at the rural location.

TABLE 11

CELLS SEVEN AND EIGHT MEASURES OF SAFE AND UNSAFE OPERATIONS

Measures of Safe and Unsafe Operations	Cell Seven Without Pavement Markings (%)	Cell Eight With Pavement Markings (%)
(a) Safe or Expected Operations		
Accepting in acceptance region	40	54
Rejecting in rejection region	14	3
Option region	25	15
Total	79	72
(b) Unsafe or Unexpected Operations		
Rejecting in acceptance region	21	27
Accepting in rejection region	0	0
Dilemma region	0	0
Total	21	28

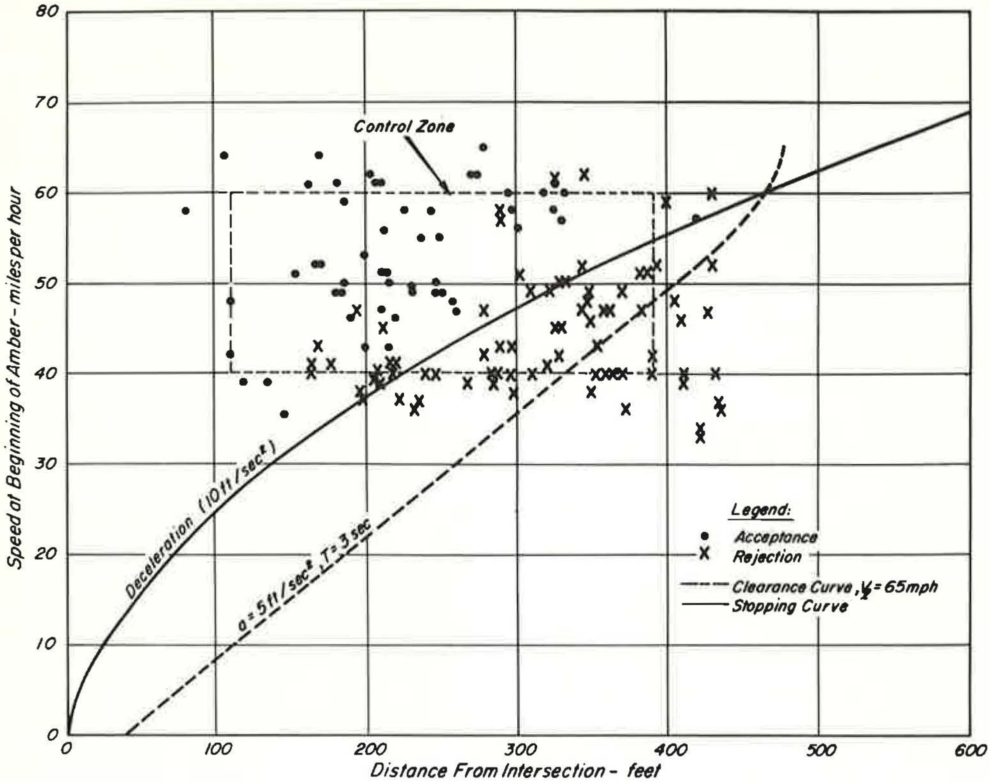


Figure 21. Cell seven acceptance-rejection characteristics.

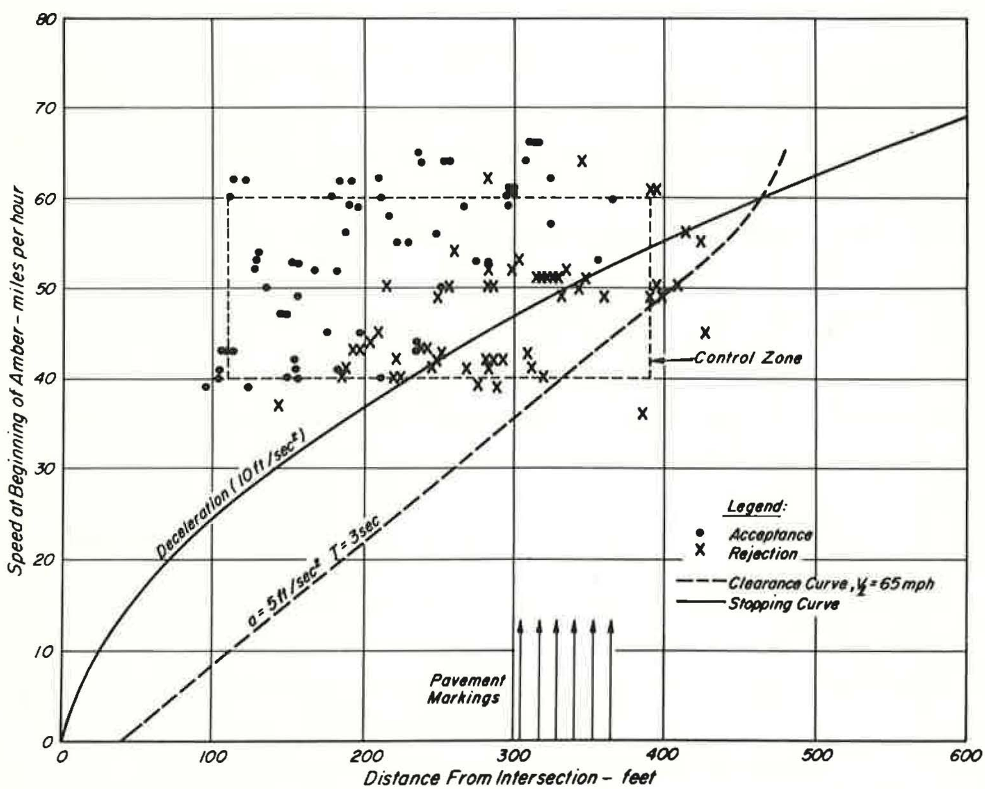


Figure 22. Cell eight acceptance-rejection characteristics.

TABLE 12

RISK MEASUREMENTS OF CELLS SEVEN AND EIGHT

Risk Measurements	Cell Seven Without Pavement Markings (%)	Cell Eight With Pavement Markings (%)
(a) Accepting Vehicles		
Exceeded speed limit after beginning of amber	2	5
Exceeded deceleration rate of 15 ft/sec²	0	2
Exceeded acceleration rate of 8 ft/sec²	38	36
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	0	2
Entered intersection on red phase	0	2
Changed lanes during amber phase	0	2
Percent of accepting vehicles involved in one or more risks	38	41
(b) Rejecting Vehicles		
Exceeded speed limit after beginning of amber	0	0
Exceeded deceleration rate of 15 ft/sec²	10	20
Exceeded acceleration rate of 8 ft/sec²	13	18
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	1	2
Changed lanes during amber phase	0	0
Percent of rejecting vehicles involved in one or more risks	22	36
Percent of all vehicles involved in one or more risks	28	38

Effect of Supplemental Advanced Signing

The measurements obtained at the Junipero Serra Boulevard location without the "prepare to stop" supplemental advanced signing (cell nine) were compared with the measurements obtained for the same location with the "prepare to stop" supplemental advanced signing (cell ten). This permitted the evaluation of the effect of supplemental advanced signing on improving safe operations.

The acceptance-rejection characteristics are shown in Figure 23 for cell nine and in Figure 24 for cell ten.

Table 13 summarizes measures of safe and unsafe operations for cells nine and ten. There was a higher percentage of vehicles operating in a safe or expected manner with the supplemental advanced signing than without signing. The most significant change that the "prepare to stop" signal seemed to have affected was a reduction in the percentage of vehicles

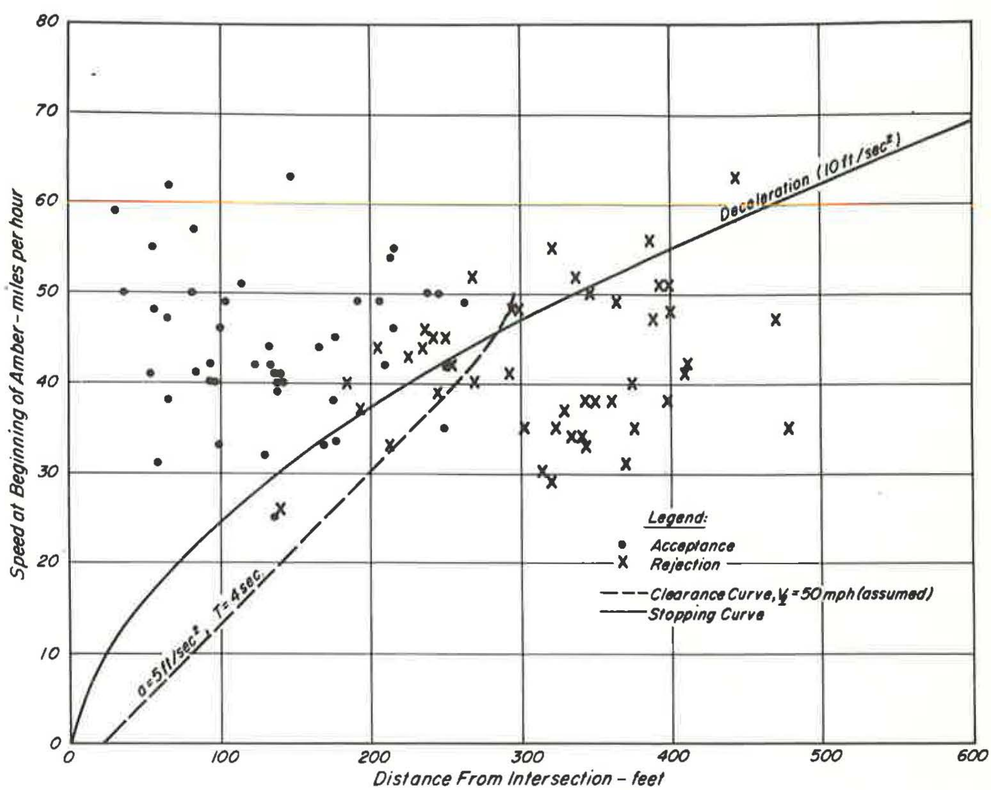


Figure 23. Cell nine acceptance-rejection characteristics.

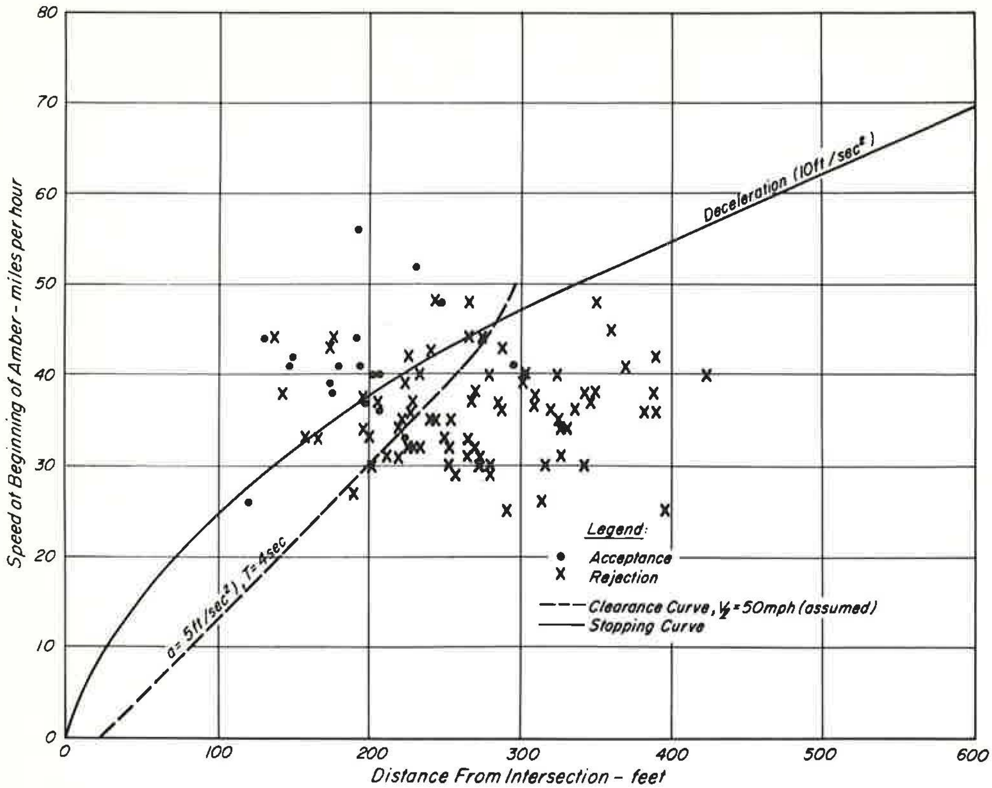


Figure 24. Cell ten acceptance-rejection characteristics.

caught in the dilemma zone from 7 percent (cell nine) to 0 percent (cell ten). This was accomplished by a marked reduction in the approach speed ceiling of the speed-distance plots (Figs. 23 and 24), which in effect had transferred to the option zone the percentage of vehicles that otherwise would have been caught in the dilemma zone (Table 13). Overall, the supplemental advanced signing resulted in the improvement of safe operations.

Table 14 summarizes the risk measurements for cells nine and ten. The values of risk measurements for the accepting and rejecting vehicles were augmented by high percentages of vehicles exceeding the indicated acceleration and deceleration rates. This was due primarily to the difficulty encountered in reading the exact positions of vehicles from the film because of the inherent peculiarities of the site in question. These values, however, were obtained under the same site conditions and same procedural method of analysis, and thus were consistent in relation to each other.

The results of cells nine and ten were then compared to determine the effect of supplemental advanced signing on safe operations. With such signing the percentages of risk measurements were less, for both accepting and rejecting vehicles, than those without signing. Overall, the

TABLE 13

CELLS NINE AND TEN MEASURES OF SAFE AND UNSAFE OPERATIONS

Measures of Safe and Unsafe Operations	Cell Nine Without Advanced Signing (%)	Cell Ten With Advanced Signing (%)
(a) Safe or Expected Operations		
Accepting in acceptance region	43	14
Rejecting in rejection region	30	57
Option region	9	18
Total	82	89
(b) Unsafe or Unexpected Operations		
Rejecting in acceptance region	10	9
Accepting in rejection region	1	2
Dilemma region	7	0
Total	18	11

TABLE 14

RISK MEASUREMENTS OF CELLS NINE AND TEN

Risk Measurements	Cell Nine Without Advanced Signing (%)	Cell Ten With Advanced Signing (%)
(a) Accepting Vehicles		
Exceeded speed limit after beginning of amber	35	10
Exceeded deceleration rate of 15 ft/sec²	49	20
Exceeded acceleration rate of 8 ft/sec²	58	45
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	29	15
Entered intersection on red phase	4	10
Changed lanes during amber phase	2	0
Percent of accepting vehicles involved in one or more risks	89	55
(b) Rejecting Vehicles		
Exceeded speed limit after beginning of amber	0	1
Exceeded deceleration rate of 15 ft/sec²	35	26
Exceeded acceleration rate of 8 ft/sec²	28	34
Exceeded deceleration rate of 15 ft/sec² and acceleration rate of 8 ft/sec²	9	6
Changed lanes during amber phase	0	0
Percent of rejecting vehicles involved in one or more risks	60	54
Percent of all vehicles involved in one or more risks	74	54

effect of supplemental advanced signing was to improve safe operations at the location studied. This inference was consistent with that obtained from the acceptance-rejection plots.

SUMMARY

It should be emphasized that the purpose of this pilot investigation was to identify promising modifications of amber period duration, transverse pavement markings, and supplemental advanced signing which gave evidence of improvements of safe operations at signalized intersections. The purpose was not to provide conclusive evidence for the modifications studied. A comprehensive summary tabulation of the two sets of risk measurements for each of the ten cells of data is shown in Figure 25.

1. An extensive search of the literature was undertaken and some 76 references were studied in detail. Although much has been written on this subject, specific means for further improving safe operations at signalized intersections have not been thoroughly tested and validated.

2. Fifteen states have laws similar to California pertaining to the behavior of motorists with regard to the amber phase. The California law is in agreement with the recently revised Manual on Uniform Traffic Control Devices. The current practice in California with regard to amber duration is similar to other states with similar laws. Although many states recognize this aspect of signal operations as a safety and/or capacity problem, only a few states have research studies under way.

3. The theoretical analyses have shown that the type of law pertaining to the behavior of motorists with regard to the amber phase has a significant influence on traffic engineering practices that provide safe operations. The current California practice is in keeping with the current California law and either eliminates or greatly minimizes the possibility of a dilemma zone. Equations have been developed to calculate the minimum amber time in order to eliminate the dilemma zone and provide safe operations.

4. Increasing the amber phase at the urban location from 3 to 5 sec increased the percentage of motorists operating in an unsafe or unexpected manner.

5. Increasing the amber phase at the rural location from 5 to 7 sec decreased the percentage of motorists operating in an unsafe or unexpected manner.

6. The installation of experimental transverse pavement markings at the urban location slightly

CONTROLLED CONDITIONS \ LOCATION	URBAN-SAN PABLO AT VALE		RURAL-RT. 4 AT SUMMERVILLE		[illegible]
	NORMAL CONDITIONS	CONTROLLED CONDITIONS	NORMAL CONDITIONS	CONTROLLED CONDITIONS	NORMAL CONDITIONS
Clearance Interval Existing State Practice	1 12/11	3 31/26	5 25/39	7 21/28	9 18/74
Clearance Interval Increased Duration	2 15/24		6 14/26		
Supplemental Advanced Signing					10 11/54
Supplemental Pavement Markings		4 22/27		8 28/38	

Figure 25. Summary of two sets of risk measurements (12/11 = % unsafe operations, acceptance-rejection/% risk measurements).

decreased the percentages of motorists operating in an unsafe or unexpected manner.

7. The installation of experimental transverse pavement markings at the rural location increased the percentage of motorists operating in an unsafe or unexpected manner.

8. The installation of supplemental advance signing location decreased the percentage of motorists operating in an unsafe or unexpected manner.

9. Future studies extending this work and directed toward providing conclusive evidence for modification improvements will require more accurate measurements than were obtained in this pilot study. Greater photographic detail, perhaps from the air, coupled with more frequent exposures per unit of time will be necessary.

ACKNOWLEDGMENTS

The Institute would like to acknowledge the excellent cooperation received from personnel of the California Division of Highways. Particular recognition is given to C.E. Wong and J. B. Elzey, Assistant Traffic Engineers in District 4, who participated in the controlled field experiments at the rural and urban locations. Appreciation is expressed to the City of San Pablo for permission to modify the traffic signal operations at the urban location during periods of field study. R. G. Newcomb was responsible for the photographic aspects of the field studies. G. B. Dierking prepared all illustrations contained in this report. The author would also like to recognize the graduate and undergraduate students who participated in the various aspects of this study: Ali Ardakanian, Robert Backman, Robert Bernard, Robert Hom, Robert Johnson, Paul Kotani, Paul Macy, Jan Roggeveen, Michael Rucker, Yosef Salam, Richard Sanders, Joseph Shaw, Robert Waldeck, and Wayne Ybarra.

This research was performed in cooperation with the California Division of Highways and the Bureau of Public Roads.

An Analysis of Flashing Systems

THOMAS J. FOODY and WILLIAM C. TAYLOR, Ohio Department of Highways

This investigation evaluated the effectiveness of various types of flashing devices in reducing the total accident rate at intersections on the rural state highway system. Several other variables were considered while the relationship between type of flashing device and reduction in accident rate was being formulated. These variables were flasher conspicuousness, intersection geometrics, volume, and line-of-sight distance.

The results of this study indicated that the reduction in intersection accident rate was optimized through the use of a particular type of flashing device in combination with certain intersection geometrics. It was also determined that the variables of intersection geometrics and total intersection volume place limiting constraints on this relationship.

•USE of traffic control devices to enhance the convenience and safety of the driving public has long been accepted practice. Without such controls, the accident toll would certainly be much higher than the present figure. However, indiscriminate use of such control devices may detract from their effectiveness, or at least, reduce their efficiency. Maximum safety can only be realized if the use of such devices is properly controlled, therefore, there must be warrants or criteria that must be applied prior to installation.

The effectiveness of any control device will depend primarily on the selection of the warrants for its use. Engineering judgment has played a major role in the selection process in the past and probably will continue to do so. When possible, this judgment should be corroborated with data collected at locations controlled by such devices. These data may be direct measurements of the desired effect or meausrements of parameters for which a relationship to the desired effect is known.

There are two methods of establishing installation warrants based on collected data. The first is to model a situation, either theoretically or physically, and derive from the results situations that can be corrected through the application of a control device. The second method is to derive empirical relationships between safety and the control device, and use these relationships to establish warrants. The traffic engineering profession has utilized the second method almost exclusively.

The warrants for traffic signals and speed zones are two examples of this procedure. There are 6 items in the warrants for traffic signals:

1. Minimum vehicular volume,
2. Interruption of continuous traffic,
3. Minimum pedestrian volume,
4. Progressive movement,
5. Accident experience, and
6. Combinations of warrants.

The inclusion of 4 of these items is based on past experience or empirical data showing that either delay or safety can be improved by the addition of a traffic signal when the conditions reach the stated level. The remaining items are added to permit the installation of signals where the maximum level of the previous items, established by

Paper sponsored by Committee on Traffic Control Devices.

empirical relationships, has not yet been reached, but the accident history indicates the existence of a problem.

The warrants for speed zoning contain 10 items. The same analysis can be made showing that 5 are related empirically and that 5 are measures of a specific problem. The proper combinations of these measures are used as evidence that a problem exists and that this problem can be alleviated by the application of speed controls. The warrants for speed zoning are as follows:

1. Apparent design speed,
2. Length of proposed zone,
3. Number of minor public highways and private access roads,
4. Number of state routes and through county or township highways,
5. Number of roadside businesses,
6. Both limits of pace between stated speeds,
7. 85 percentile between stated speeds,
8. Total accidents,
9. Driveway or intersection accidents, and
10. Average test runs, speed.

Unfortunately, there is no such set of warrants for the installation of flashing devices. This is partially because empirical relationships between flashers and safety have not been established, and partially because these devices are used primarily as interim treatments between stop sign control and signal control. When there is evidence that a safety problem exists at an intersection but the problem has not reached the necessary level to warrant a traffic signal, a flashing device is often used.

This research concerned the problem of recommending warrants for the installation of flashing traffic control devices. The procedure is based on empirical analyses designed to evaluate the effect of previous installations.

The analyses involved 82 intersections in Ohio, each of which was controlled by a flashing device. The data were categorized by (a) accident history, (b) intersection geometrics, (c) sight distance, (d) traffic volumes, and (e) flasher type. Statistical tests were conducted to determine relationships between accident reduction and the other measures. These relationships, where significant, were used to recommend installation warrants.

ACCIDENT ANALYSIS

The purpose of the accident analysis was to determine the relationship existing between accident reduction and the application of a flashing device. Factors influencing this relationship were considered: type of flashing device, geometrics of the intersection, and traffic volume characteristics. The quantitative relationships thus developed were used in the formation of recommendations governing the installation of flashing devices.

Data Collection

When an accident problem exists at an intersection in Ohio, an engineering study is conducted. The results may indicate the need for one of many changes ranging from the installation of a traffic-control device to complete reconstruction. A record file is maintained on all such intersections. This file includes records of all signal installations on the rural state highway system. From these records, all (200) intersections where flashing device systems were first installed between 1955 and 1965 were chosen for study.

Each intersection was studied, and field observations on the following were recorded: (a) description of the flashing device, (b) description of environmental features, (c) intersection geometrics, and (d) line-of-sight distance from the minor road along the main road approach.

A sketch of the intersection showing the location and description of the physical features and including a description of the flashing device, location, type of flashing unit, and flash rate, was made. It also showed any environmental features such as business

establishments, topography, and trees that might obstruct the view of the signals. The intersection geometric features of lane width, number of lanes, and angle of approach were recorded.

The line-of-sight distance was recorded as the distance on each approach of the main road visible from the minor road (minor road approach refers to "stop" approach; main road approach refers to "thru" approach). Thus, the sight-distance determination was based on the geometric limitations of the intersection rather than on the environmental features.

The next step was to compare the field observations with records kept in the intersection file in order to verify that no changes in the flashing device or intersection geometrics described by the field observer had occurred since the flashing device was originally installed. As a result of this check, only 90 of the 200 intersections had a valid before and after period during which the only alteration was the installation of a flashing device.

The accident history of each intersection was compiled for 2 yr before and 2 yr after the installation of the flashing device. Complete accident records were available for only 82 intersections; therefore, the results of the study were based on this sample size.

Data Reduction

Accident data for the 82 intersections were grouped into the following categories for analysis: (a) light conditions — day and night; (b) accident type — angle, rear end, and all others; and (c) total accidents — all accidents. The before and after periods in the analysis were determined by the actual installation date of the flashing device system.

Traffic counts are taken on all Ohio state highways every four years. Less frequent counts are available for the county road system. Linear interpolation techniques were used to establish the ADT values for each leg of the intersection. Separate values were determined for the before and after periods.

The ADT figures represent the total traffic (bidirectional) between two adjacent access points. The ADT for the two opposing legs of an intersection (east-west or north-south) were added together and halved; the result represented a more realistic estimate of the average intersection approach ADT for the 2-yr time period under study. This procedure was used to obtain ADT figures for the main road approach and the minor road approach for both study periods.

The volume characteristics of the intersection were used in the following manner to obtain accident rates for the accident analyses:

$$a_p = \frac{A_p}{k\,(V_1 + V_2)_p}$$

where

A = total number of accidents,
p = 2-yr time period,
V_1 = average ADT for main approach,
V_2 = average ADT for minor approach,

$k = \frac{730 \text{ days}}{1,000,000 \text{ vehicles}} = 730 \times 10^{-6}$, and

a_p = average intersection accident rate for time period under study.

Using this formula, before and after period accident rates for each of the 82 intersections were calculated. Each accident rate represents the average total number of accidents per million vehicles passing through the intersection during each study period.

Description of Variables

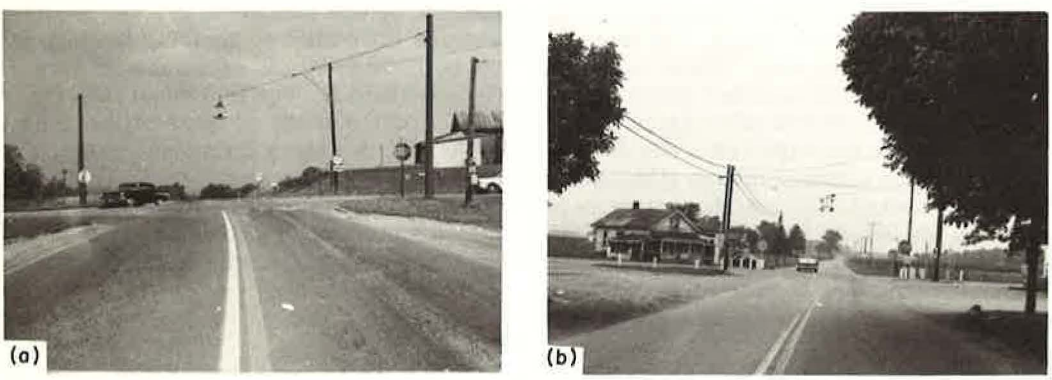
The dependent variable used as the measure of effectiveness was the total accident rate. This was considered the primary dependent variable and was used in the initial analyses. Separate records were maintained on day-night accidents and the various accident types. These two techniques of accident classification were utilized for special analyses. The independent variables were flasher type, intersection geometrics, and line-of-sight. Five distinct flasher types were available for study:

1. Type I. Standard stop sign on the side of the road with one or two flashing beacons attached to the support post. Intersections in this group had this type of control on the right-hand side or on both sides of all minor leg approaches to the intersection.
2. Type II. Overhead rectangular stop sign with two or four flashing beacons. This form of control appeared on all minor road approaches to the intersections contained in this group.
3. Type III. A single unit placed overhead in the center of the minor approach roadway and displaying two beacons flashing alternately. These beacons were arranged vertically in the majority of samples, and horizontally in the rest. The physical separation of the two beacons was approximately one foot.
4. Type IV. Two units placed overhead, each centered over an approach lane on the minor road. Each unit consisted of two beacons separated vertically and flashing alternately.
5. Type V. Two units placed overhead, each centered over a lane on the minor road. Each unit consisted of one beacon and the two units flashed simultaneously.

The five types are shown in Figures 1 through 5.

Figure 1. Type I flashing device.

Figure 2. Type II flashing device.



(a) (b)

Figure 3. Type III flashing device with individual lens (a) separated vertically, and (b) separated horizontally.

Figure 4. Type IV flashing device.

Figure 5. Type V flashing device.

The geometric categories used in the study were as follows:

1. Type I. T-intersection in which the minor approach was a two-lane roadway.
2. Type II. Four-leg intersection in which both intersection roadways were two-way facilities.
3. Type III. Four-leg intersection in which the minor road is a two-lane facility, but the major road is a multilane facility.

The line-of-sight parameter was described by three subgroups. They were:

1. Group I. Minimum line-of-sight onto either major approach less than or equal to 0.1 miles.
2. Group II. Minimum line-of-sight onto either major approach greater than 0.1, but less than or equal to 0.2 miles.
3. Group III. Minimum line-of-sight onto either major approach greater than 0.2 miles.

Analyses

The analyses proceeded from the generalized condition to specific cases. That is, flashing devices as a whole were analyzed first, and then the various classifications within the total were checked. Subclassification was continued as long as the results were statistically significant.

The first step was to determine if any significant change in the total accident rate was experienced at the 82 intersections following the installation of the flashing devices. The test chosen for this determination was the "t" statistic, the distribution of which is designed for the detection of significant differences between means.

The usual method of conducting an analysis using the t-test would be to compare the difference between the mean accident rates before and after installation of the flashing devices. This technique is applicable only when the before sample is independent of the after sample. In this study, the same intersections were used in deriving the before and after accident rates. Thus, instead of two samples of 82 intersections each, a single sample of 82 matched pairs was actually considered. The individual difference between the before and after rates was computed, and the mean of these differences compared with the expected mean difference. Testing the mean difference rather than the difference between means eliminated dependence within the pairs.

The t-statistic for matched pairs is based on a relationship of the mean difference between the before and after accident rates, the expected mean difference, and an estimate of the standard error of the mean difference. This latter term is a function of the sample variance. The form of t is

$$\frac{M_D - E(M_D)}{\text{est } \sigma_{M_D}}$$

For this analysis the expected mean difference, $E(M_D)$, was zero.

A two-tailed test was used to determine the statistical significance of the mean difference. The chosen level of significance was 0.05. The two-tailed test places one-half of the rejection region in each tail of the t-distribution. In this way, both positive and negative mean differences were detected as significant.

ACCIDENT CORRELATIONS

The mean difference in accident rate after installation of flashing devices at all 82 intersections is 1.31 accidents per million vehicles through the intersection. The value of the sample variance is 3.47 and the estimate of the standard error of the difference is 0.21. The t-value found by substitution into the formula is

$$t = \frac{1.31 - 0}{0.21} = 6.34$$

The theoretical t-value at the 0.05 level is 2.00. The sample value greatly exceeds the theoretical value, indicating that this mean difference of 1.31 has a probability of much less than 5 in 100 of occurring by chance alone.

This significant result implies that it is safe to say that some association exists between the installation of a flashing device and a reduction in accident rate. It does not indicate the strength of this association, nor does it enable prediction of the effect of a flashing device on the accident rate.

It is possible to estimate the amount of statistical association implied by any mean difference obtained. Given the value of the mean difference, there will be a variance associated with that mean which will indicate the uncertainty of the value. Specification of some independent variable, such as type of flashing device, may reduce the variance or uncertainty of the dependent variable. This reduction can be expressed as a relative reduction by dividing it by the original total variance.

This relative reduction in uncertainty can be shown by the index ω^2. Viewed either as a relative reduction in uncertainty, or as a proportion of variance accounted for, the index ω^2 represents the strength of association between independent variables. This index ω^2 is almost identical with the correlation ratio. It reflects the predictive usefulness afforded by a relationship. Its values may range from zero to unity, with a value of 1.00 indicating that the dependent variable may be predicted exactly by specifying the independent variable.

The results of the t-test for significance may be used to obtain a rough estimate of ω^2:

$$\text{est } \omega^2 = \frac{t^2 - 1}{t^2 + N - 1}$$

Inserting the values obtained in the t-test, the strength of association between flashing devices and the mean reduction in accident rate is

$$\text{est } \omega^2 = \frac{(6.34)^2 - 1}{(6.34)^2 + 82 - 1} = 0.32$$

The next step is to subgroup the 82 intersections by the independent parameters previously defined. Given significant t-test results in any of the subgroups, ω^2 is used to determine any increase in predictability achieved by the refinement in the independent variable.

Flasher Type and Intersection Type

The remaining efforts of the study were directed toward determining the most efficient application of the flashing device. The most efficient combination of flasher type and one or more other variables may be masked by less efficient combinations.

Table 1 gives the mean differences in accident rate, indicated by $\overline{D}$, for the 82 intersections, cross-classified according to type of flasher and intersection type (geometrics). The values listed at the end of each row represent the mean difference in

TABLE 1

MEAN DIFFERENCE IN TOTAL ACCIDENT RATE—
INTERSECTION TYPE VS FLASHER TYPE

INTERSECTION TYPE / TYPE OF FLASHER	TYPE I	TYPE II	TYPE III	TOTAL
TYPE I	N = 2 $\bar{D}$ = 2.46 Non-Sig.	N = 8 $\bar{D}$ = 0.97 Non-Sig.	N = 0	N = 10 $\bar{D}$ = 1.27 Sig.
TYPE II	N = 2 $\bar{D}$ = 0.13 Non-Sig.	N = 2 $\bar{D}$ = 0.93 Non-Sig.	N = 0	N = 4 $\bar{D}$ = 0.53 Non-Sig.
TYPE III	N = 6 $\bar{D}$ = 0.39 Non-Sig.	N = 43 $\bar{D}$ = 1.73 Sig.	N = 4 $\bar{D}$ = 0.05 Non-Sig.	N = 53 $\bar{D}$ = 1.45 Sig.
TYPE IV	N = 0	N = 1 $\bar{D}$ = -0.08 Non-Sig.	N = 3 $\bar{D}$ = -0.05 Non-Sig.	N = 4 $\bar{D}$ = -0.06 Non-Sig.
TYPE V	N = 0	N = 9 $\bar{D}$ = 1.49 Non-Sig.	N = 2 $\bar{D}$ = 1.47 Non-Sig.	N = 11 $\bar{D}$ = 1.49 Sig.
TOTAL	N = 10 $\bar{D}$ = 0.75 Non-Sig.	N = 63 $\bar{D}$ = 1.54 Sig.	N = 9 $\bar{D}$ = 0.33 Non-Sig.	N = 82 $\bar{D}$ = 1.31 Sig.

accident rate for each type of flasher, whereas the values at the bottom of each column carry the same meaning classified by intersection type. The values within the table represent the mean difference in accident rate for those intersections exhibiting combinations of the two variables. The sample size of each group is indicated by N. The results of the t-test conducted within each cell are also given.

Individual t-tests were conducted on the mean difference in accident rate for each flasher type. Three types of flashing devices exhibited a reduction in accident rate significantly different from zero. The group of intersections with the flashing stop sign mounted on the side of the road experienced a significant reduction in total accidents of 1.27 acc/mv. The degree of association between this type I flasher and the mean reduction in total accident rate is 0.29. Neither of these figures represents an improvement over the corresponding values for all 82 intersections; therefore, these results were not considered worthy of further investigation.

The group of intersections with two, single-lens units placed overhead, each centered over a lane on the minor road, experienced a significant reduction in accident rate of 1.49 acc/mv. This mean reduction for the 11 intersections with the type V flashing device was greater than the mean reduction for all 82 intersections. However, the degree of association ($\omega^2 = 0.30$) accompanying this higher mean reduction was slightly less than that associated to the mean reduction at all 82 intersections.

The other group of intersections yielding significant results was the group with a single flashing assembly centered overhead on the minor approach (type III). The group had a mean reduction in accident rate of 1.45 acc/mv, and improvement over the reduction shown for the total sample. The strength of association between the type III device and a reduction in accident rate is 0.34, also an improvement over the predictability afforded all 82 intersections.

The second variable, intersection type, was investigated next. Applying the t-tests to the column totals (Table 1) shows only one group with a significant reduction in accident rate. This group, 4-leg intersections with 2-lane main and minor approaches, has a mean reduction in accident rate of 1.54 acc/mv and ω^2 of 0.37. Both values are an improvement over the same measures for the total sample.

It is possible that flasher type III is effective on all three intersection types, and that all five types of flashing devices are effective when used at the type II intersection. Application of the t-test to each cell in row 3 and column 2 (Table 1) was used to determine the validity of this hypothesis. The results indicate that it is only the intersections combining a type III flashing device system and a type II intersection that have a significant reduction in accident rate. Associated with this mean reduction of 1.73 acc/mv is a predictability of 0.40. This combination of the two variables appears to be the most efficient application. This subgroup contains 43 intersections and appears to be the core affecting the overall reduction in accident rate of 1.31 acc/mv for the total sample. Table 1 indicates that no other combination has occurred more than nine times and that all of the mean reductions in accident rate experienced by these groups were not significant.

Flasher Type vs Line-of-Sight

The line-of-sight parameter may provide an even more efficient means of predicting accident reduction when used either singly or in combination with flasher type. The values for the mean reduction in accident rate at the 82 intersections cross-classified by flasher type and line-of-sight are given in Table 2.

The t-test was used as the measure of effectiveness and the predictability was estimated by ω^2. The column totals represent the sample size subgrouped by line-of-sight; the t-test results indicate that the intersections in each of the three subgroups had a significant reduction in accident rate. The strength of association between accident rate reduction and line-of-sight was determined and is given in Table 3. Group III, those intersections with a line-of-sight rating of 0.2 mi or more, had the highest mean reduction in accident rate, but the lowest strength of association.

Row 3 (Table 2) gives the mean reduction in accident rate for those intersections with a type III flashing device subclassified by line-of-sight. This group of 53 intersections had a significant reduction in accident rate of 1.45 acc/mv and an associated predictability of 0.34. The t-test was conducted on each subgroup and the ω^2 value was determined to see if these figures could be improved by considering line-of-signt in addition to flasher type.

Those intersections with a line-of-sight of 0.1 mi and 0.2 mi showed significant reductions in accident rate of 1.20 acc/mv and 1.29 acc/mv, respectively. The strength of association for each group was 0.45 and 0.49, respectively. Again, it was group III that had the highest mean reduction in total accident rate, 2.02 acc/mv, but the lowest degree of predictability, 0.22. Table 4 gives these results. The mean reductions in accidents for groups I and II were less than those for all intersections displaying a type III flashing device. However, these reductions were accompanied by higher degrees of predictability.

TABLE 2

MEAN DIFFERENCE IN TOTAL ACCIDENT RATE–
FLASHER TYPE VS LINE-OF-SIGHT

LINE OF SIGHT / TYPE OF FLASHER	GROUP I $0 < L.S. \leq 0.1$	GROUP II $0.1 < L.S. \leq 0.2$	GROUP III $0.2 < L.S. \leq \infty$	TOTAL
TYPE I STOP	N = 4 $\bar{D}$ = 0.96 Non-Sig.	N = 1 $\bar{D}$ = 1.93 Non-Sig.	N = 5 $\bar{D}$ = 1.38 Non-Sig.	N = 10 $\bar{D}$ = 1.27 Sig.
TYPE II STOP	N = 1 $\bar{D}$ = 1.13 Non-Sig.	N = 1 $\bar{D}$ = -0.86 Non-Sig.	N = 2 $\bar{D}$ = 0.93 Non-Sig.	N = 4 $\bar{D}$ = 0.53 Non-Sig.
TYPE III	N = 20 $\bar{D}$ = 1.20 Sig.	N = 19 $\bar{D}$ = 1.29 Sig.	N = 14 $\bar{D}$ = 2.02 Sig.	N = 53 $\bar{D}$ = 1.45 Sig.
TYPE IV	N = 1 $\bar{D}$ = -0.08 Non-Sig.	N = 1 $\bar{D}$ = 0.46 Non-Sig.	N = 2 $\bar{D}$ = -0.31 Non-Sig.	N = 4 $\bar{D}$ = -0.06 Non-Sig.
TYPE V	N = 5 $\bar{D}$ = 2.16 Non-Sig.	N = 2 $\bar{D}$ = 0.42 Non-Sig.	N = 4 $\bar{D}$ = 1.19 Non-Sig.	N = 11 $\bar{D}$ = 1.49 Sig.
TOTAL	N = 31 $\bar{D}$ = 1.28 Sig.	N = 24 $\bar{D}$ = 1.12 Sig.	N = 27 $\bar{D}$ = 1.52 Sig.	N = 82 $\bar{D}$ = 1.31 Sig.

TABLE 3

EFFECT OF LINE-OF-SIGHT ON ACCIDENT
REDUCTION–ALL INTERSECTIONS

Line-of-Sight	N	$\bar{D}$	Sig.	ω^2
0 < line-of-sight ≤ 0.1	31	1.28	Yes	0.37
0.1 < line-of-sight ≤ 0.2	24	1.12	Yes	0.43
0.2 < line-of-sight ≤ ∞	27	1.52	Yes	0.24
Total	82	1.31	Yes	0.32

TABLE 4

EFFECT OF LINE-OF-SIGHT ON
ACCIDENT REDUCTION–ALL INTERSECTIONS WITH
TYPE III FLASHING DEVICE

Line-of-Sight	N	$\bar{D}$	Sig.	ω^2
0 < line-of-sight ≤ 0.1	20	1.20	Yes	0.45
0.1 < line-of-sight ≤ 0.2	19	1.29	Yes	0.49
0.2 < line-of-sight ≤ ∞	14	2.02	Yes	0.22
Total	53	1.45	Yes	0.34

TABLE 5

EFFECT OF LINE-OF-SIGHT ON ACCIDENT REDUCTION–INTERSECTION TYPE II VS FLASHER TYPE III

Line-of-Sight	N	$\overline{D}$	2 Sig.	ω^2
0 < line-of-sight ≤ 0.1	16	1.44	Yes	0.53
0.1 < line-of-sight ≤ 0.2	16	1.51	Yes	0.62
0.2 < line-of-sight ≤ ∞	11	2.46	Yes	0.26
Total	43	1.73	Yes	0.40

It had been determined that the group of installations combining the type III flashing device and the type II intersection had a mean reduction in accident rate of 1.73 acc/mv and a predictability of 0.40. This group of 43 intersections was then divided into three subgroups based on line-of-sight to further identify the effect of sight distance by holding constant the variables of flasher type and intersection type. Table 5 gives the results of the analysis.

Significant results were obtained in each subgroup, which indicated that each line-of-sight classification is associated with a reduction in total accident rate. However, the degree of association or predictability varied appreciably for the three line-of-sight classifications. Group III experienced the greatest mean reduction in accident rate, but the lowest strength of association.

Similar results were obtained in all testing involving the variable of line-of-sight—significant reductions in accident rate for each subgroup, with groups I and II having lower mean reductions, but associated with higher degrees of predictability than group III. This is interpreted to mean that the effect on accident reduction of future installations of flashing devices will not be influenced by the parameter of line-of-sight. The relatively low value of ω^2 for group III intersections indicates that the mean reduction in accident rate has a much lower probability of duplication at future installations than do the mean reductions for groups I and II.

Volume Analysis

The group of four-leg intersections with all approaches containing two lanes and with the type III flashing device appeared to be the data core, which has influenced the results of all previous testing. Any subgroup which had a significant reduction in accident rate seems to have been formed primarily by intersections found in this basic group of 43 intersections. To determine the factors influencing its relatively high reduction in accident rate, the vehicular volume patterns at these intersections were investigated.

The total number of vehicles passing through an intersection is presumed to have some effect on its accident rate. The method of computing the intersection accident rate was that of dividing the number of accidents by the sum of the intersecting approach volumes. This method does not contain a direct measure of the conflict or interaction experienced by the two intersecting volumes, although it is influenced by traffic increases on either of the two highways.

It is believed that this accident-volume relationship is affected not only by the total volume through the intersection but also by the division of this total between the main and minor roadways. Therefore, this division of the total volume through an intersection was used to form the ratio of minor road volume to main road volume. Then, using correlation and linear regression, an attempt was made to relate intersection accident rate to total intersection volume and volume ratio.

The total volume figures were computed by summing the volumes on the two intersecting highways in the manner described previously. The ratio of minor road volume to main road volume was formed to represent the conflict within the total volume. The values of total intersection volume, volume ratio, and intersection accident rate were computed twice for each intersection—one set representing the before and the other the after period. The intersection accident rate was formed by dividing the total number of accidents by the total intersection volume.

The dependent variable, accident rate, represents the actual accident rate in each time period. In the preceding portions of this report, it represented the difference in accident rate between the two time periods. The difference or reduction in accident rate was not used in the regression analysis because of radical differences in the total volume and volume ratio values between the before and after periods. If the difference in accident rate had been used as the dependent variable, it would have been necessary

TABLE 6

BEFORE AND AFTER STATISTICS–FLASHER TYPE III AT TYPE II INTERSECTIONS

Mean and Std. Deviation	No Flashing Device			Type III Flashing Device		
	Acc. Rate	Total Volume	Volume Ratio	Acc. Rate	Total Volume	Volume Ratio
Mean	2.68	4445	0.400	0.95	4518	0.444
Std. deviation	2.21	2743	0.229	0.72	2846	0.264

to use the total volume and volume ratio averaged over the 4-yr study period. Instead, separate before and after relationships were derived for direct comparison.

Table 6 gives some statistics describing this sample of 43 intersections in the before and after periods. The periods represent the transition from no flashing device to the addition of the type III flashing device.

The mean accident rate at this type II intersection with no flashing device was 2.68 acc/mv. After installation of the type III flashing device, the mean accident rate was reduced to 0.95 acc/mv. The standard deviations associated with these two means are a measure of the scatter of the individual accident rates about these means. Multiple correlation was used to determine if any of the variability of these accident rates could be accounted for by the variability of volume ratio and total volume. Multiple regression was used to form an estimate of this relationship.

Significance tests were also conducted to determine if the correlation found was not merely a chance occurrence. However, the significance level obtained is of little value compared with the estimate of the ability to predict, as indicated by the square of the correlation ratio R. Low values of R^2 can be accompanied by extremely significant results, showing that the ability to predict accident rate from the two volume variables in a linear fashion is not completely absent, but that there may be no practical advantage in using this linear prediction. Serious consideration was given to results that were both significant and had an R^2 indicating an important reduction in the variance of the accident rate, given the regression equation and the two volume variables.

The results of the correlation analysis relating accident rate to volume ratio and total intersection volume indicated that a strong linear relationship was not present in the sample of 43 intersections. The square of the multiple correlation coefficient was 0.38 for the before period and 0.20 for the after. Neither value was considered worthy of further consideration. The next step was to consider each volume variable individually in relation to accident rate.

Total intersection volume had a low linear relationship to the accident rate in each study period. The R^2 values for the before and after periods were 0.11 and 0.004 respectively. This means that the variability in accident rate at the 43 intersections in each study period cannot be explained by the changes in the total volume within each study period.

The results of the correlation analysis on accident rate and volume ratio were very similar to those obtained in the multiple correlation analysis. R^2 values of 0.37 and 0.18 were obtained for the before and after periods, respectively. Table 7 gives the results of the entire correlation analysis. The statistic s′ is the standard error of the regression estimate. It is a measure of the scatter of observed accident rates about the linear regression estimates, and can be compared to the standard deviations given in Table 6 as another indication of the reduction in the variance of the dependent variable that is gained through the use of the linear relationship.

The relatively low values of R^2 indicate that the linear relationship in the before and after periods is not sufficient to provide for the establishment of warrants based on total volume or volume ratio. This correlation analysis was conducted on the 43 sample intersections with a high mean reduction in accident rate to determine if the total volume or volume ratio affected this overall reduction. The results show that, for all

TABLE 7

EFFECT OF VOLUME ON ACCIDENT RATES

Accident Rate vs:	No Flasher		Type III Flasher	
	R^2	s′	R^2	s′
Total volume and volume ratio	0.38	1.78	0.20	0.66
Total volume	0.11	2.11	0.004	0.71
Volume ratio	0.37	1.77	0.18	0.66

R = correlation coefficient; s′ = standard error of regression estimate.

TABLE 8

EFFECT OF FLASHING DEVICES ON ACCIDENT REDUCTION–DAY VS NIGHT

Time Period	No. of Accidents		Percent Reduction	χ^2	Significant
	Before	After			
Day	463	296	36	0.46	NO
Night	206	121	41		

practical purposes, the variation of the individual accident rates in the before and after periods about their respective means cannot be explained by changes in the total volume and volume ratio parameters.

Day-Night Analysis

There is some rationale that flashing devices are more effective at night because of increased visibility. This portion of the study was conducted to determine if flashing devices at the 82 intersections resulted in a significantly different reduction in nighttime accidents compared with daytime accident reduction.

The χ-square test was used to test the hypothesis that there was no significant difference, at the 0.05 level, in the ratio of day-night accidents after the installation of flashing devices.

Previous analyses were directed towards describing the effectiveness of flashing traffic control devices by relating the reduction in total accident rate to several independent variables, considered both individually and in combination. Those same methods of classifying and cross-classifying the intersections were used to determine if those groups of intersections with a significant reduction in overall accident rate were affected by the light condition. Again, the analysis proceeded from the general to the specific.

Table 8 gives the results of the χ-square analysis on the day-night accident data for the 82 intersections. The nonsignificant results indicate that the reduction in nighttime accidents was in the same proportion as for daytime accidents.

The intersections were then cross-classified according to flasher type and intersection type. Each classification group was tested to determine if day accidents and night accidents varied in the same proportion after installation of a flashing device.

Significant results were obtained in only one test. The group of 11 intersections with the type V device experienced a 68 percent reduction in nighttime accidents, differing significantly from the 29 percent reduction during daylight. Because of the relatively small type V sample size, the conclusion that this type of flashing device is best suited for reducing night accidents was not drawn.

Nonsignificant results were obtained in all other tests, indicating that the use of flashing devices had an equal effect on the reduction of accidents in both light conditions. The previous analyses were conducted to determine which combinations of independent variables seemed to influence a significant reduction in total accident rate. This analysis demonstrated that the flashing devices were equally effective with respect to day and night accidents regardless of intersection type or flasher type, with the possible exception of the type V device.

Accident Type Analysis

An accident type analysis was conducted to determine if the use of flashing devices was effective in reducing any particular type of accident. Three types were considered: angle collisions, rear-end collisions, and a combination of all other types of collisions. The χ-square test was again used, with the 0.05 level of significance. The test was first conducted on the breakdown of accidents occurring at all 82 intersections. Table 9 gives the results.

The nonsignificant results indicate that the before-after accident ratio was the same for each accident type for the total sample. The usual testing procedure was followed next.

TABLE 9

EFFECT OF FLASHING DEVICES ON ACCIDENT REDUCTION BY ACCIDENT TYPE

Accident Type	Before	After	Percent Reduction	χ^2	Significant
Angle	412	253	39		
Rear-end	125	65	48	2.15	NO
All others	133	99	26		

The only group of intersections showing significant results was the group with the type V flashing device. The reduction in angle collisions was 42 percent; in rear-end collisions, 88 percent; and in all other types of collisions, only 4 percent. These results for a sample of only 11 intersections were interpreted as an indication that the type V device could be most effective in reducing rear-end accidents.

Nonsignificant results were obtained in all of the remaining tests, indicating that the five types of flashing devices had an equal effect on the three types of collisions regardless of the type of intersection. The precise nature of this effect was determined in the initial analysis evaluating the reduction in total accident rate given these same independent parameters. Here, analysis established independence of accident type and flashing device installation, given the two variables of flasher type and intersection type.

The data were then reclassified by line-of-sight and type of flasher. The χ-square analysis was repeated to determine if there was any relation between type of accident and the combination of the two independent variables.

Nonsignificant results were again obtained, indicating that the change in the three accident types following the installation of a flashing device is independent of the line-of-sight, and also independent of the combination of line-of-sight and type of flashing device.

CONCLUSIONS

The flashing device, which consists of a bouncing ball either horizontal or vertical, resulted in the greatest decrease in accident rate. This decrease was statistically significant when compared to the other flasher types.

Neither volume alone nor the ratio of main or minor road volumes influenced the accident rate reductions determined in this study. Intersections carrying a high volume of traffic and consisting of a 4-lane divided highway and a 2-lane road did not exhibit a significant reduction in accidents following the installation of a flashing device.

The parameters of sight distance, type of accident, and day vs night accidents did not contribute to an understanding of flasher effectiveness. There were no significant changes in these parameters when classified by flasher type or intersection type.