A Reassessment of the Traffic Signal Change Interval

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ABSTRACT

Data from field studies of intersections in Arizona and information from the literature were used to make an in-depth examination of the traffic signal change interval. This examination included a review of the traditional concept and theory on which the determination of the change interval has been based and an evaluation of the applicability of this theory. The analysis of driver behavior and characteristics indicates that the majority of drivers do not conform to the model, which assumes a constant or uniform deceleration rate. In fact, the deceleration profile is related to the approach speed. An analysis of the first vehicle to stop at an intersection and the last vehicle to clear the intersection was made based on the time from the intersection at the onset of the yellow interval. This analysis revealed that the time for the last vehicle to clear the intersection is more critical in the design of the yellow interval. It is concluded that a uniform 4-sec yellow interval would be acceptable.

In response to concerns about the traffic signal change interval, research was initiated in 1981 for the Arizona Department of Transportation. The initial study served to document measures of driver behavior and variations in behavior that are associated with the traffic signal change interval at intersections in Arizona.

Based on findings of the initial study, subsequent research was undertaken by the University of Arizona for the Arizona Department of Transportation. This more recent work involved further field studies of driver behavior and traffic characteristics at intersections in Arizona as well as a comprehensive analysis of the data that were collected in the course of the field studies. The traffic signal change interval is evaluated on the basis of the analysis and findings of the extensive field studies conducted.

REVIEW OF LITERATURE

For many years, the yellow interval and the yellow plus the all-red interval was simply known as the clearance interval. Several suggestions have been made relative to the term that should be used, and the literature contains references to a number of different terms. Currently, it is more common to find the term "change interval" used in the literature.

Although early discussions of the yellow interval may be found in the literature (1,2), the current concept can be traced to the work of Gazis et al. (3). Based on their findings, the policies for the determination of the yellow interval began to be based on specific values of driver perception-reaction times and deceleration rates. The third edition of the Traffic Engineering Handbook (4) indicated that the purpose of the yellow interval is twofold, as follows:

1. To advise drivers that the green interval is about to end and to permit them to come to a safe stop, and

2. To allow vehicles having entered the intersection legally to clear the point of conflict before the release of opposing pedestrians or vehicles.

The following equations were given for use in determining the yellow interval:

$$y = t + (v/2a)$$

$$y = t = (v/2a) + [(w + l)/v]$$

where

t = driver perception-reaction time (sec),
v = approach speed (ft/sec),
a = deceleration rate (ft/sec^2),
w = width of intersection (ft), and
l = length of vehicle (ft).

The first equation calculated the time required for the driver to come to a safe stop, whereas the latter equation determined the time for an approaching vehicle to clear the intersection. The limiting values of t = 1 sec, a = 15 ft/sec^2, and l = 20 ft were suggested. It was also noted that the yellow interval was generally 3 to 5 sec, and the all-red interval was to be used where the needed clearance exceeded the selected yellow interval or where a hazardous conflict was likely.

Since the 1960s, a number of studies have addressed various aspects of the problem associated with the determination of the change interval. On the basis of field studies, Olson and Rothery (5) concluded that driver behavior did not change with different amber phase durations and that an amber phase of about 5.5 sec would be suitable for a wide range of speed zones. Herman et al. (6) also presented a discussion of the field study data in terms of the probability of stopping.

May (7) undertook a rather extensive examination of the change interval in terms of the practices that were being utilized as well as field studies that analyzed the effects of changes in the duration of the yellow interval in addition to the effects of signs and markings. In that work, it was found that increases in the duration of the yellow interval in an urban location increased the percentage of motorists operating in an unsafe or unexpected manner,

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whereas the reverse was true in a rural area. The use of the experimental pavement markings slightly decreased the percentage of motorists operating in an unsafe or unexpected manner in the urban area and increased the percentage in the rural area.

A study in 1966 by Jenkins (8) evaluated the reaction time of motorists. Based on a rather limited sample, the study found that the mean reaction time was 1.4 sec, which was slightly higher than the value of 1.14 sec that was reported in the earlier work of Gazis et al. (3).

Williams (9), in a study of an intersection in New Haven, Connecticut, found the average maximum deceleration rate for stopping vehicles to be 9.7 ft/sec^2. Other studies (10,11) suggested that a reasonable deceleration rate would be in the magnitude of 10 ft/sec^2. Also, Parsonson and Santiago (11) indicated the need for considering the effect of grade on the deceleration rate that is used in the determination of the change interval. A deceleration rate of 10 ft/sec^2 is reflected in the calculations for the minimum theoretical clearance intervals in the current edition of the Transportation and Traffic Engineering Handbook (12).

The study by Wortman and Matthias (13,14) was initiated in 1981 for the purpose of documenting driver behavior during the change interval for intersections in Arizona. The results of the study indicated that the mean deceleration rates at six sites ranged from 7.0 to 13.9 ft/sec^2, and the mean value for all observations was 11.6 ft/sec^2. The mean driver response time was found to be 1.3 sec. Comparisons of behavior at intersections with yellow-only change intervals and intersections with yellow-plus-all-red change intervals did not yield clear conclusions.

The initial work by Wortman and Matthias gave direction to further field studies in Arizona (15). This follow-up work focused specifically on determining the influence of (a) the variation in the duration of the yellow interval, (b) the effect of enforcement, and (c) intersection approach grades. For this second study, additional data were collected at five intersections in the Tucson area. Of these five intersections, only one had been used as a study site in the earlier study. The results of the field studies generally substantiated the range of values of the deceleration rates and driver response time found in the earlier work. Basically, the result of extending the duration of the yellow interval was the reduction in the percentage of vehicles entering on the red signal indication. Although the presence of a police vehicle at the site significantly reduced the percentage of vehicles entering on the red signal indication, an extension of the duration of the yellow interval provided a more effective treatment. As with the previous study, there was considerable variation in the specific values of driver behavior at the sites that were studied.

Under a contract with FHWA, the Texas Transportation Institute (TTI) undertook a somewhat parallel study of traffic signal change intervals (16). The research included field studies of sites in Texas and Virginia as well as the development of alternative methods for the design of the signal change interval. On the basis of their research, the TTI team recommended the use of a perception and brake reaction time of 1.2 sec and and a deceleration rate of 10.5 ft/sec^2. Four alternative methods for determining the change interval were examined and included:

1. Continued use of the current formula with one perception and brake reaction time and one deceleration rate for all approach speeds,
2. Continued use of the current formula with different perception and brake reaction times and deceleration rates for different approach speeds,
3. Design change interval based on time of clearing vehicles, and
4. Design change interval based on the probability of stopping or clearing.

The method involving clearing vehicles focused on the time required for the last through vehicles to enter the intersection after the onset of the yellow interval. It should be noted that this method yields a uniform yellow interval for all approach speeds. This approach to determining the yellow interval would tend to support the suggestion of a uniform yellow by Williams (9). The last method reflects a concept contained in several previous studies (9,17, 18) and is based on the probability of stopping given a distance from the intersection and the approach speed.

ITE Technical Committee 4A-16 has also recently been examining the policy for change interval design and calculation (19). The proposed formula by that group reflects the current ITE guidelines along with the inclusion of the approach grade in the calculation of the yellow interval. In addition to the calculation of the yellow interval by the formula, the ITE Committee states the following (19):

> When the percent of vehicles that are last through, which enter on red, exceeds that which is locally acceptable (many agencies use a value of 1 to 3 percent), the yellow interval should be lengthened until the percentage conforms to local standards.

As has been indicated previously, in the determination of the change interval, the time required for an approaching vehicle to stop as well as that for vehicles that have legally entered the intersection to clear should be considered. Over the years, some traffic engineers have employed an all-red interval when the required yellow time exceeded some value, such as 5 or 6 sec. The intent was to eliminate extremely long yellow signal indications. In recent years, however, several papers have addressed the legal definition of the yellow signal indication. For example, Bissell and Warren (20) argue that the yellow interval cannot be considered a clearance interval under the laws that permit the vehicle to enter an intersection during the yellow signal indication. They contend that vehicles must clear the intersection during the red signal indication. A similar discussion is presented by Butler (21). The proposed ITE policy (19) also indicates that "if clearance time is to be provided, it should be in the form of a red clearance interval." The equations for determining the red interval basically reflect the time required for a vehicle to traverse and clear the intersecting street.

The TTI study (16) proposed a method of determining an all-red interval that took into account the increase in speed of clearing vehicles and the start delay time on the cross street. The time required for the vehicle to approach and clear the intersection less the start delay time is compared with the required yellow time. If the calculated time for the clearing vehicle is greater than the yellow time, an all-red interval is necessary. Accident studies of intersections where the all-red interval has been used indicate that accident reductions were found where the interval was utilized. Newby (22) in 1961 reported a 41 percent reduction in injury accidents at intersections after the addition of an all-red period. In a study for FHWA, Benioff et al. (23) concluded that the provision of an all-red interval resulted in the reduction of accidents. They also suggested that intersections with a right-angle accident rate of greater than 0.8 accident per million entering vehicles should be considered for the addition of an all-red interval. In their review of previous work, Chang and Messer (16) cite several other studies by specific jurisdictions that indicate accident reductions with the application of the all-red interval.

The research by TTI (16) did examine the effects of grade on deceleration performance. The results of that work indicate that the following equation may be used to determine the deceleration rate for a given grade:

$$d = 10.5 \pm 0.075g$$

where d is the deceleration rate in feet per second squared and g is the percent of grade.

This equation reflects the recommended use of 10.5 ft/sec² for level roadway conditions. The TTI study recommends that as a general rule a value of 10.5 ft/sec² can be used for level and upgrade conditions, whereas 10.0 ft/sec² can be used for downgrade conditions. This work would tend to indicate that the effect of grade is somewhat minimal.

THEORETICAL CONSIDERATIONS

Generally, the work that has been done to date in relation to the signal change interval assumes a constant or uniform deceleration rate. The early development of the theory by Gazis et al. (3) as well as the policies that followed all include this assumption. Even the proposed ITE policy (19) as well as some of the alternatives considered in the TTI study continue to be founded on this basis. Although this assumption is convenient for computational purposes, there are other theoretical considerations that should be taken into account when driver and vehicle characteristics are evaluated.

The review of the literature indicates a concern about the deceleration rate that should be used, the perception-reaction or response times of drivers, and whether approach grades are significant in terms of changing the deceleration characteristics of the stopping vehicles. In utilizing the equations that have traditionally been used in the determination of the change interval, the calculation of the duration of the interval is certainly sensitive to these parameters. It would appear, however, that a more important question is related to the assumption of constant or uniform deceleration.

Given a uniform or constant deceleration rate, the following equations can be used to calculate the deceleration rate:

$$a = v^2/2x \quad (1)$$

$$a = 2x/t^2 \quad (2)$$

$$a = v/t \quad (3)$$

where

- a = deceleration rate (ft/sec²),
- v = initial approach speed (ft/sec),
- x = distance traveled while stopping (ft), and
- t = time required to stop (sec).

These equations are valid for a situation in which the vehicle comes to a complete stop at the end of the period of deceleration, as would be the case at a signalized intersection. Figure 1 shows the theoretical relationship of vehicle speed, time, and distance for a decelerating vehicle. The distance traveled is represented by the area under each curve, and the deceleration rate is shown by the slope of the curve. Curve I represents a case where the deceleration rate is constant; thus this curve is a straight line. As has been indicated previously, the current policies on change intervals utilize this concept. With constant deceleration rates, the application of any of the three equations will yield the same answer.

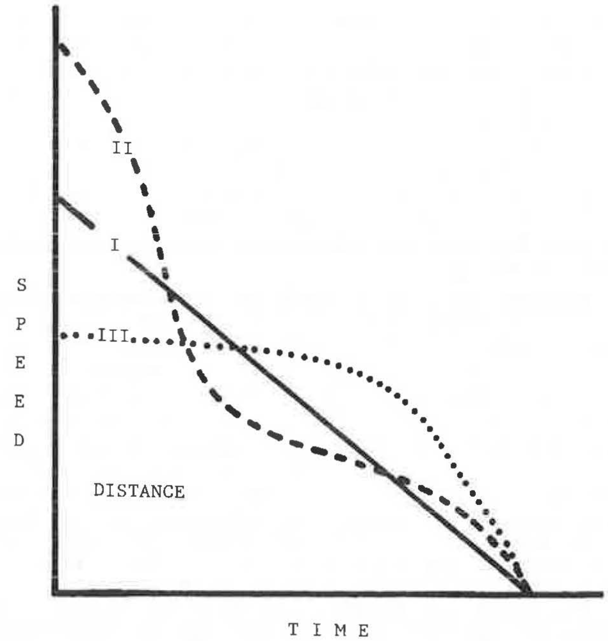

FIGURE 1 Theoretical relationship of vehicle speed, time, and distance.

In cases involving nonuniform deceleration, as shown by Curves II and III in Figure 1, the relationship between speed, time, distance, and the deceleration rate breaks down. For example, in the three cases shown in Figure 1, the initial speed differs, but the time of deceleration and the distance traveled are the same. The difference in the three curves is caused by differing deceleration profiles. Similar curves can be developed for cases in which the initial speed and the time of deceleration are the same; however, the distance traveled differs for the different deceleration profiles. Also, the same would be true for a situation in which the deceleration times differ even though the initial speed and the distance traveled are the same.

It is important to recognize that the equations that were presented earlier do not yield consistent results for situations involving nonuniform deceleration. In fact, the calculated deceleration rates can vary considerably depending on the parameters used to calculate the value. The variation in the calculated values of the deceleration rates increases with the increases in the deviation from the linear deceleration profile represented by a constant deceleration.

In the field studies for the Arizona research (13,15), time-lapse photography was used for the field collection of data. With this study method, it was possible to obtain the approach speed, the time of deceleration, and the distance traveled for each of the stopping vehicles in the data sample.

Utilizing the data from the five intersections included in the the latter Arizona field studies, an analysis was undertaken to determine the percent of stopping vehicles that decelerated in what approximated a uniform or constant rate. The analysis compared the calculated deceleration rates for the equations that use (a) the initial approach speed and deceleration distance and (b) the deceleration distance and time. Again, in cases where the vehicular deceleration approximates a constant deceleration rate, the calculated values should be approximately the same. The results of this analysis revealed that only about 31 percent of the stopping vehicles had deceleration profiles that approximated the constant rate condition; therefore a majority of the stopping vehicles displayed nonuniform deceleration rates.

Further analysis revealed a relationship between initial approach speed of the vehicle and the deviation from the profile representing a constant deceleration rate. For this analysis, a ratio was calculated that indicated the relative difference in the deceleration rates as determined from the different equations. The ratio of the deceleration-rate values (Q) was computed by dividing the deceleration rate using Equation 1 by the deceleration rate using Equation 2. A Q-value of 1.0 indicates a constant uniform deceleration rate. If the Q-value is greater than 1.0, the analysis revealed that the driver selected a higher deceleration rate initially and then a lower deceleration rate as the vehicle slowed. With a Q-value of less than 1.0, the inverse would be true and the driver would increase the deceleration rate as the intersection was approached.

The relationship of the Q-values and the initial approach speed of the stopping vehicles is plotted in Figure 2. It should be noted that there is a change in the deceleration profile with changes in approach speed. In fact, at higher approach speeds, drivers select a higher deceleration rate initially and then reduce the deceleration rate as the intersection is approached. The importance of this relationship is that a vehicle with a higher approach speed will take the same time to come to a stop as one with a lower approach speed. In addition, the results of this analysis indicate that there is a reversal in the deceleration profiles at approach speeds of about 48 mph where the Q-value is 1.0. This has a dramatic effect on the actual time required for a vehicle to stop and raises questions about the traditional manner in which minimum-yellow-time calculations have been made.

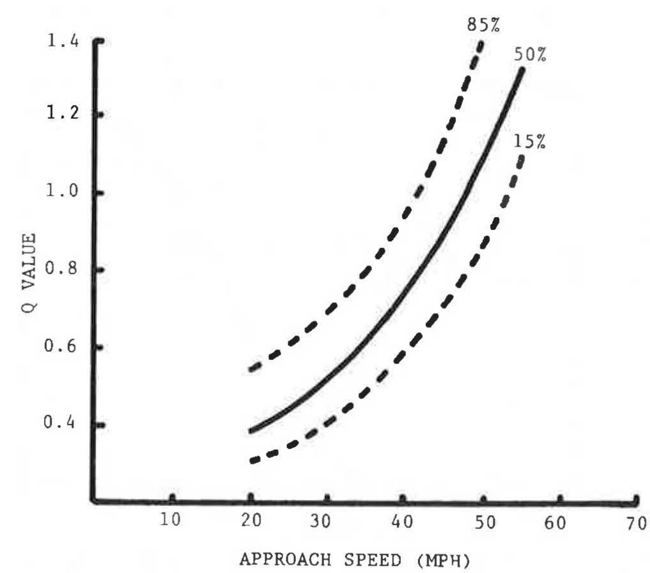

FIGURE 2 Relationship of Q-values and approach speed.

ANALYSIS OF DRIVER BEHAVIOR

Based on the findings that revealed the nonuniform nature of the deceleration rates selected by the driver and the relationship of the deceleration profile and the initial approach speed, an examination was made to determine how this affects the design of the signal change interval. Analyses of the Arizona data were undertaken in an attempt to define similarities and differences in driver behavior for different intersection conditions. The literature indicates that researchers have generally attempted to analyze the behavior of drivers in terms of the distance from the intersection. In fact, the recent data collection efforts in Arizona (13,15) included analyses of the distance from the intersection for the last vehicle through the intersection and the first vehicle to stop after the onset of the yellow interval.

In determining a signal change interval, distance from the intersection at the onset of the yellow interval is simply a surrogate measure; the true measure of interest is the time from the intersection. This analysis of behavior therefore examined the location of the last vehicle through the intersection and the first vehicle to stop in terms of the time from the intersection at the onset of the yellow interval. For the last vehicles through the intersection, the time from the intersection could be determined from the recorded data. The time from the intersection for the first vehicles to stop was calculated by using the distance from the intersection and the approach speed. A number of analyses involving the time from the intersection were undertaken in an attempt to identify the effect of approach speeds and intersection conditions on driver behavior.

Figure 3 shows the time from the intersection at the onset of the yellow interval for the first vehicles to stop and the last vehicle through the intersection for approach speeds of 30, 40, and 50 mph. The curves are plotted showing the cumulative percent. From the field data, the first vehicle to stop at all of the approach speeds was approximately 2 sec from the intersection if the driver had proceeded to the intersection at that approach speed. Although the 40-mph curve shows a value that is slightly higher, the values for the 30- and 50-mph curves are virtually the same.

For the first vehicles to stop, it should be noted that a comparison of the shape of the curves for the

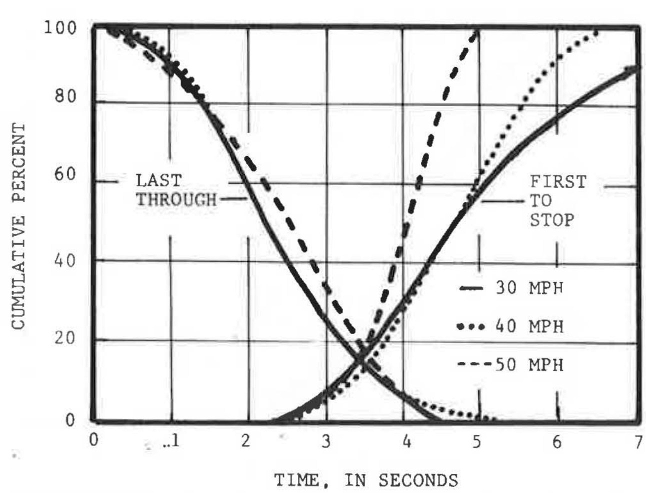

FIGURE 3 Time from intersection at onset of yellow interval: 30-, 40-, and 50-mph approach speeds.

various speed ranges indicates that the slope of the curve increases with approach speed. In the field data collection, approximately 350 to 400 ft of the intersection approach was recorded on film. With higher approach speeds, the first vehicle to stop could easily be outside of the view of the camera. Because of this situation, the maximum time from the intersection that could be recorded for a vehicle decreased with the increase in approach speeds.

For the last vehicles through the intersection, the curves for the various approach speeds show considerable similarity. For signal change interval timing purposes, the critical portion of the curve is at the lower percentages. For example, the curves indicate that regardless of the approach speed, approximately 95 percent of the last vehicles through the intersection are 4 sec or less from the intersection. For all of the approach speeds, the 4-sec value for the 5th-percentile vehicle is quite consistent.

Further examination of this lower portion of the curves for the two groups of vehicles reveals a fact that has major implications in terms of considering the required yellow interval. The time for the last vehicle through the intersection is more critical than that for the first vehicle to stop; thus the determination of the yellow time should be a function of the time for the last vehicle through the intersection.

Also, these curves can be used to define the dilemma zone. For example, the minimum time for the first vehicle to stop and the maximum time for the last vehicle through the intersection indicate the limits of the dilemma zone. In this time range, some drivers stop, whereas other drivers choose to proceed through the intersection.

Certainly, a valid question regarding such an examination of driver behavior pertains to the effect of change interval duration on driver decisions. In the latter Arizona study, data were collected at two intersections where the yellow interval was extended from 3 to 4 sec. At these two intersections, the change interval also included a 2-sec all-red interval. Figure 4 indicates the time curves for the two yellow-interval durations. The extension of the yellow interval did result in slight changes in the curves for the two groups of vehicles, but again only about 5 percent of the last vehicles through the intersection exceed the 4-sec value.

To determine the effect of longer change intervals, data from the earlier study (13) were used. Data were collected at one intersection that had a 5-sec yellow plus a 3-sec all-red change interval. The time curves for this intersection are shown in Figure 5. Again, the critical portions of the curves are similar to those for other intersections. In fact, the data revealed that the last vehicle cleared the intersection before the end of the 5-sec yellow interval. This would tend to refute the theory that drivers enter the intersection later with longer yellow intervals.

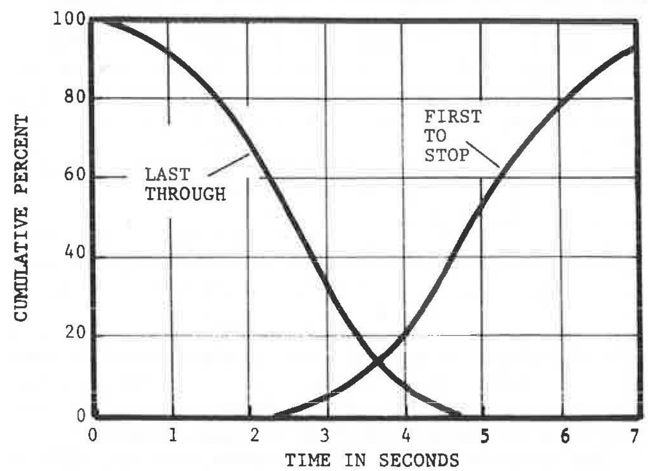

FIGURE 5 Time from intersection at onset of yellow interval: 5-sec yellow plus 3-sec all-red change interval.

In the same analysis for one of the intersections included in the earlier Arizona study at which day and night data were obtained, there was a significant difference in the comparison of the day and night deceleration rates (Figure 6); however, the comparison of the time curves does not indicate a real difference in the characteristics for the first vehicle to stop. At night, however, the data sample indicated that the last vehicles through the intersection did not enter as late in the overall change interval.

It is interesting to note that the approach grade had little effect on the time values for the approaching vehicles. Figure 7 shows the time curves for an intersection that had an approach downgrade of 2.0 percent.

All the data included in the analysis were taken at intersections in the Phoenix and Tucson metro-

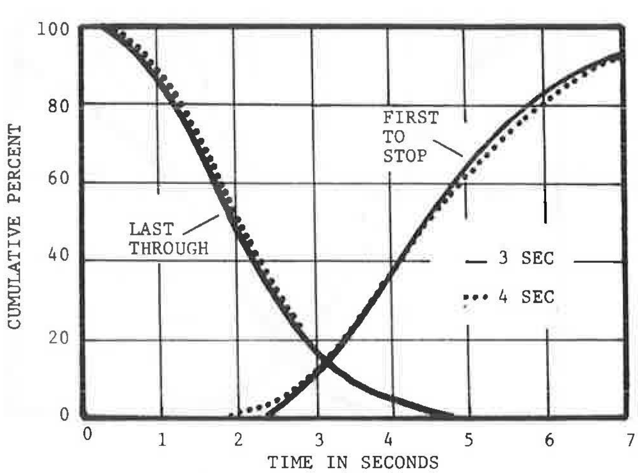

FIGURE 4 Time from intersection at onset of yellow interval: 3-sec versus 4-sec yellow interval.

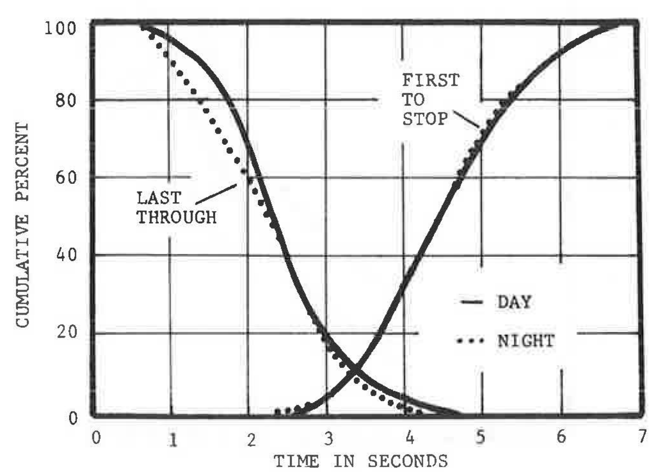

FIGURE 6 Time from intersection at onset of yellow interval: day versus night.

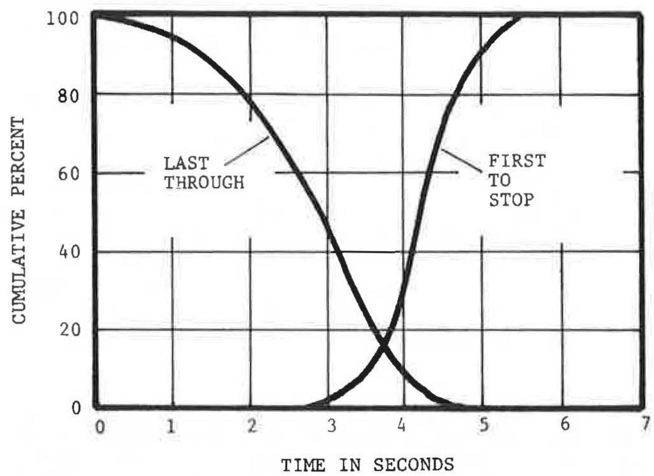

FIGURE 7 Time from intersection at onset of yellow interval: approach with 2 percent downgrade.

politan areas; thus there was concern whether this represented driver behavior in other parts of the United States. The TTI study (16) revealed similar findings and showed that the 95th-percentile time value for all of the approach speeds is about 4 to 4.5 sec. This is slightly higher than that found in the work of this project, but it tends to indicate similar findings. Furthermore, the results of this analysis along with these reported findings from the TTI study support the concept of a uniform yellow interval that has been advocated by some.

CONCLUSIONS

The research described in this paper addressed the validity of the use of a uniform deceleration rate in the calculation of the traffic signal change interval. Furthermore, driver behavior that is associated with the first vehicle to stop and the last vehicle through the intersection was evaluated in terms of the time from the intersection at the onset of the yellow interval. The analysis was based on field studies conducted at intersections in the Phoenix and Tucson metropolitan areas. As with any research of this type, the total number of intersections included in the field study is somewhat limited; however, the data set is one of the most extensive that has been developed to date for this purpose. Based on the research undertaken, the following conclusions can be drawn:

1. About 31 percent of the first vehicles to stop after the onset of the yellow interval had deceleration profiles that approximated a uniform deceleration rate. The majority of the drivers selected deceleration rates that did not remain constant or uniform during the stopping process. It was also found that the deceleration profile was a function of the approach speed and that the initial deceleration rate was higher with the higher approach speeds.
2. The analyses of the time from the intersection at the onset of the yellow reveal that the first vehicle to stop was approximately 2 sec or more from the intersection. At one intersection, a value that was less than 2 sec was observed; however, this was an exception.
3. For the last vehicles through the intersection, it was found that driver behavior did not vary significantly with approach speeds, approach grades, or the duration of the yellow interval. On the basis of the intersections observed, only about 5 percent of the vehicles entered the intersection more than 4 sec after the onset of the yellow. This statistic did not vary significantly with changes in intersection conditions. These findings suggest that the first vehicles to stop after the onset of the yellow interval generally do not conform to the assumption of a constant or uniform deceleration rate that is used in the traditional method of calculating the signal change interval. For the design of the yellow interval, the time for the last vehicle to clear the intersection is more critical, and a uniform yellow interval of 4 sec is required for the 95th-percentile level of operation. The 4-sec value is associated with entry into the intersection and does not include the time required for the vehicle to clear the intersection.

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Abridgment

Countering Sign Vandalism with Public Service Advertising

KATHERINE FRITH

ABSTRACT

Nationwide, millions of dollars are spent each year replacing stolen and vandalized signs. In Iowa, sign vandalism creates hazards that cost taxpayers over $1 million per year. Research that was conducted in Iowa to determine teenagers' attitudes toward and perceptions of the sign vandalism problem is discussed. On the basis of the research, public service advertising messages were developed and tested in a university newspaper. The ads were shown to be significantly effective in raising students' awareness of the fines and penalties attached to sign vandalism. Public service advertising is strongly recommended as an effective countermeasure that should be pursued at the national, state, and local levels to combat sign vandalism.

In a study compiled for the Transportation Research Board in 1983 by Chadda and Carter (1), it was noted that about $50 million is being spent annually in the United States by state departments of transportation to replace stolen and vandalized highway signs. In addition, the indirect costs incurred by state governments for injury and tort liability in accidents that result from missing and vandalized highway signs are estimated to be of about the same magnitude.

Highway signs have become a symbol of modern culture to today's teenagers. They often hang stolen highway signs on the walls of their university dormitories and fraternity or sorority houses (1). So common is the practice of using highway signs as room decorations that stop signs can even be seen on

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