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OPTIMIZATION OF TRAFFIC SIGNAL CHANGE INTERVAL

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16. ABSTRACT

Utilizing data from field studies of intersections in Arizona and information from the literature, an indepth examination of the traffic signal change interval was undertaken. 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.

Because the majority of the first vehicles to stop after the onset of the yellow interval did not conform to a constant and uniform deceleration model, the last vehicles through the intersection were found to be more critical in the design of the change interval. Factors such as approach speeds, approach grades, and the duration of the yellow interval had little or no influence on driver behavior relative to decisions to stop or continue through the intersection. Based on analyses of the time distance of vehicles from the intersection at the onset of the yellow interval, the research suggests that a uniform yellow interval could be utilized.

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METRIC CONVERSION FACTORS

The material contained in this report is presented in terms English units. The following factors may be used to convert the measures used in this report to the International System of Units (SI):

1 mile per hour (mph) = 1.6093 kilometers per hour (kph)

1 kph = 0.6214 mph

1 foot = 0.3048 meter

1 meter = 3.2808 feet

1 foot per second per second = 0.3808 meter per second per second

1 meter per second per second = 3.2808 feet per second per second

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INTRODUCTION

In recent years, highway and traffic engineers have shown an increased concern about the design and duration of the traffic signal change interval. This concern reflects issues and questions which are associated with such areas as traffic safety, liability, and intersection operational efficiency.

Beginning in 1981, research on the signal change interval was undertaken by The University of Arizona and Arizona State University for the Arizona Department of Transportation. This initial study was entitled "An Evaluation of Driver Behavior at Signalized Intersections". The results of that study were published in January of 1983 by the Arizona Department of Transportation(1). A summary of the study was subsequently published in a paper by Wortman and Matthias(2).

The intent of the initial study was to document measures of driver behavior and variations in behavior which are associated with the traffic signal change interval. Given this intent, the project focused on measuring (a) deceleration rates used by drivers, (b) possible differences in behavior due to the intersection environment, (c) the effect of an all-red phase in the change interval, and (d) driver response times. At the time the study was undertaken, it was recognized that the research was an initial effort in Arizona with respect to the documentation of driver behavior during the change interval. It was also recognized that the research would probably give direction to additional efforts which were needed to give a better understanding of the problems associated with the design of the signal change interval.

Based on findings of the initial study, the current project was proposed and included two phases. Phase I was involved with additional field studies of driver behavior which would provide information on the influence of:

(a) the variation in the duration of the yellow interval,

(b) the effect of enforcement, and

(c) intersection approach grades.

The results of the Phase I part of the project are contained in a report entitled "Optimization of Traffic Signal Change Intervals - Interim Report"(3).

The second phase of the project focused on:

(a) a review of literature and research that are pertinent to traffic signal change intervals,

(b) an evaluation and assessment of information and research to date, and

(c) the development of a set of definitive guidelines for use in determining traffic signal change intervals.

This document is the final report for the project and includes the findings and results of Phases I and II of the research.

REVIEW OF LITERATURE

A review of the literature reveals that historically there has been rather limited work which pertains to the signal change interval. More recently, a number of studies have been conducted in a effort to gain a greater understanding of the problem in determining the change interval and defining the numerical values which are appropriate for use. During the past several years, there has been an increased interest in the change interval. This interest has resulted in studies which have taken rather comprehensive examinations of the signal change interval. Some of the studies which have been conducted have included extensive literature reviews. In view of these reviews, no attempt has been made to report on all of the previous work which is contained in the literature.

For many years, the yellow interval and the yellow plus the all-red interval were 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 terms such as the inter-green interval. Currently, it is more common to find the term "change interval" used in the literature.

THE EVOLUTION OF THE CHANGE INTERVAL POLICY

An early discussion of the yellow interval(4) indicated that the duration of the interval could be determined by the following equation:

$$y = .8 + .04V + .7D/V$$

where V = average speed of vehicles, and

D = the distance from the near property line to the far curb lane

In the discussion, it was indicated that the formula did take the average stopping distance of an automobile into account. It is interesting to note that it was stated that "usually the time needed for vehicles to clear the intersection runs from three to five seconds and may often be estimated with sufficient accuracy".

By 1950, the discussion of the clearance interval in the second edition of the Traffic Engineering Handbook(5) indicated the possibility of using the all-red interval. It stated that some engineers preferred to use an all-red interval to provide the necessary clearance time in excess of 5 seconds. Also it indicated that some traffic engineers preferred to use a short all-red interval to separate conflicting flows for "safety's sake". In addition to providing the previous equation for the determination of the clearance interval, the Handbook cited the work of T. M. Matson which suggested the use of the following equations:

$$y = 0.682W/V$$

$$y = 0.682\ (W + S)/V$$

where W = intersecting street width in feet,

V = speed of the clearing vehicle in miles per hour, and

S = minimum vehicle stopping distance in feet.

The first equation was to be used to determine the time required for a vehicle to clear the intersecting street while the second included time to approach and clear the intersecting street if the vehicle had passed a point where comfortable stopping could be achieved.

Based on the work of Gazis, Herman, and Maradudin(6), the policies for the determination of the yellow interval began to be based on specific values of driver perception-reaction times and deceleration rates. By 1965, the third edition of the Traffic Engineering Handbook(7) indicated that the purpose of the yellow interval is twofold as follows:

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

b) to allow vehicles having entered the intersection legally to clear the point of conflict prior to the release of opposing pedestrians or vehicles.

This purpose is based on the fact that a driver may legally enter the intersection during the yellow interval and continue to clear on the red indication. 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 in seconds,

v = approach speed in feet per second,

a = deceleration rate in feet per second per second,

w = width of intersection in feet, and

l = length of vehicle in feet.

The first equation calculated the time required for the driver to come to a safe stop while the latter equation determined the time for an approaching vehicle to clear the intersection. The limiting values of $t = 1$ second, $a = 15$ feet per second per second, and $l = 20$ feet were suggested. At this time, it indicated that the yellow interval was generally 3 to 5 seconds, 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 period of the 1960's, a number of studies have addressed various aspects of the problem associated with the determination of the change interval. Based on field studies, Olson and Rothery(8) concluded that driver behavior did not change with different amber phase durations and that an amber phase of about 5.5 seconds would be suitable for a wide range of speed zones. Herman, Olson, and Rothery(9) also presented a discussion of the field study data in terms of the probability of stopping.

May(10) undertook a rather extensive examination of the change interval in terms of the practices that were being utilized as well as field studies which 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 while the reverse was true in the 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 the use of the markings increased the percentage in the rural area.

A study in 1966 by Jenkins(11) evaluated the reaction time of motorists. Based on a rather limited sample, the study found that the mean reaction time was 1.4 seconds which was slightly higher than the value of 1.14 seconds that was reported in the earlier work of Gazis et. al.(6).

Williams(12), in a study of an intersection in New Haven, Connecticut, found that the average maximum deceleration rate for stopping vehicles to be 9.7 feet per second per second. Other studies(13,14) suggested that a reasonable deceleration rate would be in the magnitude of 10 feet per second per second. Also, Parsonson and Santiago(14) 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 feet per second per second is reflected in the calculations for the minimum theoretical clearance intervals in the current edition of the Transportation and Traffic Engineering Handbook(15).

In Arizona, the current Arizona Department of Transportation policy(16) indicates that a deceleration rate of 10 feet per second per second should normally be used in the determination of the clearance interval; however lower and upper limits of 8 and 12 feet per second per second are also given in the policy. In addition, suggestions are made for including the potential effects of downgrades on the operation of an approach vehicle. It recommends the use of an all-red interval if the time required for clearing the intersection exceeds 6 seconds. The Arizona Department of Transportation Policy is included in the Appendix.

The study by Wortman and Matthias(1,2) was initiated in 1981 for the purpose of documenting driver behavior during the change interval for intersections in Arizona. At that time, the study represented one of the most extensive efforts to collect field data which pertained to the change interval problem. The results of the study indicated that the mean deceleration rates at six sites ranged from 7.0 to 13.9 feet per second per second, and the mean value for all observations was 11.6 feet per second per second. The mean driver response time was found to be 1.3 seconds. Comparisons of behavior at intersections with yellow only change intervals and intersections with yellow plus all-red change intervals did not yield clear conclusions. In fact, differences in behavior were noted even for intersections with the same change interval design.

The initial work by Wortman and Matthias gave direction to further field studies in Arizona(3). 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. 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. Generally, the result of extending the duration of the yellow interval was the reduction in the percentage of the vehicles entering on the red signal indication. While the presence of a police vehicle at the site did significantly reduce 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 the Federal Highway Administration, the Texas Transportation Institute undertook a somewhat parallel study of traffic signal change intervals(17). 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. Based on their research, the TTI team recommended the use of a perception and brake reaction time of 1.2 seconds and a deceleration rate of 10.5 feet per second per second. The following four alternative methods for determining the change interval were evaluated:

Method 1A - Continued use of current formula with one perception and brake reaction time and one deceleration rate for all approach speeds.

Method 1B - Continued use of current formula with different perception and brake reaction times and deceleration rates for different approach speeds.

Method 2 - Design change interval based on clearing vehicles.

Method 3 - Design change interval based on the probability stopping or clearing.

Table 1 presents a comparison of the minimum yellow interval using Method 1A for the reaction time and deceleration rates recommended by TTI and those contained in the Institute of Transportation Engineers

TABLE 1

COMPARISON OF TTI METHOD 1A WITH ITE GUIDELINES(17)

	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
Signal Yellow Time t = 1.2 sec d = 10.5 ft/s^2	3.0	3.3	3.6	4.0	4.3	4.7	5.0
ITE Values t = 1.0 sec d = 10.0 ft/s^2	2.8[a]	3.2	3.6	3.9	4.3	4.7	5.0

a: Minimum value considered safe by ITE guidelines is 3.0 seconds although the calculation value is 2.8 seconds.

guidelines. Table 2 presents the resulting values for Method 1B from the TTI work. It should be noted that the values shown in Tables 1 and 2 reflect the yellow time required for the stopping vehicles and not necessarily the total time required for a vehicle to enter and clear the intersection.

Method 2, which was considered by TTI, focuses on the time required for the last through vehicle to enter the intersection after the onset of the yellow interval. Table 3 summarizes the values for the yellow interval based on this method. It should be noted that this method basically yields a uniform yellow interval. This approach to determining the yellow interval would tend to support the suggestion of a uniform yellow by Williams(12).

Method 3 reflects a concept contained in several previous studies(12,18,19) and is based on the probability of stopping given a distance from the intersection and the approach speed. Table 4 indicates the minimum values of the yellow interval for this method and summarizes the values for all of the alternatives considered by TTI. In addition, TTI considered the time for a vehicle to clear the intersection. This aspect of the change interval is discussed later along with the all-red interval.

ITE Technical Committee 4A-16 has also recently been examining the policy for change interval design and calculation(20). At the current time, the recommended practice proposed by that Committee is still undergoing formal review. Basically, the proposed formula reflects the current ITE guidelines along with the inclusion of the approach grade in

TABLE 2

YELLOW TIME FOR TTI METHOD 1B(17)

	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
Perception-Brake Reaction Time	1.5	1.4	1.3	1.2	1.1	1.0	1.0
Deceleration Rate	8.0	8.5	9.0	9.5	10.0	10.5	10.5
Signal Yellow Time	3.8	4.0	4.2	4.3	4.4	4.5	4.8

TABLE 3

YELLOW SIGNAL INTERVALS FOR TTI METHOD 2(17)

	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
85% of Clearing Vehicles	4.0	4.0	4.0	4.0	4.0	4.0	4.0
95% of Clearing Vehicles	4.5	4.5	4.5	4.5	4.5	4.5	4.5

TABLE 4

SUMMARY OF YELLOW INTERVALS FOR TTI ALTERNATIVE METHODS(17)

Alternative Methods	Approach Speeds (mi/h) 25	30	35	40	45	50	55
1A	3.0	3.3	3.6	4.0	4.3	4.7	5.0
1B	3.8	4.0	4.2	4.3	4.4	4.5	4.8
2							
85% Clearing	4.0	4.0	4.0	4.0	4.0	4.0	4.0
95% Clearing	4.5	4.5	4.5	4.5	4.5	4.5	4.5
3							
Probability of Stopping 85%	---	---	5.0	4.7	4.5	4.2	4.2
ITE Values	2.8[a]	3.2	3.6	3.9	4.3	4.7	5.0

a: Minimum value considered safe by ITE guideline is 3.0 seconds although the calculation value is 2.8 seconds.

the calculation of the yellow interval. For this calculation, the following formula is indicated:

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

where y = length of yellow interval in seconds

t = driver perception-reaction time in seconds

v = vehicle approach speed in feet per second

a = deceleration rate in feet per second per second

G = acceleration due to gravity

g = approach grade in percent divided by 100

In addition to the calculation of the yellow interval by the formula, the ITE Committee states the following:

"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."

THE ALL-RED INTERVAL

As has been indicated previously, the determination of the change interval should consider the time required for an approaching vehicle to stop as well as permit vehicles which have legally entered the intersection to clear. The previous discussion of the TTI research and the proposed ITE recommended practice focused on the time required for a vehicle to stop after the onset of the yellow signal indication. This section presents a discussion of the time required to clear the intersection.

Over the years, some traffic engineers employed an all-red interval when the required yellow time exceeded some value, such as 5 or 6 seconds. The intent in this case 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(21) argue that the yellow interval cannot be considered as 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 all-red signal indication. A similar discussion is presented by Butler(22). The proposed ITE policy(20) 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 traverse and clear the intersecting street.

The TTI study(17) proposed a method of determining an all-red interval which 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, then an all-red interval is necessary. The study does note that the consideration of the start delay time should be used with caution, and the legal implications of the local laws and ordinances should be checked prior to using the reduction.

Accident studies of intersections where the all-red interval has been used indicate that accident reductions were found where the interval is utilized. Newby(23) in 1961 reported a 41 percent reduction in injury accidents at intersections after the addition of an all-red period. In a study for the Federal Highway Administration, Benioff et. al.(24) concluded that the provision of an all-red interval resulted in the reduction of accidents. They also suggest that intersections with a right-angle accident rate of greater than 0.8 accidents per million entering vehicles should be considered for the addition of an all red interval. In their review of previous work, TTI(17) cites several other studies by specific jurisdictions that indicate accident reductions with the application of the all-red interval.

Other studies have addressed driver behavior at intersections where the all-red interval was utilized. Wortman and Matthias(1) compared the response times and deceleration rates at intersections with the yellow only change interval with those having a yellow plus an all-red interval. No significant differences were found in terms of response times, and the comparison of the deceleration rates yielded mixed responses. In addition, Ryan and Davis(25) examined the driver entry into the intersection during the red signal indication.

THE EFFECT OF APPROACH GRADES

Parsonson and Santiago(14) as well as the proposed ITE policy(20) have indicated the need for considering the effect of the approach grade on the deceleration rate which is used in the calculation of the change interval. Certainly, grade is considered in the traditional calculations of stopping distance of vehicles.

The research by TTI(17) 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 = deceleration rate in feet per second per second, and

g = percent of grade

This equation reflects the recommended use of 10.5 feet per second per second for level roadway conditions. The TTI study recommends that as a general rule a value of 10.5 can be used for level and upgrade conditions while 10.0 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.(6) as well as the policies which followed all include this assumption. Even the proposed ITE policy(20) as well as some of the alternatives considered in the TTI study continue to be founded on this basis. While this assumption is convenient for computational purposes, there are other theoretical considerations which should be taken into account when evaluating driver and vehicle characteristics.

The review of literature to date basically 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 which 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:

$$a = v^2 / 2x \qquad \text{(Eq. 1)}$$

$$a = 2x / t^2 \qquad \text{(Eq. 2)}$$

$$a = v / t \qquad \text{(Eq. 3)}$$

where a = deceleration rate in feet per second per second,

v = initial approach speed in feet per second,

x = distance traveled while stopping in feet, and

t = time required to stop in seconds.

These equations are valid for a situation where the vehicle comes to a complete stop at the end of the period of deceleration as would be the case at a signalized intersection. For the uniform deceleration situation, Figure 1 depicts the theoretical relationship of vehicle speed, time, and distance. The distance traveled is represented by the area under the curve, and the deceleration rate is shown by the slope of the curve. As shown in Figure 1, the deceleration rate is constant; thus the curve representing the deceleration rate is a straight line. As has been indicated previously, the philosophy related to 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.

In cases involving non-uniform deceleration, the relationship between speed, time, distance, and the deceleration rate breaks down. For example, Figure 2 depicts three cases where the initial speed differs; but the time of deceleration and the distance traveled are the same. The difference is caused by differing deceleration profiles. Figure 3 depicts a similar situation where the initial speed and the time of deceleration are the same; however the distance traveled differs for the different deceleration profiles. Finally, Figure 4 shows a situation where the deceleration times differ even though the initial speed and the distance

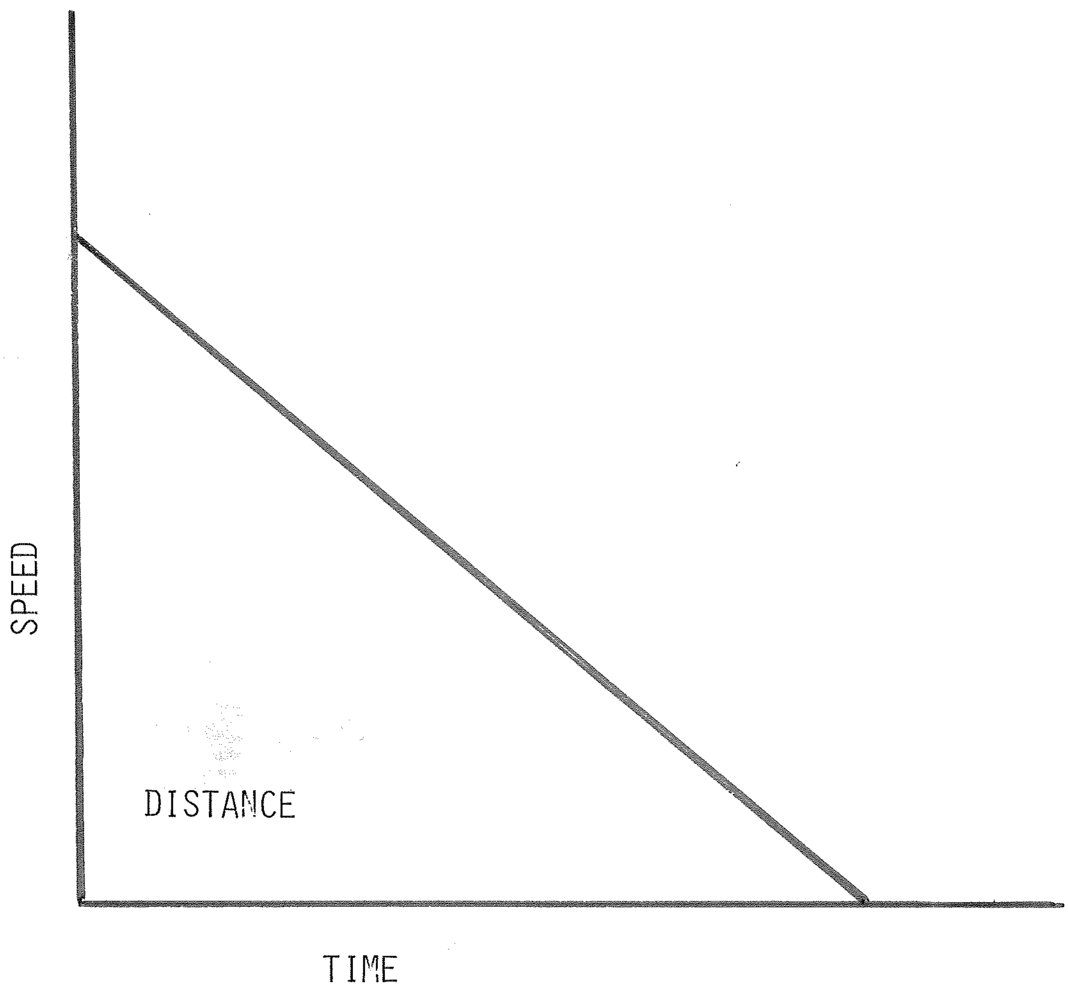

FIGURE 1

THEORETICAL RELATIONSHIP OF VEHICLE SPEED, TIME, AND DISTANCE

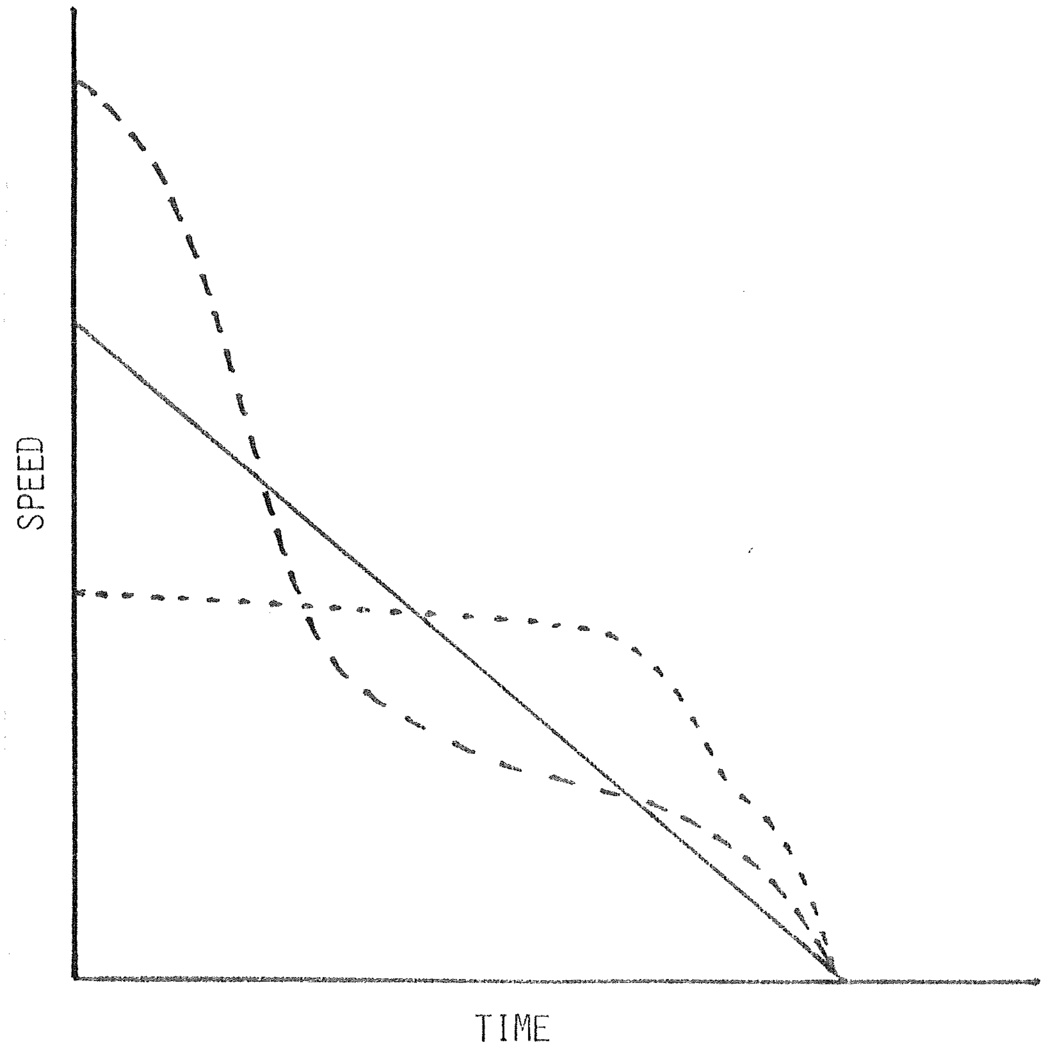

FIGURE 2

DECELERATION PROFILES WITH EQUAL DECELERATION TIME AND DISTANCE

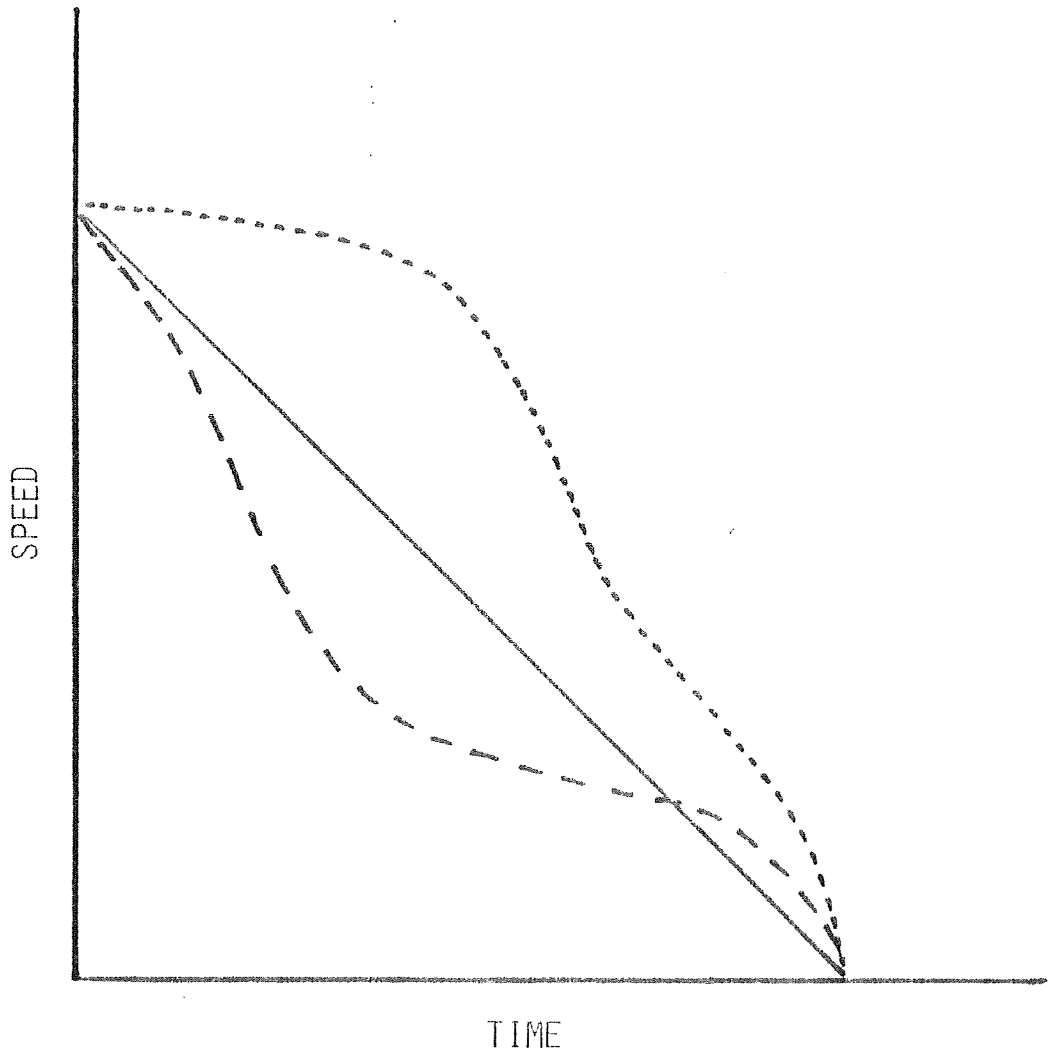

FIGURE 3

DECELERATION PROFILES WITH EQUAL INITIAL SPEED AND DECELERATION TIME

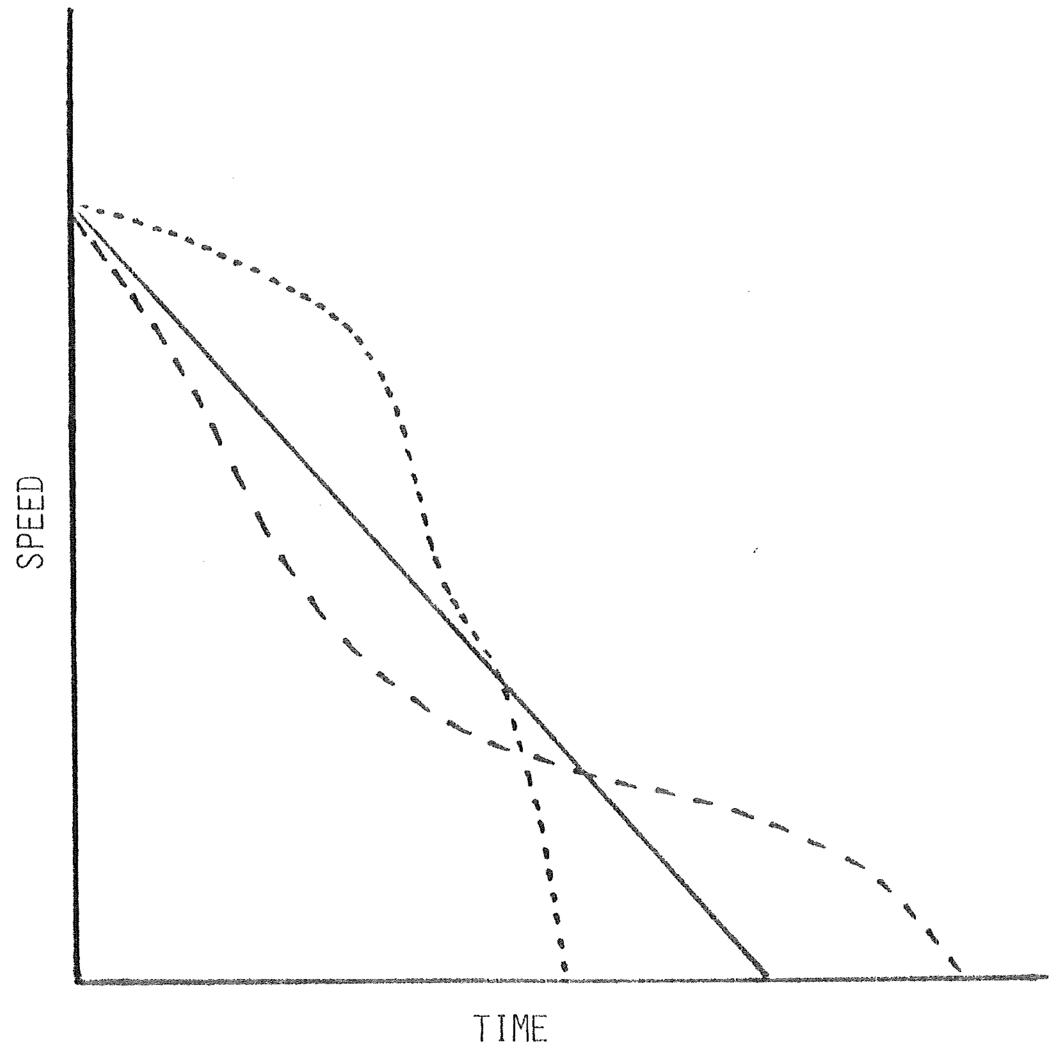

FIGURE 4

DECELERATION PROFILES WITH EQUAL INITIAL SPEED AND DECELERATION DISTANCE traveled are the same. Certainly, more than one of the parameters may vary; but these are shown in this manner for discussion purposes.

It is important to recognize that the equations that were presented earlier do not yield consistent results for situations involving non-uniform deceleration. In fact, the calculated deceleration rates can vary considerably depending on the parameters which are 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 this research project(3) as well as the previous research by Wortman and Matthias(1), 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 field studies undertaken as part of this project, 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 which use (a) the initial approach speed and deceleration distance and (b) the deceleration distance and time. Again, given an approximation of 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 non-uniform 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 which 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 initial deceleration rate and then selected 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.

Figure 5 indicates a plot of the relationship of the Q values with the initial approach speed of the stopping vehicles. 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 initial deceleration rate 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 indicates that there is a reversal in the deceleration profiles at approach speeds of about 48 miles per hour where the Q value is 1.0. This has a dramatic effect on the actual time required for a vehicle to

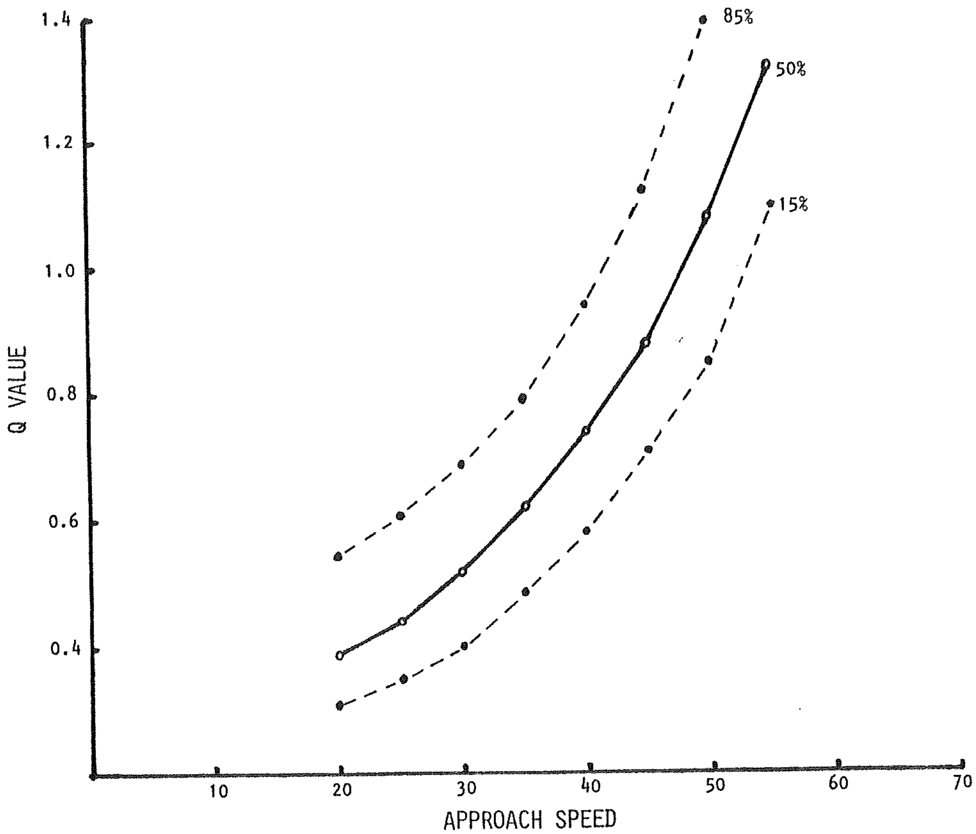

FIGURE 5

THE RELATIONSHIP OF Q VALUES AND APPROACH SPEED stop, and raises questions about the traditional manner in which minimum yellow time calculations have been made.

ANALYSIS OF DRIVER BEHAVIOR

The analysis of driver behavior focuses on (a) driver characteristics given the non-uniform deceleration, (b) driver response times, and (c) assumed average uniform deceleration rates. The latter two topics are pertinent if a traditional approach to the determination of the change interval is used.

DRIVER CHARACTERISTICS WITH NON-UNIFORM DECELERATION

Based on the findings which revealed the non-uniform nature of the deceleration rates selected by the driver and the relationship of the deceleration profile with the initial approach speed, an examination was made to determine how this affects the signal change interval design. Analyses 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(1,3) 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 is simply a surrogate measure, and the real measure of interest is the time distance 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 distance from the intersection at the onset of the yellow interval.

Several analyses involving the time distance from the intersection were undertaken in an attempt to identify the effect of approach speeds and intersection conditions on driver behavior. Figures 6, 7, and 8 show the cumulative percentage curves for the time distance from the intersection for approach speeds of 30, 40, and 50 miles per hour. Based on the field data, the first vehicle to stop at all of the approach speeds occur was approximately 2 seconds from the intersection if the driver had proceeded to the intersection at that approach speed. While 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 various speed ranges indicates that the slope of the curve increases with approach speed. For the field data collection, approximately 350 to 400 feet 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 distance from the intersection that could be recorded for a vehicle decreased with the increase in approach speeds.

For the last vehicle 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

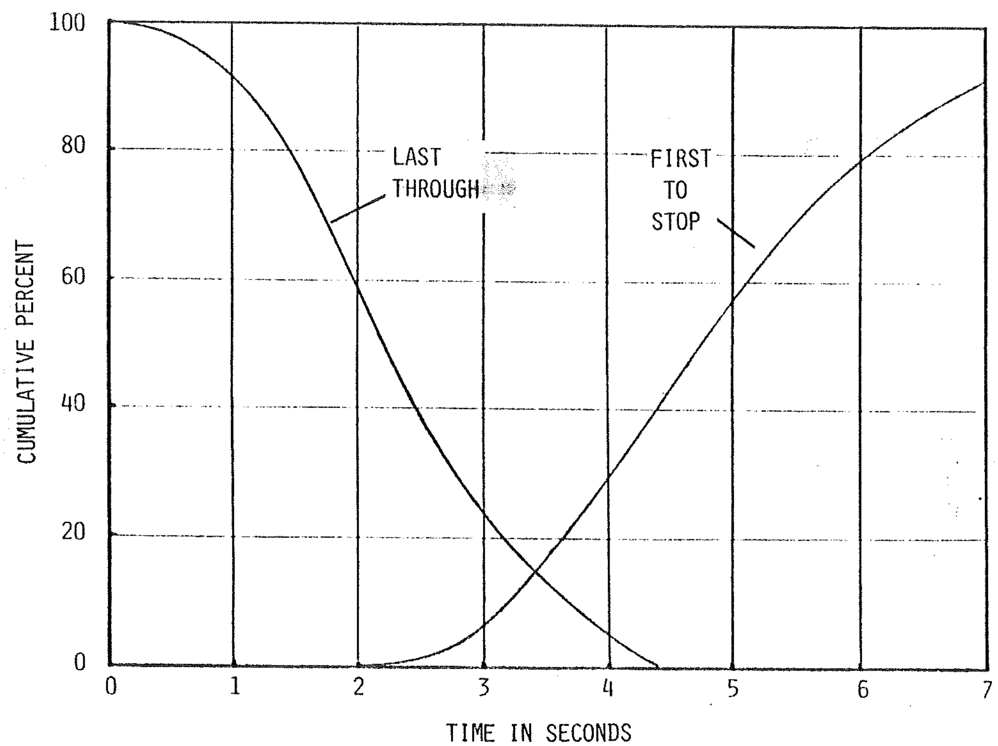

FIGURE 6

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

30 MPH APPROACH SPEED

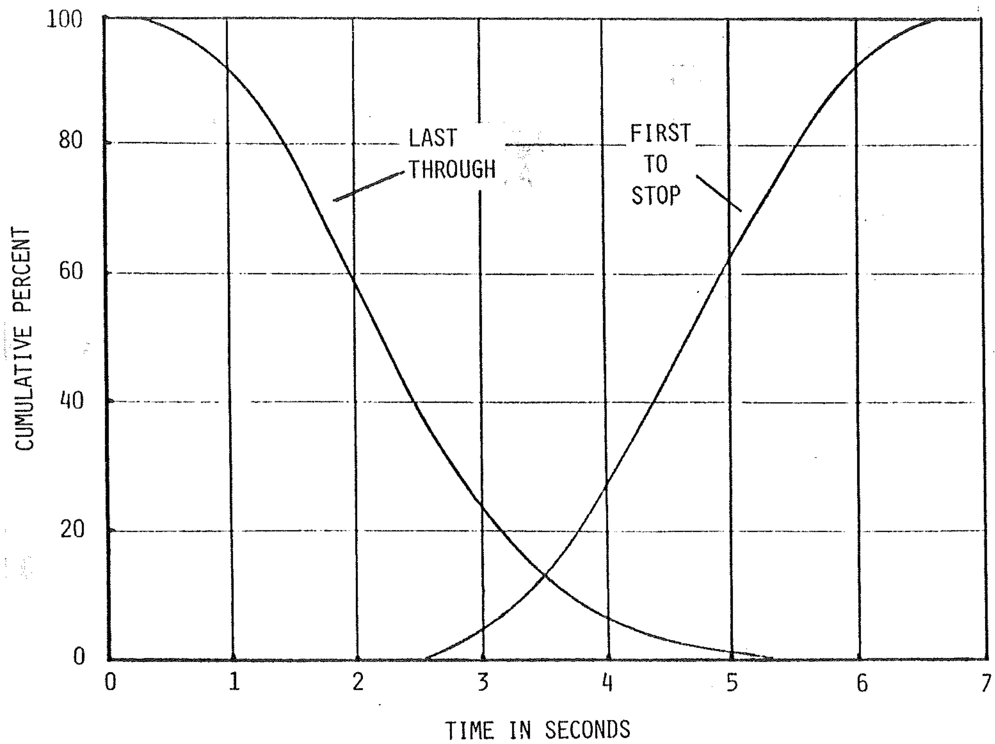

FIGURE 7

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

40 MPH APPROACH SPEED

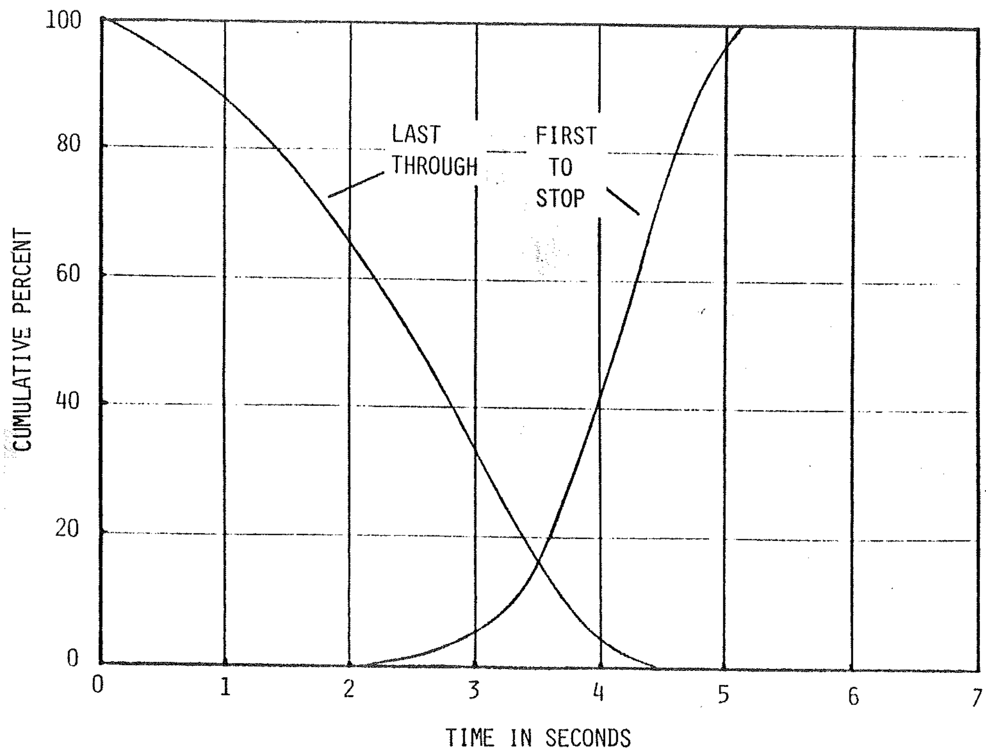

FIGURE 8

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

50 MPH APPROACH SPEED

the approach speed, approximately 95 percent of the last vehicles through the intersection are 4 seconds or less from the intersection. For all of the approach speeds, the 4 second value for the 5 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. Basically, the last vehicle through the intersection is more critical than the first vehicle to stop; thus the determination of the yellow time should be a function of the last vehicle through the intersection.

Also, these curves can be used to define the dilemma zone. For example, the minimum time distance for the first vehicle to stop and the maximum time distance for the last vehicle through the intersection indicate the limits of the dilemma zone. In this range, some vehicles stop while 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 Phase I of this project, data were collected at two intersections where the yellow interval was extended from 3 to 4 seconds. At these two intersections, the change interval also included a 2 second all-red interval. Figure 9 indicates the time distance 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 second value.

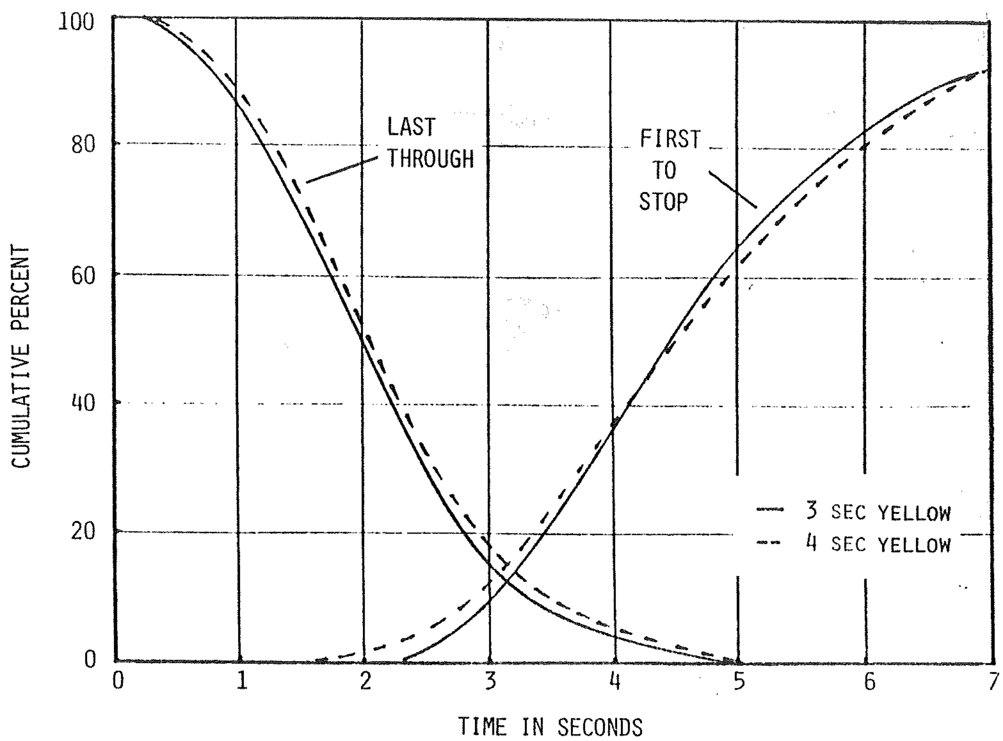

FIGURE 9

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

COMPARISON OF 3 VERSUS 4 SECOND YELLOW INTERVALS

To determine the effect of longer change intervals, data from the earlier study(1) were used. Data were collected at one intersection which had a 5 second yellow plus a 3 second all-red change interval. The time distance curves for this intersection are shown in Figure 10. Again, the critical portion of the curves are similar to those for other intersections. In fact, the data revealed that the last vehicle through the intersection occurred before the end of the 5 second yellow interval. This would tend to refute the theory that drivers enter the intersection later with longer yellow intervals.

Figure 11 depicts the same analysis for one of the intersections at which day and night data were obtained. At this intersection, there was a significant difference in the comparison of the day and night deceleration rates; however the comparison of the time distance 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.

Also as part of Phase I of this project, the influence of enforcement was tested with and without a police vehicle present on the intersection approach. The earlier conclusion regarding this test was that the measures of driver behavior really did not significantly change except the percentage of drivers entering the intersection on the red signal indication did decrease with the police vehicle present. The time distance curves for part of the research as shown in Figure 12 do indicate that the presence of the vehicle did modify the behavior of the drivers.

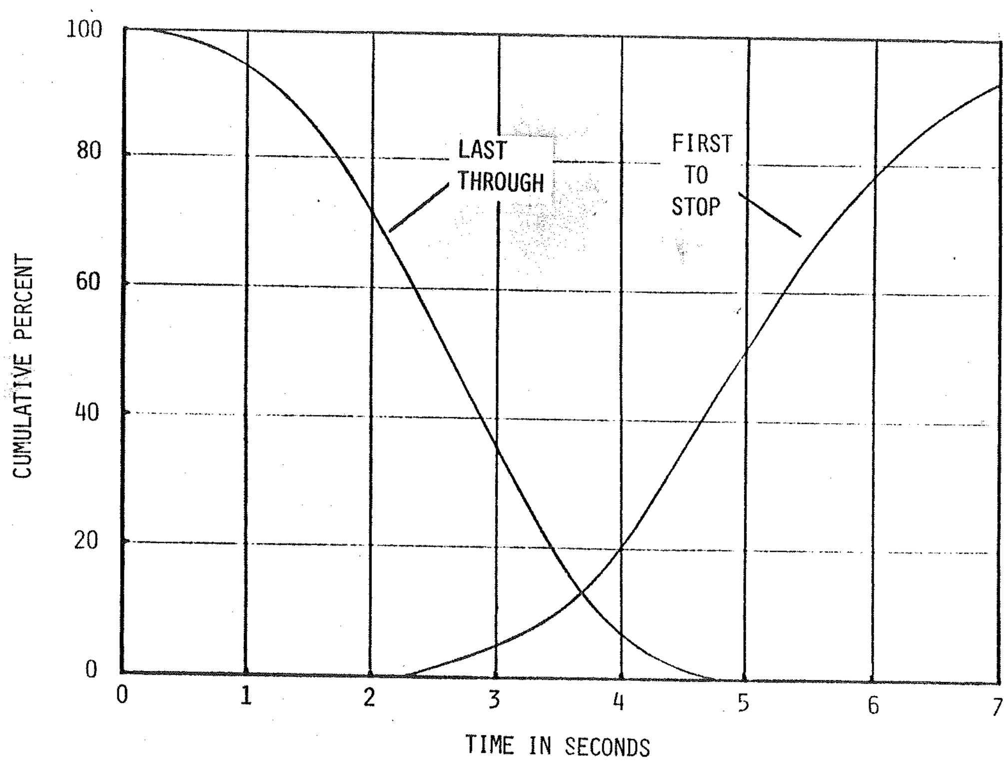

FIGURE 10

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

5 SECOND YELLOW PLUS 3 SECOND ALL-RED CHANGE INTERVAL

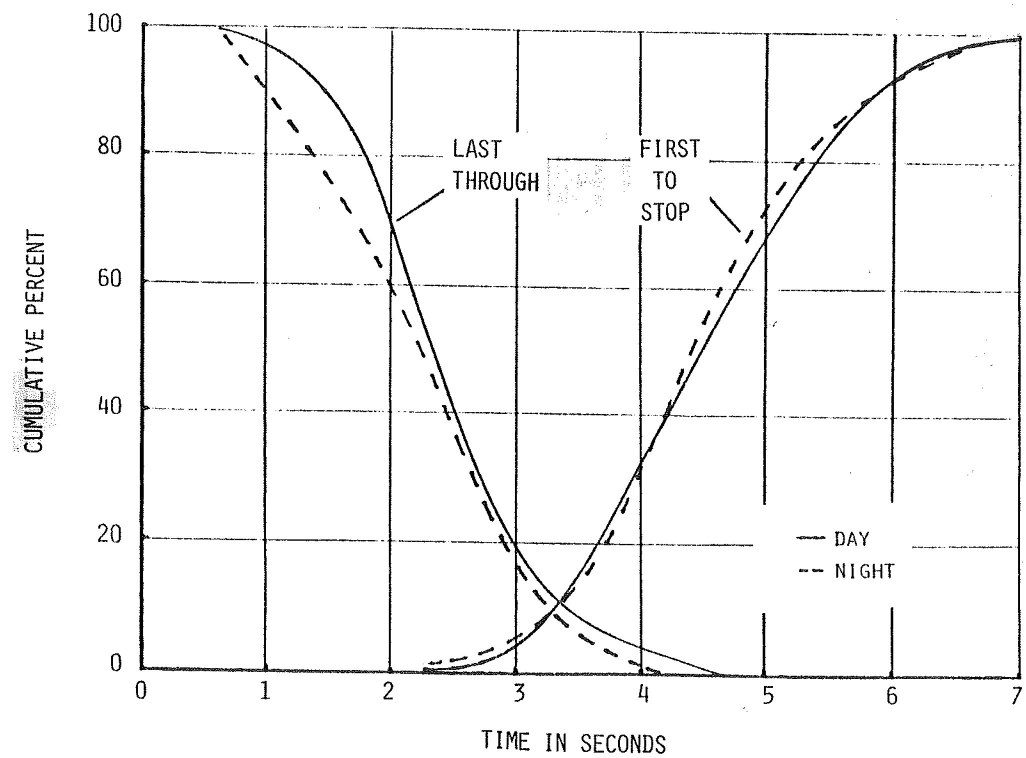

FIGURE 11

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

DAY VERSUS NIGHT COMPARISON

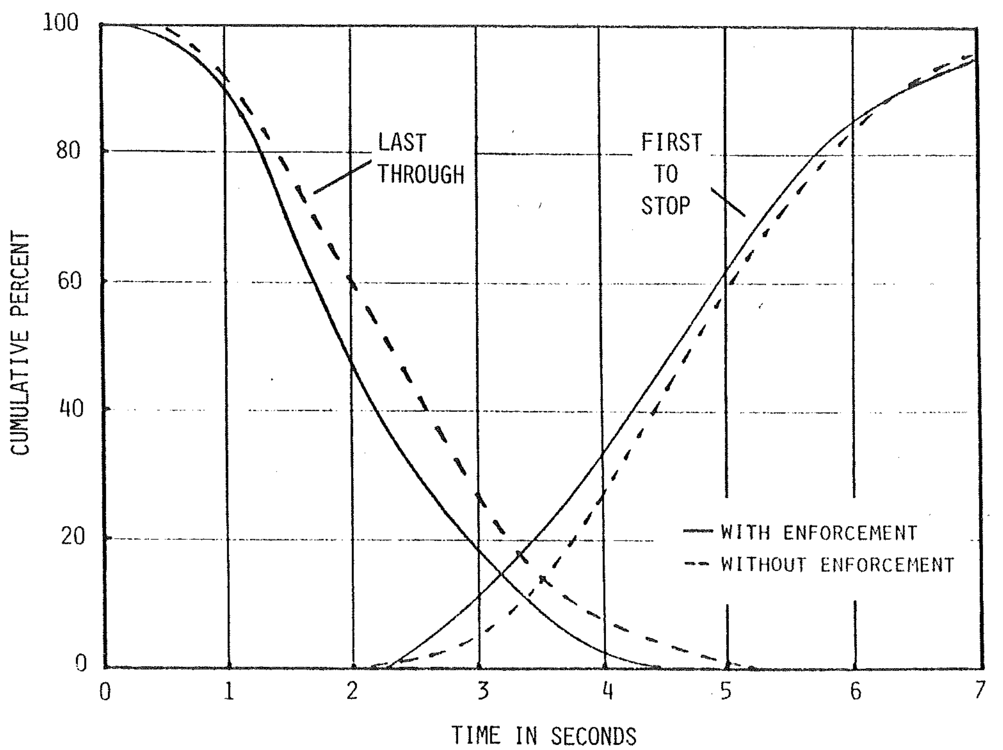

FIGURE 12

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW

INTERVAL ENFORCEMENT COMPARISON

Obviously, this change was temporary because the subsequent studies at that intersection did show a return to the pattern represented by the without enforcement condition.

It is interesting to note that the approach grade had little affect on the time distance values for the approaching vehicles. Figure 13 represents the time distance curves for the Swan Road approach which had an approach downgrade of 2.0 percent. Actually, the curve is very similar to Figure 8 because this intersection had the highest approach speeds in the data sample.

All of the data included in the analysis was taken at intersections in the Phoenix and Tucson metropolitan areas; thus there was concern whether this represented driver behavior in other parts of the United States. While the detailed data from the TTI study(17) were not available for analysis, Figure 14 is taken from the final report for that study and indicates the driver's decision to stop or go by time from the stop line. The 95 percentile time value for all of the approach speeds is about 4 to 4.5 seconds. 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.

DRIVER RESPONSE TIME

Both of the Arizona studies of the change interval measured the response time of the driver to the onset of the yellow signal interval.

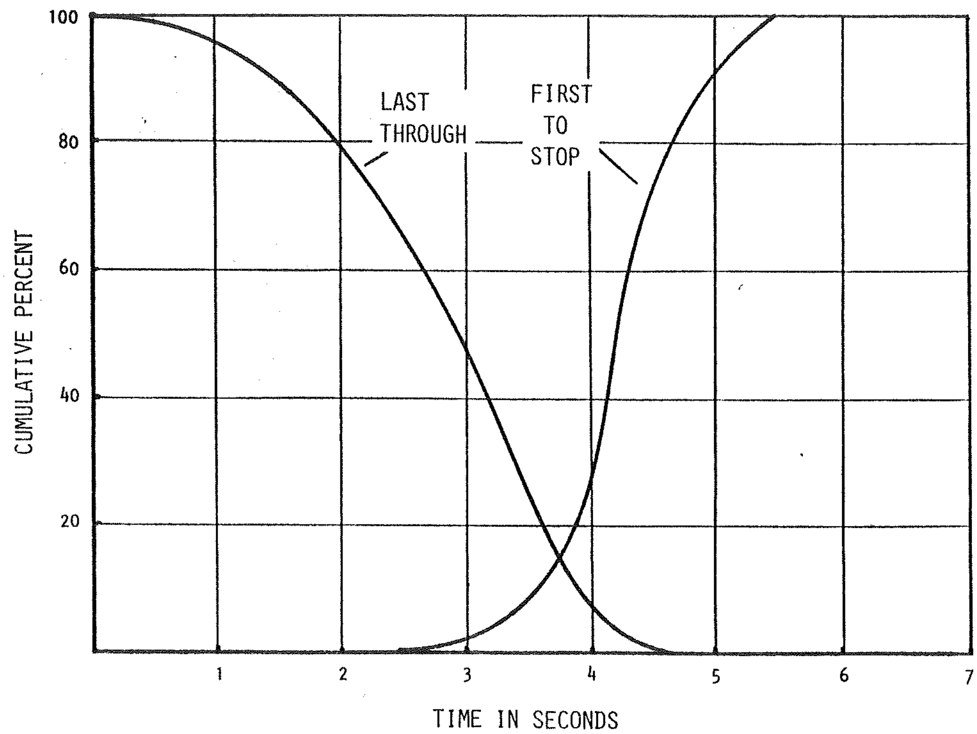

FIGURE 13

TIME DISTANCE FROM THE INTERSECTION AT THE ONSET OF THE YELLOW INTERVAL

SWAN ROAD APPROACH WITH A 2 PERCENT DOWNGRADE

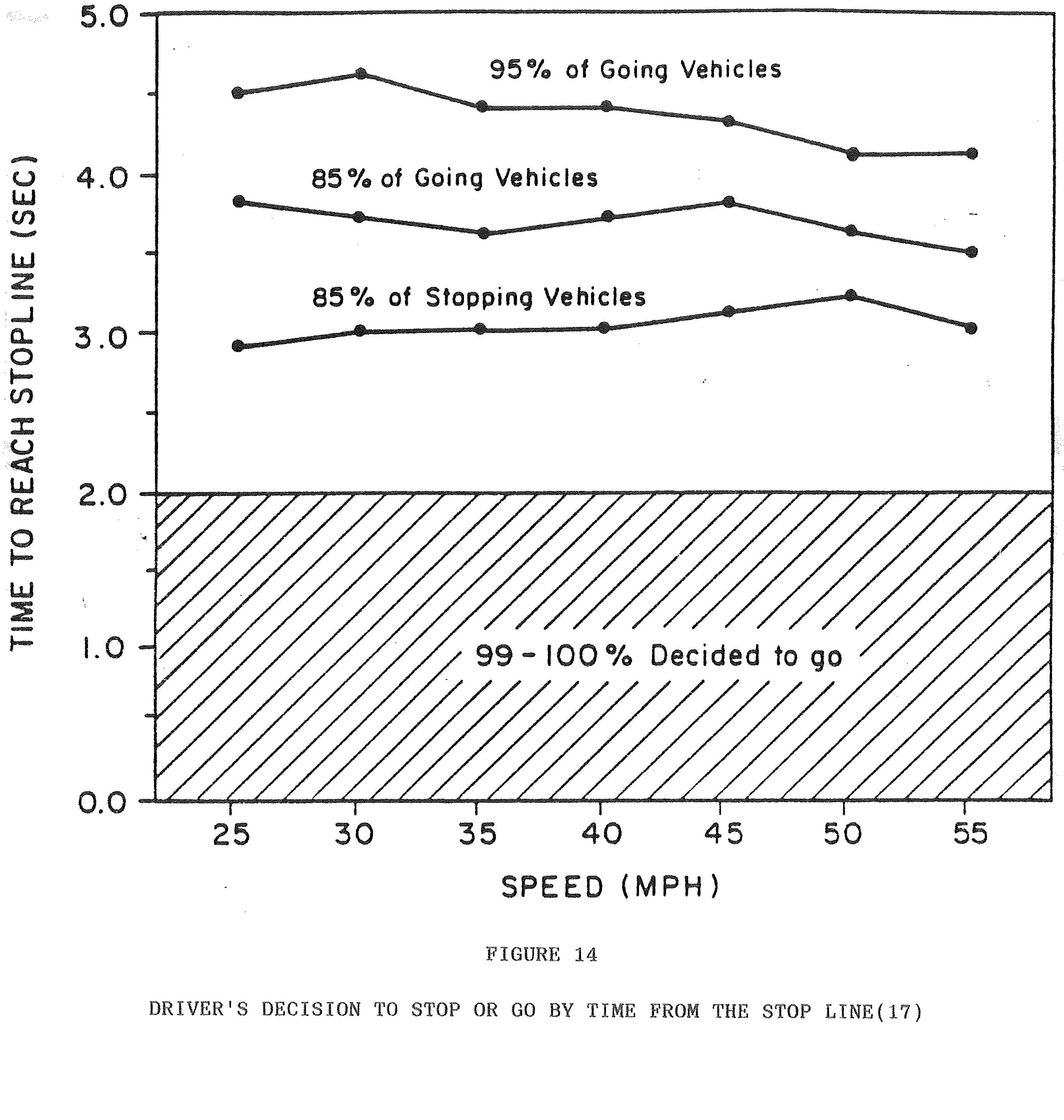

FIGURE 14

DRIVER'S DECISION TO STOP OR GO BY TIME FROM THE STOP LINE(17)

The response time was measured as the time between the onset of the yellow interval and the illumination of the brake lights on the vehicle. Table 5 summarizes the response times that were found in the initial study by Wortman and Matthias(1), and Table 6 provides the same type of information based on data that were collected as part of this project(3). As noted in Table 5, the overall mean response time was 1.30 seconds. In the current study, the overall response time for the base conditions for all of the intersections yielded the same value.

The TTI research(17) also found the same mean value. While the TTI study found the driver's response time was affected by distance to the intersection at the onset of the yellow interval, approach speed, and time available to reach the stop line after the onset of the yellow interval, an analysis of the data from the Arizona work did not reveal these relationships.

DECELERATION RATES

Tables 7 and 8 indicate the observed deceleration rates from the field studies in Arizona. It can be noted that there is considerable variation in the mean values for the individual intersections. These values were calculated using the distance traveled during deceleration and the deceleration time. As has been indicated previously, the value of the deceleration rate is sensitive to the equation that is used to calculate the value when non-uniform deceleration occurs. The variation which is shown in the Tables 7 and 8 can be attributed to the effect of the non-uniform deceleration.

TABLE 5

DRIVER RESPONSE TIMES - WORTMAN AND MATTHIAS STUDY(1)

Intersection Approach	Mean Time (sec)	Standard Deviation	85% Time (sec)
University Drive	1.28	0.82	2.0
Southern Ave. (Day)	1.49	0.62	1.9
Southern Ave. (Night)	1.43	0.73	2.0
U. S. 60	1.38	0.60	2.1
First Avenue	1.24	0.51	1.8
Sixth Street	1.55	0.70	2.0
Broadway Blvd. (Day)	1.16	0.48	1.5
Broadway Blvd. (Night)	1.09	0.44	1.5
All Approaches	1.30	0.60	1.8

TABLE 6

DRIVER RESPONSE TIMES - CURRENT RESEARCH STUDY

Intersection Approach	Mean Time (sec)	Standard Deviation	85% Time (sec)
FIRST AVENUE			
Base Condition	1.3	0.5	1.7
With Police Car	1.4	0.9	2.0
Before Extension	1.2	0.5	1.5
After Extension (DRY)	1.1	0.4	1.5
After Extension (WET)	1.3	0.5	1.9
WILMOT ROAD			
Before Extension	1.4	0.8	1.9
After Extension	1.3	0.7	1.9
SWAN ROAD	1.0	0.3	1.4
ORACLE ROAD	1.1	0.4	1.4
SPEEDWAY BOULEVARD	1.4	0.9	1.9

TABLE 7

DECELERATION RATES - WORTMAN AND MATTHIAS STUDY(1)

Intersection Approach	Mean Rate (fps^2)	Standard Deviation	85% Rate (fps^2)
University Drive	7.0	3.8	11.5
Southern Ave. (Day)	10.7	3.0	13.9
Southern Ave. (Night)	11.6	2.6	14.8
U. S. 60	11.8	3.4	15.8
First Avenue	12.4	3.5	16.1
Sixth Street	13.9	4.5	18.2
Broadway Blvd. (Day)	12.8	4.1	17.2
Broadway Blvd. (Night)	9.7	3.0	12.5
All Approaches	11.6		

TABLE 8

DECELERATION RATES - CURRENT RESEARCH STUDY

Intersection Approach	Mean Rate (fps^2)	Standard Deviation	85% Rate (fps^2)
FIRST AVENUE			
Base Condition	11.9	3.2	14.8
With Police Car	12.5	3.8	17.0
Before Extension	12.9	4.2	17.7
After Extension (DRY)	12.1	3.2	15.9
After Extension (WET)	11.0	3.3	13.1
WILMOT ROAD			
Before Extension	13.2	4.7	17.6
After Extension	12.0	4.3	16.8
SWAN ROAD	8.3	2.1	10.8
ORACLE ROAD	10.1	3.2	13.2
SPEEDWAY BOULEVARD	12.6	4.0	16.4

In an effort to determine an estimate of the uniform deceleration rates which are selected by drivers, an analysis was undertaken using the data samples where the deceleration profile approximated a uniform deceleration case. This was accomplished by again using the three equations for the calculation of the deceleration rate. For a stopping vehicle, the deceleration rates calculated by each of the three equations were compared. If the rates were approximately equal, the sample was considered to conform to the uniform deceleration case.

Data from five intersections with differing approach grades were included in the analysis; thus it was possible to evaluate the influence of the approach grade on the deceleration rate that is used by the driver. The following indicates the mean deceleration rate and approach grade for each of the locations that were included in the analysis:

	mean deceleration rate	approach grade
Roger Road	10.6	+0.8%
Wilmot Road	11.1	+1.0%
Speedway Boulevard	10.3	-0.5%
Swan Road	10.1	-2.0%
Oracle Road	9.3	-2.6%

It should be recognized that the values for the mean deceleration rates differ from those shown previously, because these data include only those vehicles that were considered to decelerate using a constant or uniform rate.

For this data set, a regression analysis resulted in the following equation which has an "r" value of 0.92:

$$a = 10.5 \pm .38g$$

where a = deceleration rate in feet per second per second, and

g = approach grade in percent with the downgrade being negative.

This equation provides two results that are of considerable significance. First, the equation results in a deceleration rate value of 10.5 feet per second per second for a level intersection approach. This is the same value found in the TTI study, and it is the deceleration rate value that is contained in the recommended policy resulting from that research(17). The two research efforts, therefore, support each other in terms of the uniform deceleration rate even though the data bases are from different geographic areas. Also, the findings generally validate the value of 10 feet per second per second which been adopted in most of the current policies for the determination of the change interval.

The second significant aspect of the regression equation is related to the effect of grade. In the current ADOT policy(16) as well as the proposed ITE policy(20), the effect of grade is incorporated into the equations on a theoretical basis. On a theoretical basis, the coefficient associated with the approach grade in the regression equation would be 0.322. The empirical value of 0.38 tends to validate the theory that has been proposed in terms of the effort of grades.

It must be recognized that the empirical value is based on a rather limited sample size, and the difference with the theoretical value could be attributed to the limited sample. Also, it must be emphasized that the field study samples included intersections with a rather limited range of intersection approach grades. In the course of the field studies, attempts were made to find intersection approaches with steeper downgrades. Such intersection approaches, however, presented data collection problems in that drivers frequently applied the brakes while on the downgrade section even when the traffic signal was green. For this reason, it was extremely difficult to determine when a driver began brake application after the onset of the yellow interval.

A comparison of the effect of grade found in this study with that found in the TTI study(17) does reveal a difference. The TTI study reported a coefficient of 0.075 which indicates that the approach grade had a much less influence based on their data set.

CONCLUSIONS

In the course of this project, field studies were undertaken which served to document driver behavior in relation to the traffic signal change interval at intersections in Arizona. This information together with the data collected as part of the earlier Arizona change interval research provided one of the most extensive data bases for analyzing traffic characteristics and driver behavior which are pertinent to the design of the signal change interval. As is the case with many research efforts or traffic studies, the total number of intersections included in the field studies is somewhat limited. The data base, however, has a greater breadth and depth than that used in most of the previous work. Because of the depth and breadth of the data base, it was possible to develop insights relative to the change interval that were not previously possible. This understanding should provide valuable input in any future development of traffic signal change interval policies.

In recent years, several parallel research studies have been undertaken. It would appear that these studies have yielded results which are quite similar even though the study sites were located in different parts of the United States. This would indicate that the findings of the Arizona research are representative portrayals of driver behavior; however further comparative analysis of the results from each of these studies will be required before drawing final conclusions.

Traditionally, evaluations of driver behavior have focused on parameters such as deceleration rates, driver reaction or response times, and distance from the intersection at the onset of the yellow interval. When these traditional measures of driver behavior were compared for the intersections included in the research, there were unexplained variations in the results. While this research made comparisons using the traditional measures, the analysis focused on an evaluation of driver behavior based on the time distance from the intersection at the onset of the yellow interval. The time distance parameter was selected because it is more meaningful in terms of the determination of the change interval. In fact, measures such as the deceleration rate and response time are used simply to calculate a time value for the change interval.

Based on analyses of the data from the field studies in Arizona, the following conclusions can be drawn:

1. The majority of the first vehicles to stop after the onset of the yellow interval had deceleration profiles that reflected deceleration rates which were not constant or uniform. In fact, only 31 percent of the vehicles had deceleration profiles that approximated a uniform and constant deceleration rate. Furthermore, the deceleration profile was a function of the approach speed with higher initial deceleration rates being associated with higher approach speeds. This finding raises questions about the continued use of the theoretical relationships that are the basis for the traditional approach to determining the signal change interval.

2. For change interval determination, the behavior of the last vehicle through the intersection after the onset of the yellow interval is more critical than the first vehicle to stop. In essence, drivers chose to

continue through the intersection at time distance values greater than those associated with the stopping vehicles.

3. Driver behavior was analyzed given approach speeds of 30, 40, and 50 mph. It was found that approach speed had little or no effect on driver behavior relative to decisions to stop or proceed through the intersection. This finding can be explained by the fact that drivers did not use constant or uniform deceleration rates.

4. The effect of the yellow interval was tested by obtaining field data at sites with various yellow interval durations as well as extending the yellow interval at two intersections. Again, the duration of the yellow interval had little or no effect on driver decisions.

5. While the literature suggests that approach grades need to be considered in determining the yellow interval, the analysis of the intersections included in this study did not reveal that approach grades resulted in differences in driver behavior.

6. A comparative analysis of day/night behavior in terms of the traditional parameters indicated that the nighttime conditions might result in different behavior patterns. The analysis of driver decisions based on the time distance from the intersection revealed that the day/night behavior was not different.

7. In the test of enforcement, the presence of a police vehicle at the site did reduce the time after the onset of the yellow that the last vehicle entered the intersection. This reduction was temporary and drivers returned to previous behavior patterns when the police vehicle was not at the site.

8. As was the case with the duration of the yellow interval, the use of an all-red interval had no influence on driver behavior at the intersections which were studied.

In summary, the research suggests that the traffic signal change interval design could be based on a uniform yellow interval. The research indicates that factors such as approach speeds, approach grades, and the duration of the yellow interval or even the duration of the change interval have little or no influence on driver behavior relative to decisions to stop or continue through the intersection.

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APPENDIX

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
Traffic Engineering Section

PGP-4-4B-3-0
October, 1980

SUBJECT: TRAFFIC SIGNAL CLEARANCE INTERVAL

Paragraph 1. Purpose
2. Definitions
3. References
4. Policy Statement
5. Approvals
6. Design Standard Reference

1. PURPOSE

To establish a uniform method of determining the length of the clearance interval for traffic signals on the state highway system.

2. DEFINITIONS

a. Traffic Signal Clearance Interval - A portion of the cycle following the green interval which may be a yellow interval or a combination of a yellow interval and an all-red interval.

b. Yellow Clearance Interval (Vehicle Change Interval) - A steady yellow indication to warn traffic of an impending change in the right-of-way assignment.

c. All-Red Clearance Interval - A steady red indication between the yellow interval and the start of the green interval for the opposing traffic.

3. REFERENCES

MUTCD 4B-5 Meaning of Signal Indications
4B-15 Vehicle Change Interval

Uniform Vehicle Code -- Section 11-202

ARS-28-645 Traffic Control Signal Legend

4. <u>POLICY STATEMENT</u>

The purpose of the clearance interval is twofold:

1. To advise the motorist that the red interval is about to commence and permit the motorist to come to a safe stop, or;

2. To allow vehicles that have legally entered the intersection sufficient time to clear the point of conflict prior to the release of opposing pedestrians or vehicles.

The laws of physics make it impossible for all vehicles to stop at the onset of the yellow interval, therefore, it is necessary to set the length of the yellow interval so that the driver can react and safely decelerate and come to a stop before entering the intersection, or to clear the point of conflict before the opposing traffic is released. Experience has shown that a perception-reaction time of 1 second is acceptable. Also, that deceleration rates of 8 and 12 feet per second, per second are the lower and upper limits for establishing yellow clearance intervals. Typically drivers in larger urban cities will accept higher rates of deceleration than will drivers of smaller towns and rural highways. The length of the yellow clearance interval will be established within the parameters described above based on engineering judgment; however, the clearance interval should normally be established using a deceleration rate of 10 feet per second, per second.

When the yellow clearance interval exceeds 6 seconds due to the approach speed and/or the intersection width an <u>all-red</u> clearance interval should be used. However the <u>yellow</u> clearance interval should never be less than the sum of the reaction time and the deceleration time for the approach speed selected or 3 seconds, whichever is larger.

If the approach to the traffic signal is on a downgrade of -1% to -10%, add 3% per 1% of grade to the minimum yellow time, e.g., grade=-5%, speed=40 mph, minimum yellow=3.9 sec., add 0.6 sec. to the minimum yellow 3.9+0.6=4.5 sec. For downgrades over 10%, the clearance interval should be calculated using the following formula :

$$Y + AR = t_1 + \frac{1.47V}{(2a \pm 64.4g)} + \frac{W + L}{1.47V} \quad \text{for V in MPH}$$

When the approach traffic contains a significant number of slow vehicles, i.e., heavily loaded trucks, the clearance interval should be calculated for the 15th percentile speed and if larger than the the minimum clearance interval in Table 2, it should be used.

5. APPROVALS

The length of the yellow clearance interval shall be approved by the Assistant State Engineer, Traffic Engineering Section.

6. DESIGN STANDARD REFERENCE

See Tables 1, 2, and 3.

clearance interval should be calculated for the 15th percentile speed and if larger than the the minimum clearance interval in Table 2, it should be used.

5. APPROVALS

The length of the yellow clearance interval shall be approved by the Assistant State Engineer, Traffic Engineering Section.

6. DESIGN STANDARD REFERENCE

See Tables 1, 2, and 3.

TABLE 1

APPROACH SPEED MPH	MINIMUM YELLOW TIME (1) SECONDS	MINIMUM CLEARANCE INTERVAL SECONDS (2) W IN FEET						
		40	60	80	100	120	140	160
25	3.2	4.8	5.4	5.9	6.5	6.9	7.5	8.0
30	3.8	5.1	5.6	6.0	6.5	6.9	7.4	7.8
35	4.2	5.3	5.7	6.1	6.5	6.9	7.3	7.6
40	4.7	5.7	6.0	6.3	6.7	7.0	7.4	7.7
45	5.1	6.0	6.3	6.6	6.9	7.2	7.5	7.8
50	5.6	6.4	6.6	6.9	7.2	7.5	7.7	8.0
55	6.0	6.7	7.0	7.2	7.4	7.7	7.9	8.2

(1) Based on a deceleration rate of 8 $ft./sec.^2$ and a reaction time of 1 sec.

(2) Clearance Interval = Reaction Time ($t_1=1$) +

Deceleration Time $\left(t_2 = \frac{1.47V}{2a}\right)$ +

Intersection Clearance Time $\left(t_3 = \frac{W+L}{1.47V}\right)$

V = Approach speed in MPH

a = Deceleration rate in $ft./sec.^2$

W = Intersection width in ft.

L = 17 feet

TABLE 2

APPROACH SPEED MPH	MINIMUM YELLOW TIME (1) SECONDS	MINIMUM CLEARANCE INTERVAL SECONDS (2) W IN FEET						
		40	60	80	100	120	140	160
25	3.0	4.4	4.9	5.4	6.0	6.5	7.1	7.6
30	3.2	4.5	5.0	5.4	5.9	6.3	6.7	7.2
35	3.6	4.7	5.1	5.5	5.9	6.3	6.7	7.0
40	3.9	4.9	5.2	5.6	5.9	6.2	6.6	6.9
45	4.3	5.2	5.5	5.8	6.1	6.4	6.7	7.0
50	4.7	5.5	5.8	6.0	6.3	6.6	6.8	7.1
55	5.0	5.7	6.0	6.2	6.4	6.7	6.9	7.2

(1) Based on a deceleration rate of 10 ft/sec.^{2} and a reaction time of 1 sec.

(2) Clearance Interval=Reaction Time ($t_1=1$)+Deceleration Time ($t_2=\frac{1.47V}{2a}$)+Intersection Clearance Time ($t_3=\frac{W+L}{1.47V}$).

V = Approach Speed in MPH

a = Deceleration Rate in ft/sec.^{2}

W = Intersection Width in ft.

L = 17 feet

TABLE 3

APPROACH SPEED MPH	MINIMUM YELLOW TIME (1) SECONDS	MINIMUM CLEARANCE INTERVAL SECONDS (2) W IN FEET						
		40	60	87	100	120	140	160
25	3.0	4.0	4.6	5.1	5.7	6.2	6.8	7.3
30	3.0	4.1	4.6	5.0	5.5	5.9	6.4	6.8
35	3.1	4.2	4.6	5.0	5.4	5.8	6.2	6.6
40	3.4	4.4	4.7	5.0	5.4	5.7	6.1	6.4
45	3.8	4.7	5.0	5.3	5.6	5.9	6.2	6.5
50	4.0	4.8	5.0	5.3	5.6	5.9	6.1	6.4
55	4.4	5.1	5.4	5.6	5.8	6.1	6.3	6.6

(1) Based on a deceleration rate of 12 ft./sec,^{2} and a reaction time of 1 sec.

(2) Clearance interval = Reaction Time (t_1=1) +

Deceleration Time ($t_2 = \frac{1.47V}{2a}$) +

Intersection Clearance Time ($t_3 = \frac{W+L}{1.47V}$

V = Approach speed in MPH

a = Deceleration rate in ft./sec.^{2}

W = Intersection width in ft.

L = 17 feet