Timing Traffic Signal Change Intervals Based on Driver Behavior

MYUNG-SOON CHANG, CARROLL J. MESSER, and ALBERTO J. SANTIAGO

ABSTRACT

Driver behavior to traffic signal change intervals (yellow plus all-red) was evaluated by using the data collected from timelapse cameras at seven sites. In particular, signal change interval timing was examined as a function of driver response characteristics involving speed, distance, and time to reach the stop line. Driver-selected yellow response time and deceleration rates were analyzed. New perception and brake reaction time of 1.2 sec with a 10.5 ft/sec^2 of deceleration rate for level grade is suggested. The potential use of a constant yellow interval of 4.5 sec is discussed. In addition, a new method is presented to determine an all-red interval.

The yellow signal indication is a warning of impending loss of right-of-way to the traffic receiving the previous green phase. On seeing the yellow onset, drivers must decide whether to stop or continue through the intersection.

INTRODUCTION

An analysis of physical laws, empirical evidence, and personal driving experience suggests that drivers' behavior in this situation appears to be affected by vehicle speed, position, and physical characteristics as well as other geometric, environmental, and possibly traffic control factors.

If the driver fails to respond safely, a major right angle collision at the intersection is possible and if the driver is startled or overreacts, a hazardous rear-end collision is possible. Because of the complexity of the driver-vehicle-environment control system involved and the potential severe consequences of system failure (a fatal accident), the design of the traffic signal change intervals (yellow time plus any following all-red interval) should be optimized, based on the best understanding of the engineering factors involved. The magnitude of the problem requires that traffic engineers do no less.

Legal Meaning of Change Interval

In 1962, the Uniform Vehicle Code (1) was modified to allow a vehicle to legally enter the intersection on the yellow and to legally clear the intersection when the red signal is displayed. This can be labeled a "permissive rule" in contrast to a "restrictive rule" that required vehicles to clear the intersection before the end of the yellow signal.

Although all states have not adopted the modified Uniform Vehicle Code meaning for the yellow signal and there is a mixture of restrictive and permissive rules across the nation, Bissel and Warren (2) contend that all states operationally allow the intersection clearance to occur during the beginning of the red. Further, a recent survey by Benioff and others (3) has indicated that the procedure used for selecting the change interval was statistically independent of the state law regarding the meaning of the yellow indication.

Signal Change Interval Design

On observing the yellow onset, drivers approaching an intersection are faced with the choice of either stopping the vehicle before entering the intersection or continuing through the intersection. Although several methods and ranges of change interval have been suggested (4), signal change interval design is more frequently based on the equation:

$$Y + AR = t + (v/2d) + [(l + w)/v] \quad (1)$$

where

- $Y + AR$ = duration of the change interval (yellow plus all-red) (sec),
- t = perception and brake reaction time of the driver (sec),
- v = approach speed (ft/sec),
- d = deceleration rate (ft/sec^2),
- w = width of intersection (ft), and
- l = length of vehicle (ft).

Note: Equation 1 was developed by Gazis and others (5) by using the following modeling formulation: A car approaching an intersection is at distance x from the intersection at the yellow onset. If the driver is to stop before entering the intersection, it can be expressed as

$$(x - vt) \geq v^2/2d \quad (2)$$

and if the driver is to clear the intersection completely without acceleration before the green cycle appears on the other street, it can be expressed as

$$(x + w + l) \leq v(Y + AR) \quad (3)$$

Assuming the equality, Equation 2 defines a stopping distance (x_s) as

$$x_s = vt + v^2/2d \quad (4)$$

and Equation 3 defines the clearing distance (x_c) as

$$x_c = v(Y + AR) - (w + l) \quad (5)$$

If $x_c > x_s$, and a driver is positioned between x_s and x_c such that $x_s < x < x_c$, then the driver can either stop or clear the intersection (called the nondilemma zone). However, if $x_c < x_s$ and a driver is positioned between x_c and x_s such that $x_c < x < x_s$, then he will be in a position where he can neither stop safely nor proceed through the intersection completely (called the dilemma zone). Therefore, the minimum change interval satisfying the safe execution of either one of the alternatives (stopping or going through the intersection without acceleration) corresponds to $x_c = x_s$. Then,

$$(Y + AR)v - (w + l) = vt + (v^2/2d) \quad (6)$$

By dividing both sides by v, however,

$$Y + AR = t + (v/2d) + [(w + l)/v] \quad (7)$$

Study Objectives

The objectives of this study are to (a) develop a comprehensive understanding of drivers' responses to the change intervals, (b) examine change interval timing as a function of driver behavior, and (c) quantify the values of the variables associated with driver reaction time and deceleration rates as applied in the Equation 1 from field studies.

DATA COLLECTION

Two timelapse cameras were used to collect data for each approach at an intersection to reduce the potential reading error in distance near the intersection when employing only one camera on an approach. The detailed description of the data collection method along with the definition of sample vehicles is found elsewhere (6,7,8).

Seven intersections (three in Virginia and four in Texas) were studied during the summer of 1983. The geometric and traffic control characteristics at the seven intersections studied are presented in Table 1. Intersections observed for this study included a variety of combinations in intersection width, controller type, grade, and change interval. Passenger cars and through vehicles approaching at speeds higher than 20 mph (composed of 1,035 clearing and 579 stopping vehicles) were collected during operating conditions that included day and night, dry and wet pavement, and peak and offpeak periods.

The list of data collected to evaluate the traffic signal change interval design and driver's response is as follows:

1. The distance from intersection and speed of approach vehicle at the onset of yellow and the driver's decision to continue or to stop,
2. The time and distance at which brakes were applied,
3. The time and distance when a vehicle stopped,
4. The time when a vehicle entered and cleared an intersection,
5. The time when a vehicle in queue started moving, and
6. The type and directional movement of a vehicle.

DATA REDUCTION

To convert film distance to roadway distance in data reduction, four roadway reference points (RRPs), three of which were not on a straight line, were established during data collection.

Establishment of Film/Roadway Relationship

Exposed film was loaded into a TIMELAPSE Model 3420 Data Analyzer Projector. Then, x and y coordinates of those field reference objects could be read with a convenient scale. The corresponding roadway coordinates of these four film reference points were already measured in the field and were available. By using the basic characteristics of photogrammetry (9), the relationship between film and roadway plane can be developed. A computer program modified from that of Bleyl (10) was developed to convert roadway coordinates to film coordinates.

Drawing of Roadway Distance Contour on Film Plane

Given four corresponding coordinates of reference points for each film and roadway, any roadway distance or points can be converted to that of the film plane. A 10-ft distance contour map was drawn from computer output. Then, the film RRPs were superimposed onto the graph RRPs. After completion of this superimposition, data reduction can be performed to read time, distance, and other characteristics of a particular vehicle on the film.

TABLE 1 Geometric and Traffic Control Characteristics of Study Sites

Location Area	Intersection Approaches	Grade (%)	Type	Speed Limit (mph)	Yellow Time (sec)	All Red Time (sec)	Intersection Width (ft)
Urban	1. Commerce, Texas	-2.5	A[a]	30	4.5	1.5	125
	2. Industrial, Texas	0.5	A	35	4.5	1.5	125
	3. US 29, Virginia	-4.5	P[b]	35	3.0	1.5	220
	4. US 50, Virginia	-5.0	P	35	3.0	1.5	200
	5. Texas Avenue, Texas	0.8	P	45	4.0	1.5	100
	6. University Drive, Texas	-0.8	P	40	4.5	1.5	100
Suburban	7. South Lamar, Texas	-3.5	P	45	4.0	0.0	60
	8. Old Keene 1, Virginia	-3.5	A	45	5.0	0.0	60
	9. Old Keene 2, Virginia	-6.0	A	45	5.0	0.0	60
Rural	10. US 1, northbound, Virginia	1.0	A	50	5.0	1.0	80
	11. US 1, southbound, Virginia	-6.5	A	50	5.0	1.0	80
	12. SH 1, Texas	0.0	A	55	4.0	1.0	80
	13. SH 2, Texas	0.0	A	50	3.0	1.0	80

[a]A = actuated traffic controller.
[b]P = pretimed traffic controller.

Derivation of Yellow Response Time and Deceleration Rate

Yellow response time is measured in this study as the time elapsed from the onset of yellow until the brake light is observed to come on.

Deceleration rates can be derived from either one of the following equations:

$$d_t = 2L/T^2 \quad (8)$$

or

$$d_v = v^2/2L \quad (9)$$

L = braking distance (ft),
T = time elapsed from brake actuation to a complete stop,
v = speed at the time of brake actuation,
d_t = deceleration rate derived from using L and T, and
d_v = deceleration rate derived from using L and v.

It should be noted that between two deceleration rates in Equations 8 and 9, the expression vT = 2L must hold. If any measurement error is involved in one of v, T, and L, the two deceleration rates will not be identical. For this study, the (vT - 2L) has an error of ±5 ft and the average of d_t and d_v is used as a deceleration rate (d) for this study.

STUDY FINDINGS

The observed relative frequency of stop and go characteristics introduce the findings. Basic descriptive statistics on yellow response time and deceleration rates observed from field studies are presented. The potential perception and brake reaction time is deduced from the yellow response time with consideration of speed influence. Subsequently, other relevant information, such as the time taken for the clearing vehicle to reach the stop line, follows. Further, other factors that may influence driver-selected characteristics of yellow response time, deceleration rate, and the decision to stop at or go through a yellow light were analyzed.

Driver Response Characteristics to Change Interval

The data in Table 2 indicate the observed relative frequency of driver response characteristics with respect to signal change intervals. It shows that the overall relative ratio of stopping or going is one to two. Fifty-seven percent of the total vehicles entered intersections during the yellow cycle. Among these vehicles, two-thirds cleared the intersection during the yellow cycle but the other one-third cleared during the red cycle. The high number and percentage of "yellow entering" and "red clearing" at the site of US 29 and US 50 is attributed to the extreme width of the intersection.

The Table 2 data also indicate that 7 percent of the total number of vehicles entered the intersection during the red cycle. A substantial portion of those vehicles were observed at the sites of US 29 and US 50 in Virginia and on state highways in Texas. These two sites were operated at long cycle lengths and were observed to experience frequent long queues. The traffic operation appeared to contribute the impetus for drivers to take a high risk by entering during the red cycle.

From observed site geometric and traffic operational conditions, two suggestions can be made to reduce the frequency of vehicles that enter and clear during the red cycle: (a) that a sufficient all-red interval is to be used for those wide intersections, and (b) that traffic operations particularly as a result of long cycle length should be improved at those sites that experience a high proportion of vehicles that enter during the red cycle.

Yellow Response Time

Table 3 summarizes the values observed for yellow response time at each intersection approach and the total vehicles observed. It shows that the mean yellow response time of all drivers in the subject population was 1.3 sec and the median was 1.1 sec. Eighty-five percent of stopping vehicles applied their brakes within 1.9 sec while 95 percent did it within 2.5 sec. The cumulative distribution of yellow response time for 579 stopping vehicles is shown in Figure 1.

Yellow response time usually also includes some lag time because most situations do not require an immediate braking reaction. To derive a perception and brake reaction time from yellow response time, speed influence is introduced. The hypothesis is that drivers' yellow response time at high speed (for example, 50-55 mph) may be closely equivalent to their perception and brake reaction time because their high speeds require immediate reactions to avoid excessive deceleration or even collision with other vehicles.

Figure 2 presents the yellow response time classified by speeds. The observed speeds were classified into seven categories from 25 to 55 mph. The speed shown is the middle point of 10-mph intervals. It is shown that the median yellow response time is stabilized at 0.9 sec at speeds over 45 mph. The current value of 1 sec suggested by the ITE handbook (11) corresponds to 70 percent of the total vehicles observed in the 55 mph speed categories in this study.

TABLE 2 Observed Relative Frequency of Driver Response to Signal Change Interval

	Intersection Approach No.														
Vehicle Action	1	2	3	4	5	6	7	8	9	10	11	12	13	Total (actual)	Total (%)
S[a]	20	27	64	126	37	43	59	38	14	81	37	16	17	579	36
YEC[b]	10	6	0	0	24	34	46	73	40	253	131	2	1	620	38
YERC[c]	2	22	75	105	10	3	20	9	4	25	4	6	14	299	19
RE[d]	1	1	18	64	8	0	1	2	0	2	2	9	8	116	7
Total	33	56	157	295	79	80	126	122	58	361	174	33	40	1,614	

[a]S = stopping.
[b]YEC = enter and clear during yellow
[c]YERC = enter during yellow and clear during red.
[d]RE = enter during red.

TABLE 3 Summary Characteristics of Stopping Vehicles

Variables	Intersection Approach No. 1	2	3	4	5	6	7	8	9	10	11	12	13	Total
YRT[a]														
Mean	1.2	1.5	1.4	1.4	1.3	1.3	1.4	1.1	0.7	1.0	1.2	1.0	1.0	1.3
Median	1.0	1.4	1.2	1.3	1.1	1.0	1.3	1.0	0.7	0.9	1.1	1.0	1.0	1.1
85 percent	1.7	2.1	1.9	2.2	1.9	1.8	2.0	1.6	1.0	1.5	1.6	1.3	1.1	1.9
95 percent	2.0	2.4	3.2	2.9	3.4	2.7	2.8	1.9	1.2	1.9	1.9	1.5	1.4	2.5
DR[b]														
Mean	9.3	8.6	8.6	7.8	10.6	9.8	7.8	10.7	13.4	11.5	10.8	8.3	8.9	9.5
Median	7.7	8.6	8.1	7.6	10.9	9.4	7.9	10.2	12.9	10.8	11.3	8.1	9.1	9.2
85 percent	4.2	5.9	5.3	5.0	6.5	6.9	4.8	5.6	9.7	8.4	6.4	6.2	5.4	5.6
95 percent	3.1	3.6	3.8	4.2	5.5	5.5	3.7	4.6	7.4	6.8	4.2	4.0	4.9	4.3

[a]YRT = yellow response time (sec).
[b]DR = deceleration rate (ft/sec^2).

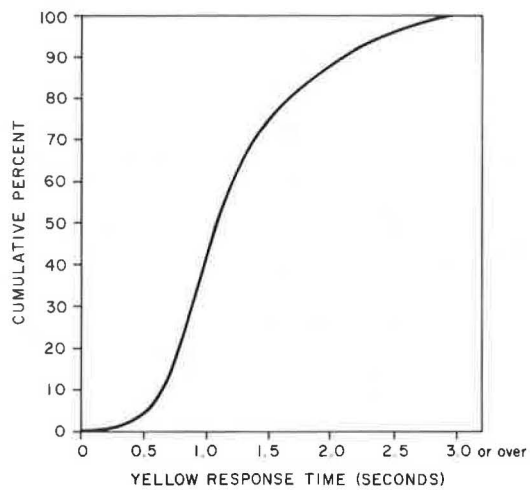

FIGURE 1 Cumulative distribution of yellow response time.

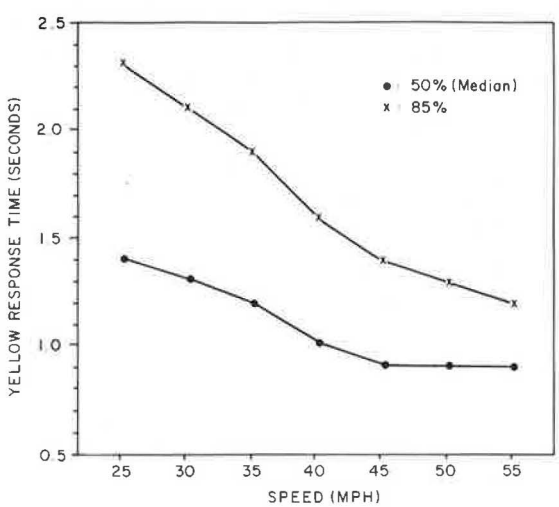

FIGURE 2 Yellow response time by speed categories.

To further validate the conceptual appropriateness of the derivation of perception and brake reaction time from yellow response time at high speed categories, another similar case requiring immediate reaction is considered in terms of distance for vehicles traveling over 40 mph. The reader is reminded that as vehicles move closer to the intersection, drivers tend to react immediately, whereas, when they are further away, the yellow response time will involve a substantial amount of response lag time. Figure 3 presents the yellow response time by distance for vehicles traveling over 40 mph. The distances shown are the middle points of 100-ft intervals. It is shown that when vehicles are relatively closer to the intersection, their 85 percent yellow response time was 1.1 sec at 200 ft and 1.3 sec at 250 ft. It is also noted that the median yellow response time for vehicles approaching over 40 mph is 0.9 sec. Combined results from Figures 2 and 3 indicate that the median perception and brake reaction time of drivers is 0.9 sec. The response lag time for the median drivers is not expected to be significant and the probable value may be around 0.1 sec.

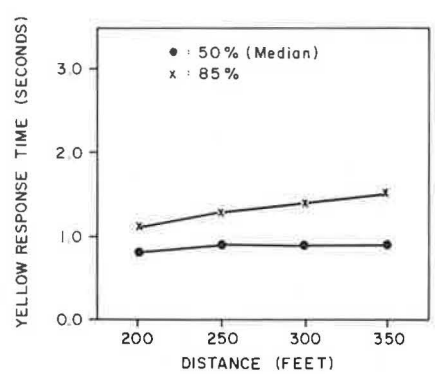

FIGURE 3 Yellow response time by distance for speed over 40 mph.

If the practice of setting the speed limit as 85 percent of the approach speed is adopted, the combined results from Figures 2 and 3 indicate that 1.2 sec of yellow response time observed from both higher speed categories and the closer distances to the intersection appears to be a good estimator of perception and brake reaction time. It is also noted that the 1.2 sec will also include an unidentified amount of response lag time. Therefore, the 85 percentile value taken from yellow response time may be close to a 90 or 95 percent value in the perception and brake reaction time distribution.

Factors That Affect Yellow Response Time

The general characteristics of yellow response time affected by driver approach speeds, distance to the

intersection, and interaction of these two are reported in a previous pilot study (8). This expanded analysis revealed similar characteristics. The effect of speed on yellow response time previously shown in Figure 2 illustrated that the yellow response time decreases as speed increases, and increases as speed decreases. This relationship is apparently attributed to driver response lag time, which usually occurs between perception and brake reaction.

To examine the effect of distance on yellow response time, the observed distances were classified into six categories ranging from 100 to 350 ft. Figure 4 presents the effect of distance on yellow response time over all speed categories. The distance shown is the middle point of 100-ft intervals. It shows, in general, that yellow response time increases as the distance to the intersection increases, and decreases as the distance to the intersections decreases. The driver response lag time is also applicable to this phenomenon.

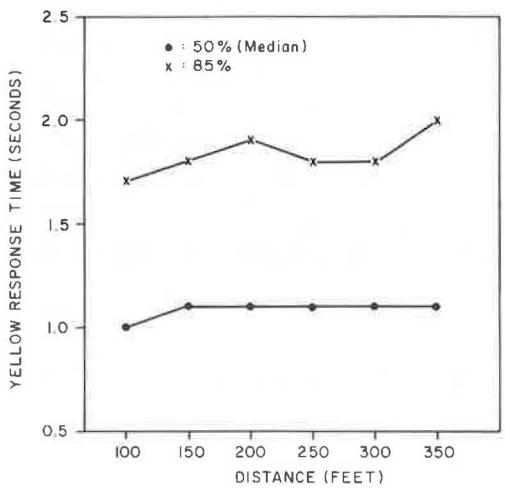

FIGURE 4 Yellow response time by distance categories.

To understand the combined effects of speed and distance, and its interaction on yellow response time, stepwise multiple regression was used and the best model obtained at $\alpha = .03$ is as follows:

$$YRT = 0.507 - [0.712\ (DONSETY/100)] + [0.423\ (DONSETY/ASPEED)] + \{0.091\ [(DONSETY/100)^2]\} \quad R^2 = 0.39$$

where

YRT = yellow response time (sec),
DONSETY = distance to intersection at yellow onset (ft), and
ASPEED = approach speed of vehicle (ft/sec).

Graphic presentation of the model in Figure 5 shows that driver yellow response time decreases as approach speed increases, and decreases as distance to the intersection at yellow onset decreases. Further, the model revealed that driver's yellow response time decreases as time available to reach the stop line decreases.

Effect of Signal Controller Type on Yellow Response Time

A test was performed to see if there is any effect of signal controller type on yellow response time. The test revealed that there is a statistically significant effect, at $\alpha = .05$, as a result of a different controller type on yellow response time. It was specifically noted that drivers tended to react more quickly to the actuated controller than to a fixed time controller; however, the magnitude of difference was found to be less than 0.1 sec. Thus, the effect of controller type on yellow response time is practically negligible.

Effects of Light and Weather on Yellow Response Time

Three intersections that have sufficient samples encompassing day and night were tested to see if there is any difference in yellow response time as a result of light conditions. The test results indicated that there is no difference (at $\alpha = .05$). This suggests that drivers tend to react consistently during day and night.

One intersection that had sufficient samples covering dry and wet pavement conditions was tested to see if there is any difference in yellow response time due to weather. The test results indicated that there is no difference in yellow response time (at $\alpha = .05$). This suggests that drivers do not appear to adjust their response particularly as a result of wet pavement conditions.

Discussions of the Various Effects on Yellow Response Time

The effects of various traffic control and environment conditions on yellow response time were tested. (Note that these effects were not tested on perception and brake reaction time.) The test results are to be interpreted such that the majority of drivers do not appear to react differently because of different conditions. It may be expected that the same driver may react more quickly during night and/or wet pavement conditions. However, the test based on identical drivers could not be performed for this study. It should also be noted that even if there is a difference in perception and brake reaction time for a different condition by a driver, the practical difference may be small because of the limitations on the extent of mental and physical reactions.

Deceleration Rate

The data in Table 3 also indicate the deceleration performance observed from field studies: They indicate that the mean and the median deceleration performance for the total vehicles was 9.5 and 9.2 ft/sec^2, respectively. Eighty-five percent of vehicles selected a deceleration rate of 5.6 ft/sec^2 or more and 95 percent used 4.3 ft/sec^2 or more. The cumulative distribution of deceleration rates observed for 579 stopping vehicles is shown in Figure 6. It should be noted that the deceleration rate observed from field studies is primarily the result of the driver's selection of comfort. It is not an indication of whether they can perform certain deceleration rates.

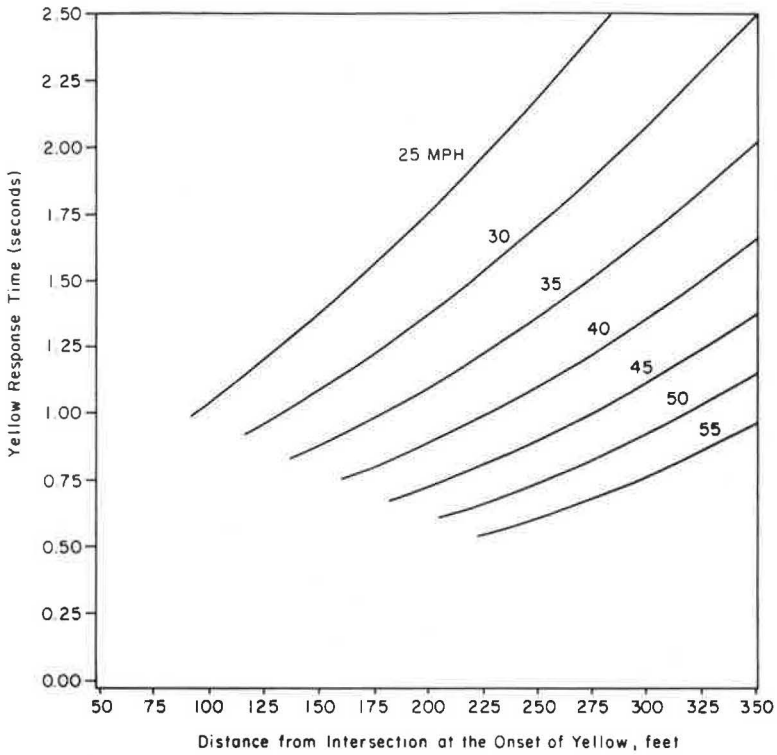

FIGURE 5 Yellow response time as a function of distance, speed, and time.

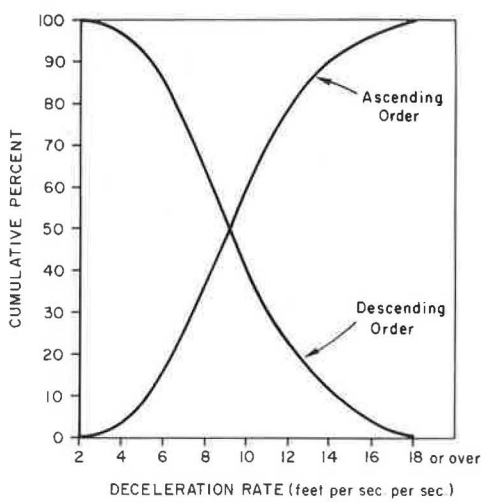

FIGURE 6 Cumulative distribution of deceleration rate.

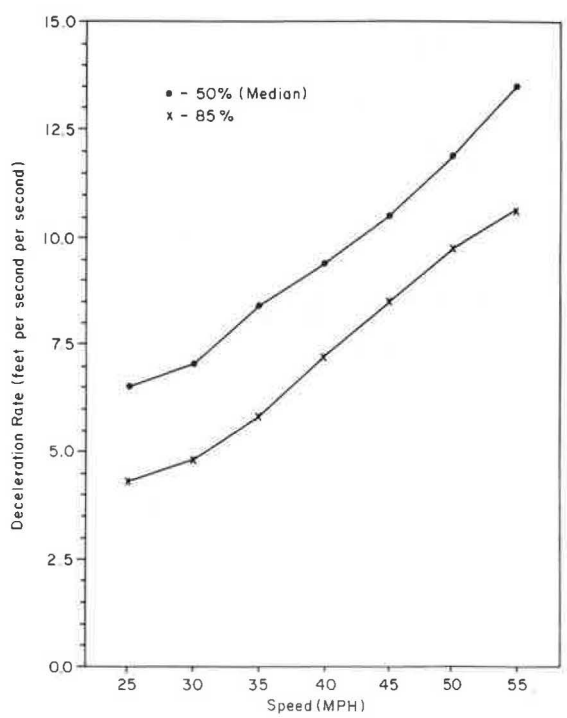

FIGURE 7 Deceleration rates by speed categories.

To derive a deceleration rate that drivers can perform, the speed influence approach used in deriving perception and brake reaction time is adopted. Figure 7 shows the deceleration performance categorized by speed. It shows that 85 percent of vehicles at 55 mph speed categories can perform deceleration rates of 10.6 ft/sec^2 or more. The deceleration rate of 10 ft/sec^2 assumed by the ITE Handbook (11) corresponds to 90 percent performance in this speed category. It is further noted that this 10 ft/sec^2 deceleration rate can also be performed by more than 85 percent of trucks (12). A deceleration rate of 10.5 ft/sec^2 is suggested for level grade.

Factors That Affect Deceleration Rate

The model to evaluate the factors that affect deceleration rate must be guided by the laws of motion from physics. These indicate that the deceleration rate is affected by speed, distance, time, and

grade. Because the braking distance available depends on the distance traveled during yellow response time, the additional interaction variable between speed and yellow response time is introduced. The deceleration rate (DR) model obtained from stepwise regression at $\alpha = .01$ is as follows:

$$DR = 13.365 + \{0.176\ [(ASPEED/10)^2]\} - [2.933\ (DONSETY/100)] + 0.085\ GRADE - [1.110\ (DONSETY/ASPEED)] + [0.044\ (ASPEED \times YRT)] \qquad R^2 = 0.86$$

The model shows that deceleration rate increases as speed, grade, and distance traveled during yellow response time increases. It increases as distance to the intersection at yellow onset and available time to reach the stop line decrease. The square distance term ($DONSETY^2$) is added for graphic presentation for the case of grade = 0 and YRT = 1 sec shown in Figure 8 to increase its predictability.

Grade Effect on Deceleration Rate

The adjustment of grade effect on deceleration performance has been advocated by some researchers (13). Consequently, the effect of grade on deceleration rate was tested. Several multiple linear regression models were evaluated by using the general linear test method. Throughout the models tested, the coefficients of grade were significant at $\alpha = .05$ and remained relatively stable at around .065 to .085 at an average of .075.

When the exact adjustment of grade effect on deceleration rate is desired, the following equation may be used:

$$d = 10.5 \pm 0.075g \qquad (10)$$

where g is the absolute percent of grade (use positive for upgrade and negative for downgrade). For safety and practical purposes, use of the following deceleration rate is recommended:

For level and upgrade	$d = 10.5$ ft/sec^2
For downgrade	$d = 10.0$ ft/sec^2

Effect of Other Factors on Deceleration Rate

A test was performed for three intersections that had sufficient samples covering day and night samples to see if there is any difference on deceleration rate as a result of light condition. The test results indicated that there was no difference in deceleration rate (at $\alpha = .05$). This suggests that drivers do not appear to select significantly different deceleration rates during night as opposed to day. Further, a test was performed for an intersection that had sufficient samples covering dry and wet pavement conditions to see if there was any difference on deceleration rates as a result of weather. The test results indicated that there was no difference in deceleration rate (at $\alpha = .05$).

It should be noted that the limitations previously described in the "Deceleration Rate" section also hold for this test.

Time Effects on Drivers' Decision to Stop or Go

It is hypothesized that drivers' perceived times to reach the stop line may influence their decision to stop or go. The time to reach the stop line for stopping vehicles is obtained by assuming constant approach speed. The time to reach the stop line for going vehicles is the actual time elapsed from the yellow onset to reach the stop line.

Figure 9 presents the time effect illustrated by speed categories. It shows that

1. Practically no vehicles stopped when they were 2 sec or less away from the intersection,
2. Eighty-five percent of stopped vehicles did stop because they were about 3 sec or more away from the intersection,
3. Eighty-five percent of going vehicles continued through the intersection because they could actually enter it within approximately 3.7 sec or less travel time,
4. Ninety-five percent of going vehicles took less than 4.5 sec to enter the intersection, and
5. The time effect was relatively stable across the speed categories. (Note: It is important to remember that these data pertain to stopping and going vehicles.)

Figure 9 also shows the dilemma of continued use of current change interval design formula. The current formula provides increased time for higher speeds while practice provides minimum yellow interval for lower speeds. Figure 9 shows that the real

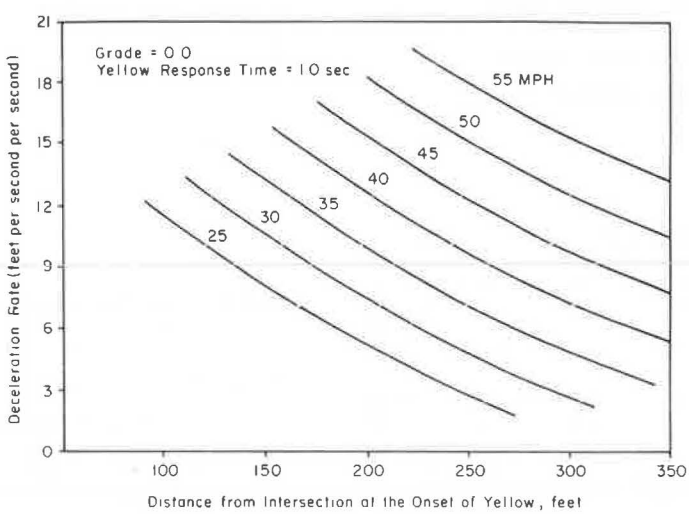

FIGURE 8 Deceleration rate as a function of distance, speed, and time.

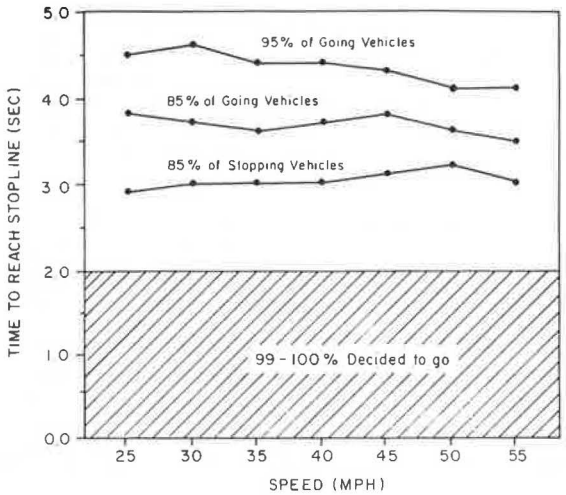

FIGURE 9 Driver's decision to stop or go by time.

danger may lie in lower speed categories below 40 mph.

Figure 9 also provides a good opportunity to consider the use of a constant yellow interval across the speed ranges. Olson and Rothery (14) suggested a constant yellow interval of 5.5 sec proclaiming that such yellow duration will provide all or nearly all drivers with time to clear an intersection. While their justification does not appear to be sound because of different dimensions of intersection width, Figure 9 appears to show a warranting condition for a constant yellow interval of 4.5 sec from the fact that 95 percent of going vehicles did go through when they took less than about 4.5 sec regardless of their speeds. The determination of yellow interval based on going vehicles is warranted because the fundamental problem of the yellow interval lies in the clearing vehicle rather than the stopping vehicle. The basic reason is that the first car stopped has no vehicles with which to collide. The following vehicles may collide with the first vehicle stopped. However, the rear end collision in this case is a result of driver expectancy violation along with following too closely, rather than a consequence of the yellow interval.

Probability Modeling of Driver Decision to Stop or Go

Past studies reported that driver decisions to stop or go were affected by approach speed, distance from the intersection at the yellow onset, and the time to reach the stop line (15,16,17). These three different decision-affecting factors are illustrated in Figure 10. Figure 10a shows the case of speed dominance decision in which the slope of the same time is downward to the lower probability of stopping as distance is increased. Figure 10b shows the distance dominance decision in which the slope of the same time is upward to the higher probability of stopping as distance is increased. Figure 10c shows the time dominance decision in which the slope of the same time has approximately the same probability of stopping. The model by Williams (15) has a time characteristic of Figure 10b while the model by Sheffi and Mahmassani (16) has a time characteristic of Figure 10c.

Logistic regression (or logit model) was used to derive the probability of stopping or going as a function of speed, distance, and time. For the stopping vehicle, time is derived by assuming constant speed as mentioned previously. The stepwise logistic regression model revealed that the first important variable entered was time, the second was distance, and the third was speed in sequence. However, when the distance and speed were entered, time became insignificant at the chi-square value of 0. Thus, the model obtained at $\alpha = .05$ is as follows:

$$\text{Probability of stopping} = 1/\{1 + \exp[2.083 - 2.755\,(\text{DONSETY}/100)] + [0.071 \times \text{ASPEED}]\}$$

The model revealed that the probability of stopping decreases as distance decreases and it decreases as approach speed increases. The graphic presentation shown in Figure 11 illustrates the characteristics. Figure 11 also revealed that the driver decision is a distance dominance pattern previously illustrated in Figure 10b. The predicted performance of the probability model compared to the observed frequencies of stopping and going is shown in Table 4. The probability model predicted with 80 percent accuracy the responses of stopping and going.

Effect of Grade on Driver Decision to Stop or Go

It is postulated that grade may have an effect on drivers' decision to stop or go. It is expected that more drivers may decide to go through rather than to stop, given the same approach speed and distance to the intersection, at downgrades than upgrades. The effect of grade on drivers' decision to stop or go

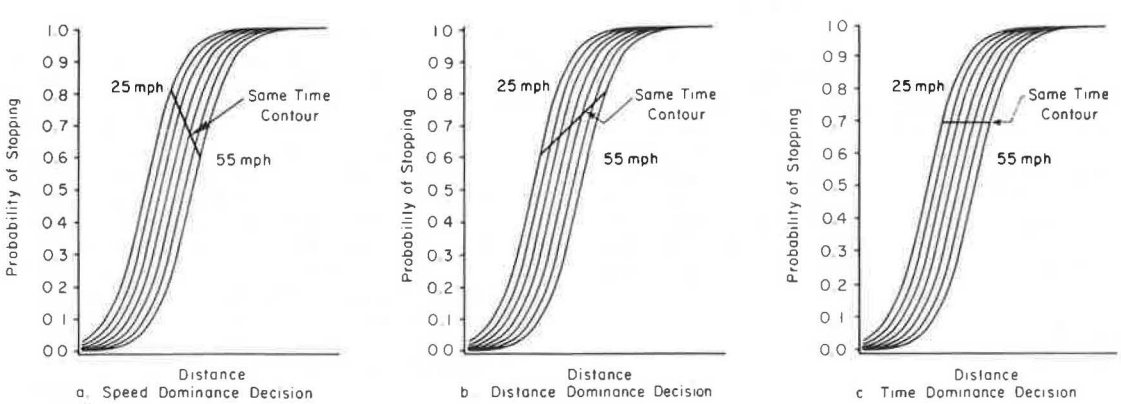

FIGURE 10 Relationship between probability of stopping and driver's decision pattern.

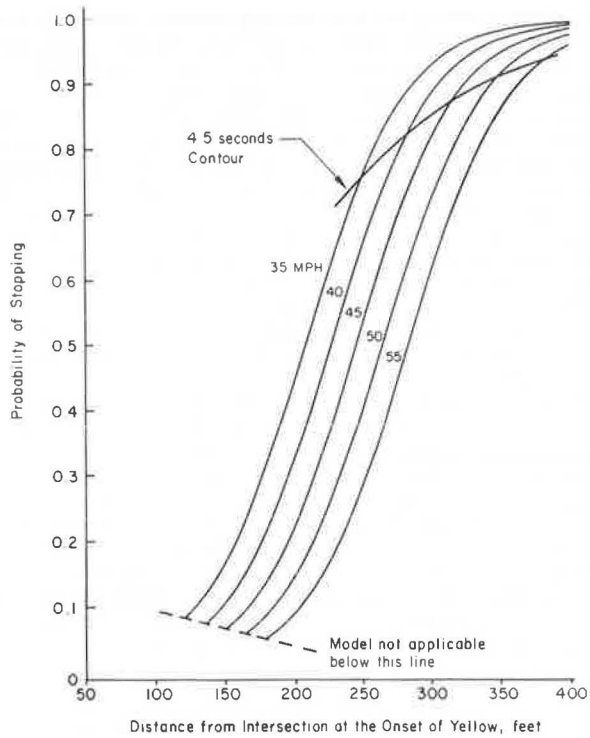

FIGURE 11 Probability of stopping as a function of distance and speed.

TABLE 4 Predicted Performance of Probability Model

	Predicted		
Observed	Going	Stopping	Total
Going	887	148	1,035
Stopping	182	397	579
Total	1,069	545	1,614

Note: The correct rate = (887 + 397)/(1,614 = 79.6 percent; the false stopping rate = 148/545 = 27.2 percent; and the false going rate = 182/1,069 = 17.0 percent.

was tested using the logistic model. The model obtained at α = .01 is as follows:

$$\text{Probability of stopping} = 1/\{1 + \exp[1.870 - 2.790\,(\text{DONSETY}/100)] + 0.069 \times \text{ASPEED} - 0.115 \times \text{GRADE}\}$$

where grade is the percent of grade (use positive for upgrade and negative for downgrade).

The model revealed that the probability of stopping increases as grade increases and it decreases as grade decreases. In other words, more drivers tend to stop on upgrades but tend to go through on downgrades. The provision of an all-red interval on downgrades would be helpful to counterbalance the potential accidents as a result of the greater tendency toward drivers going through on downgrade approaches.

Effect of Intersection Width on Driver Decision to Stop or Go

It is also postulated that intersection width may also have an effect on the driver's decision to stop or go. It is expected that more drivers may decide to go through rather than stop, given the same approach speed, distance, and grade to the intersection, at narrower intersections than at wider intersections. The effect of intersection width on the driver's decision to stop or go was tested by using the logistic model. The model obtained at α = .01 is as follows:

$$\text{Probability of stopping} = 1/\{1 + \exp[5.038 - 3.013\,(\text{DONSETY}/100)] + 0.044 \times \text{ASPEED} - 0.198 \times \text{GRADE} - 0.014 \times \text{INTW}\}$$

where INTW is the intersection width in feet. The model revealed that the probability of stopping increases as intersection width increases and it decreases as intersection width decreases. In other words, more drivers tend to stop at wider intersections but tend to go through at narrower intersections.

All-Red Interval

All-red time is often provided at some intersections to let vehicles clear the intersection during the protected time. All-red time is particularly useful when the intersection is wide and when many vehicles tend to enter the intersections during the latter part of the yellow interval.

Speed Influence on All-Red Time

The current change interval design provides clearance time in the form of (l + w)/v. It is noted that a constant approach speed is used to derive clearance time. Observations revealed that the majority of drivers accelerate when they need to clear the intersection. It also appears to be a duty for those drivers who entered the latter part of the yellow cycle to clear the intersection as soon as possible within their vehicles' capabilities. The speed difference between the approach speed before the yellow cycle (ASPEED) and the final average speed after the yellow cycle until the vehicle clears the intersection (FSPEED) was analyzed and the relationship was obtained at α = .01 as shown in the following equation:

$$\text{FSPEED} = 1.08\ \text{ASPEED} \qquad R^2 = 0.99 \tag{11}$$

This suggests that use of constant speed may provide unnecessarily long all-red time particularly when the intersection is wide.

Starting Delay Influence on All-Red Time

There is a delay from the time the driver first sees the green onset until the driver starts moving the vehicle from a stopped position. The starting delay obtained from this study was analyzed and the starting characteristics of a total of 3,527 vehicles (being the first ones positioned in queue) were observed at the onset of the green cycle. Twenty-seven vehicles started before green onset (called "light jumping"), which was 0.8 percent of the total samples. (It should be noted, however, that light jumping is an illegal violation of traffic signal display.) The mean starting delay was 1.8 sec and the median was 1.7 sec. Eighty-five percent of vehicles took longer than 1 sec to start. Ninety-five percent of vehicles took more than 0.8 sec to start. Since the stop line was set back from the path of cross traffic, the use of a 1-sec starting delay may be applicable to 95 percent of vehicles.

Application of Results--Example to Determine All-Red Interval

Consider the following example of determining the duration of the all-red interval. The following intersection characteristics will be assumed:

- Speed limit or 85 percentile speed = 40 mph (58.7 ft/sec),
- Intersection width = 100 ft,
- Passenger car length = 20 ft, and
- Yellow time = 4 sec.

Step 1. Calculate the distance traveled during the yellow interval: Distance = 58.7 x 4 = 235 ft;
Step 2. Add the intersection width and the passenger car length: Distance = 235 + 100 + 20 = 355 ft;
Step 3. Divide the Step 2 distance by FSPEED: Time = 355/(58.7 x 1.08) = 5.6 sec;
Step 4. Subtract the starting delay from the Step 3 time: Time = 5.6 - 1.0 = 4.6 sec;
Step 5. Subtract the yellow time from Step 4: All-red interval = 4.6 - 4.0 = 0.6 sec.

If the value obtained in Step 5 is negative, no all-red time is necessary. Therefore, this intersection would need 0.6 sec of all-red time.

It is warned, however, that the starting delay of 1 sec should be applied with extreme caution and a value of 0 should be used under the following conditions: (a) the driver's view to the intersection is obstructed as a result of either intersection geometrics or adjacent large vehicles such as trucks, and (b) the crossing signal is visible or progression is provided such that approaching vehicles either do not tend to stop completely or do not take significant time to start. Further, legal implications based on local laws and ordinances should be investigated before the engineer decides to apply the cross-flow reduction value.

Change Interval and Signal Lost Time

Signal lost time is a parameter used for the calculation of signal timing and is consequently applied to effective green time and level of service at signalized intersections. The signal lost time is defined as

$$T_l = (Y + AR) - U_y + T_s \quad (12)$$

where

T_l = signal lost time (sec),
U_y = utilized yellow time (sec), and
T_s = starting delay (sec).

The current study revealed that the mean travel time for clearing vehicles was 2.6 sec and the mean starting delay was 1.8 sec. Therefore, mean signal lost time will be

$$T_l = (Y + AR) - 2.6 + 1.8 = Y + AR - 0.8 \quad (13)$$

Taking a conservative value, the signal lost time corresponds to (Y + AR) - 1 sec.

CONCLUSIONS

The following conclusions were drawn from the data collected and field observations made within this study. They apply within the seven intersections studied and the observed operational environments.

1. The observed mean yellow response time selected by drivers was 1.3 sec and the median was 1.1 sec. Eighty-five percent of stopping drivers applied their brakes within 1.9 sec after the yellow onset while 95 percent of drivers did it within 2.5 sec.
2. The derived 85 percent perception and brake reaction time excluding driver's response lag time from yellow response time was 1.2 sec.
3. Driver's yellow response time is affected by distance to the intersection at the yellow onset, approach speed, and the time available to reach the stop line after the yellow onset.
4. The observed mean deceleration rate selected by drivers was 9.5 ft/sec^2, and the median was 9.2 ft/sec^2.
5. Grade affects deceleration rate approximately 0.075 ft/sec^2 for each percentage of grade. For safety and practical purposes, a deceleration rate of 10.5 ft/sec^2 is suggested for level and upgrades, and 10.0 ft/sec^2 is suggested for downgrade.
6. Driver-selected deceleration rate was affected by approach speed, the distance to the intersection at the yellow onset, the time available to reach the stop line after the yellow onset, and the distance traveled during the yellow response time.
7. Eighty-five percent of stopping vehicles stopped when they were more than 3 sec away from the intersection.
8. Eighty-five percent of going vehicles went through the intersection when they were less than 3.7 sec away. Ninety-five percent of going vehicles continued when their travel times to the intersection was less than 4.5 sec.
9. The safety implication of going vehicles and the stability of going vehicles with respect to time suggest the potential use of a constant yellow interval of 4.5 sec across all speeds.
10. Driver probability of stopping or going was affected by approach speed and the distance to the intersection at the yellow onset.
11. The higher risk of accidents as a result of the yellow interval appears to exist for the lower speed categories below 40 mph.
12. The average speed for going vehicles from the yellow onset to clearing the intersection is 8 percent higher on the average than their approach speeds before the yellow onset.
13. The mean starting delay to the green onset was 1.8 sec and the median was 1.7 sec. Eighty-five percent of the vehicles took more than 1 sec to start. This starting delay may be applicable to determine all-red time.

ACKNOWLEDGMENTS

This study was made possible by the cooperation and assistance provided by a number of public agencies in the States of Virginia and Texas. These include cities, counties, and state departments of transportation. We are grateful to the traffic engineering staffs of each of these agencies. We also appreciate the help and cooperation of the Office of Safety and Traffic Operations Research and Development of the Federal Highway Administration. The authors would further like to thank their fellow staff members of the Texas Transportation Institute. Contributions by our research technicians for data collection and reduction is sincerely appreciated. Particular appreciation is extended to Do Il Kim and Sei-Kwon Park who provided consistent help throughout the project.

REFERENCES

1. Uniform Vehicle Code and Model Traffic Ordinances. National Committee on Uniform Traffic Laws and Ordinances, Revised 1968. The Michie Company, Charlottesville, Virginia, 1968.
2. H.H. Bissel and D.L. Warren. The Yellow Signal is NOT a Clearance Signal. ITE Journal, Feb. 1981, pp. 14-17.
3. B. Benioff, F.C. Dock, and C. Carson. A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals. Vol. 2, Clearance Intervals. Report FHWA-RD-78-47. FHWA, U.S. Department of Transportation, 1978.
4. W.A. Stimpson, P.L. Zador, and P.J. Tarnoff. The Influence of the Time Duration of Yellow Traffic Signals on Driver Response. ITE Journal, Nov. 1980, pp. 22-29.
5. D. Gazis, R. Herman, and A. Maradudin. The Problem of the Amber Signal in Traffic Flow. Traffic Engineering, July 1960, pp. 19-26.
6. M.S. Chang and C.J. Messer. Engineering Factors Affecting Traffic Signal Yellow Time. Report FHWA-RD-85-54. FHWA, U.S. Department of Transportation, 1984.
7. C.J. Messer and M.S. Chang. Engineering Factors Affecting Traffic Signal Yellow Time: Data Collection Manual. Report FHWA-RD-85-55. FHWA, U.S. Department of Transportation, 1984.
8. M.S. Chang, C.J. Messer, and A. Santiago. Evaluation of Engineering Factors Affecting Traffic Signal Change Interval. In Transportation Research Record 956, TRB, National Research Council, Washington, D.C., 1984, pp. 18-21.
9. B. Hallert. Photogrammetry, Basic Principles and General Survey. McGraw-Hill, New York, 1960.
10. R.L. Bleyl. Traffic Analysis of Time Lapse Photographs Without Employing a Perspective Grid. Traffic Engineering, Aug. 1972, pp. 29-31.
11. Traffic and Transportation Handbook. Institute of Traffic and Transportation Engineers, Washington, D.C., 1982.
12. P.A. Winter. 1974 Brake Performance Levels for Truck and Passenger Cars. Public Roads, Dec. 1976, pp. 108-115.
13. P.S. Parsonson and A. Santiago. Design Standards for Timing--the Traffic Signal Clearance Period Must Be Improved to Avoid Liability. In Compendium of Technical Papers, ITE, 1980, pp. 67-71.
14. P.L. Olson and R.W. Rothery. Deceleration Levels and Clearance Times Associated with Amber Phase of Traffic Signals. Traffic Engineering, Apr. 1972, pp. 16-19.
15. W.L. Williams. Driver Behavior During the Yellow Interval. In Transportation Research Record 644, TRB, National Research Council, Washington, D.C., 1977, pp. 75-78.
16. Y. Sheffi and H. Mahmassani. A Model of Driver Behavior at High Speed Signalized Intersections. Transportation Science, Feb. 1981, pp. 50-61.
17. C.V. Zegeer. Effectiveness of Green-Extension Systems at High-Speed Intersections. Research Report No. 472. Kentucky Department of Transportation, Louisville, May 1977.

Publication of this paper sponsored by Committee on Traffic Control Devices.