Evaluation of Driver Behavior at Signalized Intersections

ROBERT H. WORTMAN AND JUDSON S. MATTHIAS

Time-lapse photography was used to study driver behavior associated with the traffic signal change interval at a total of six intersections in the Phoenix and Tucson metropolitan areas. In addition, nighttime studies were conducted at two of these intersections. An evaluation of the time-lapse film permitted the determination of the approach speeds of the vehicles, the average deceleration rates of the stopping vehicles, the perception-reaction times of the drivers of the stopping vehicles, and the distance that the vehicle was from the intersection at the onset of the yellow interval. The distance from the intersection was measured for both the stopping vehicles as well as those that proceeded through the intersection. The results of the study indicated that the mean deceleration rates at the six sites ranged from 7.0 to 13.9 ft/s/s, and the mean value for all observations was 11.6 ft/s/s. The observed mean perception-reaction time was approximately 1.3 s while the 85th percentile times ranged from 1.5 to 2.1 s. Comparisons of intersections with yellow only versus yellow plus all-red intervals produced mixed results in terms of differences in observed behavior. Even for intersections with the same change interval design, there were cases where the observed deceleration rates were significantly different.

The failure to properly recognize and understand driver behavior in signalized-intersection design can contribute to operational and safety problems. A review of 1980 traffic accidents in Arizona reveals that approximately 3 percent of the reported accidents had disregard of a traffic signal listed as a contributing circumstance (1).

The recognition of driver behavior is particularly critical in the design of the traffic signal change interval. The change interval has two recognized purposes. The first is to advise the motorist that the red interval is about to commence, and the second is 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 determination of the change interval considers driver perception-reaction time and vehicle deceleration rates; the use of unrealistic values for these factors would potentially affect driver compliance, safety, and even intersection capacity. In the equations used in the determination of the duration of the change interval, a perception-reaction time of 1 s and a deceleration rate of 10 ft/s/s are suggested (2). The latter value has been decreased from 15 ft/s/s, which was used in practice for a number of years.

The current Arizona Department of Transportation policy (3) indicates that 8 and 12 ft/s/s are the upper and lower limits for deceleration rates that are used in establishing the change interval duration. This range provides some degree of latitude in applying engineering judgment to the determination of the change interval; however, the policy further states that 10 ft/s/s should normally be used.

In recent years, several studies have focused on driver behavior during the yellow signal interval. For example, Williams (4) reported that a study of an intersection in New Haven, Connecticut, revealed that the average maximum deceleration rate for stopping vehicles was 9.7 ft/s/s. Other studies (5,6) have suggested that a reasonable deceleration rate would be in the magnitude of 10 ft/s/s.

Two previous studies included field measurements of driver reaction time. In an early study of the change interval problem, Gazis, Herman, and Maradudin (7) found a mean reaction time of 1.14 s based on a sample of 87 observations. A study in 1966 by Jenkins (8) contained a sample of 21 observations of reaction time. An analysis of the data sample obtained by Jenkins reveals that the mean reaction time was approximately 1.4 s.

The Transportation and Traffic Engineering Handbook (2) states that excessively short or long yellow intervals are undesirable; thus, the common practice is to use yellow intervals of 3-5 s. If a longer duration is required to clear the intersection before the cross traffic enters, an all-red period can be used in addition to the selected yellow interval. The Arizona Department of Transportation policy (3) indicates that a yellow interval of up to 6 s may be used. If a longer change interval is required, then an all-red interval should be used.

In Arizona, as in most states, it is legal for the vehicle to enter the intersection during the yellow signal indication. It is not necessary for the vehicle to have cleared the intersection prior to the onset of the red signal indication.

The purpose of this study focused on examining driver behavior that is related to the change interval. The intent was to document driver behavior parameters and possible variations in behavior that exist in Arizona. More specifically, the study objectives focused on the determination of the following items:

1. The actual deceleration rates and the range of deceleration rates that are used by drivers,
2. Possible differences in driver behavior due to the intersection environment, and
3. The effect of the use of the all-red phase on driver behavior; in addition, the study provided information about the perception-reaction times of drivers as well as measures of driver compliance in terms of the signal indications.

It should be noted that it was not feasible to conduct before-and-after studies of the effects of changes in the change interval design and duration at a particular intersection. The study was limited, therefore, to examining behavior at existing intersections with existing signal timing.

Driver behavior associated with the change interval is certainly a complex phenomenon in that there are numerous facets to the overall problem. It was not the intent of this particular study to include an evaluation of the current practice used in the determination of change intervals.

METHOD OF STUDY

Time-lapse photography was used to record the daytime driver behavior associated with the change interval at six intersections. Two of the six intersections were also observed during the nighttime period. Vehicles approaching the intersection were filmed for a few seconds prior to the onset of the yellow, during the change interval, and until the vehicle either stopped or cleared the intersection. Given the onset of the yellow signal indication, the study focused on the first vehicle to stop and the last vehicle to pass through the intersection. Where multiple approach lanes were involved, this information was included for each lane. It was not possible to determine such information such as the age, sex, experience, and route familiarity of the driver.

The camera was located so that it was possible to record the intersection and the signal indication as well as the operation of approaching vehicles within 350-400 ft of the intersection. A super 8-mm movie camera with a zoom lens and a 16-mm movie camera were used for data collection. Because of the age of the 16-mm equipment, it proved to be less reliable and more expensive to operate than the super 8-mm unit; thus, it was used only on a limited basis. Also, higher-speed film for the night observations was readily available for the super 8-mm camera; therefore, that camera was used for all of the nighttime studies. The cameras were operated at a frame speed of 18 frames/s to increase the accuracy of the time measurements.

The day filming was accomplished by using a Kodak Kodachrome color film. This made it possible to easily distinguish the signal indications as well as when brake lights were illuminated on a vehicle. For the night filming, a Kodak Ektachrome color film was used. Although it was not possible to see the entire vehicle at night, the lights on the vehicle and the traffic signals were recorded on the film.

If available, the intersection was filmed from a nearby building or structure. If such a facility was not available at a particular location, the Arizona Department of Transportation furnished a truck with an elevating platform that would extend to a height of approximately 30 ft. The camera was placed on a tripod, which resulted in a total height of about 35 ft. The truck was parked in a parking lot or vacant area approximately 400-450 ft from the intersection. Where the truck was employed for filming, it was located about 30-50 ft from the edge of the roadway. There was concern that the presence of the truck might influence the behavior of the drivers. Based on observations by the study teams at the sites, there was no evidence that the drivers were cognizant of the filming activities.

Distances from the intersection were noted by using reference points on the roadway. Generally, strips of tape were placed at 50-ft intervals along the edge of the approach. In some cases, the dashed-lane striping was measured and used as the reference for distance from the intersection.

Site Selection

The following six intersections in the Phoenix and Tucson metropolitan areas were selected for study:

1. Phoenix metropolitan area: University Drive and Rural Road (located in Tempe), Southern Avenue and McClintock Drive (located in Tempe), and US-60 and Greenfield Road (located in Mesa); and
2. Tucson metropolitan area: First Avenue and Roger Road, Sixth Street and Campbell Avenue, and Broadway Boulevard and Columbus Boulevard.

Only one approach at each of the intersections was studied, and the study approach was located on the first street that is shown in the list of intersections. Table 1 summarizes the characteristics of each of the intersection approaches that were observed.

Major difficulties were encountered in the selection of intersections that were to be used for study observations. These difficulties served to severely limit the choice of intersections that could even be considered. Even with the option of placing the camera in a nearby building or on a truck with an elevating platform, it was frequently difficult to find a suitable building or an acceptable place to locate the truck. Also, if the traffic signal at an intersection is operated as part of an areawide system, approaching vehicles frequently may not be in proximity to the intersection at the onset of the yellow interval. The same may be true for signalized intersections in outlying areas that have traffic-actuated controllers. When such conditions exist, the probability of obtaining a data sample is relatively small, and the time required for data collection is greatly increased. Jurisdictions are reluctant to temporarily isolate a signal from the control system because of liability questions.

Filming of Intersection Approaches

Because of the arrangements that were necessary for access to buildings or to use a truck with a platform, filming activities were accomplished in periods of several hours duration. For example, a study team would frequently film an approach for a 6- to 8-h period. For the night observations, filming was undertaken during the months of December and January; thus, data collection generally began about 6:00 p.m. and was terminated about 11:00 p.m.

It took about 12-16 h of filming at each site to obtain a sample size of approximately 100 stopping vehicles. Even with this filming period, there were some intersection approaches that yielded less than 100 samples when the film was analyzed. It might seem that this duration of filming is somewhat excessive for such a sample size; however, there are two explanations. First, cycle lengths were sometimes as high as 120 s. Second, there were numerous cases for which there were no vehicles in the proximity of the approach at the onset of the yellow interval. Because of the duration of filming activities, it was not possible to focus on particular periods of time during the day. The observations that were made, therefore, represent a cross section of the traffic conditions at an intersection.

Data Reduction

By using the distance reference points that were

Table 1. Characteristics of study sites.

Site	Estimated ADT[a]	Change Interval[b] (s)	Approach Configuration	Left-Turn Signalization
University Drive* and Rural Road	24 000	5Y	Two lanes plus exclusive left-turn lane	Exclusive left-turn phase
Southern Avenue* and McClintock Drive	22 100	5Y	Three lanes plus exclusive left-turn lane	Exclusive left-turn phase
US-60* and Greenfield Road	24 200	5Y+3AR	Three lanes plus exclusive left-turn lane	Exclusive left-turn phase
First Avenue* and Roger Road	21 400	3Y+2AR	Two lanes plus exclusive left-turn lane	Turns permitted on a permissive basis during through movement
Sixth Street* and Campbell Avenue	18 300	3Y+2AR	Two lanes plus exclusive right- and left-turn lanes	Turns permitted on a permissive basis during through movement except in peak hours when left-turn lane is used as a reversible lane
Broadway Boulevard* and Columbus Boulevard	35 800	3.6Y + 2AR	Three lanes plus exclusive right- and left-turn lanes	Exclusive left-turn phase plus permissive left turns during through movement

Note: ADT = average daily traffic, and *denotes street on which the observed approach was located.

[a]ADT is for the street on which the observed approach was located.
[b]Denotes design and duration of change interval. For example, 5Y + 3AR indicates 5 s of yellow plus 3 s of all-red time.

Table 2. Study site speed characteristics.

		Approach Speed							
		Last Vehicle Through the Intersection				First Vehicle to Stop			
Intersection Approach	Posted Speed Limit (mph)	Sample Size	Mean Speed (mph)	Standard Deviation	85th Percentile (mph)	Sample Size	Mean Speed (mph)	Standard Deviation	85th Percentile (mph)
University Drive	35	16	35.46	7.49	45.8	64	33.69	8.81	43.4
Southern Avenue									
Day	45	69	36.31	5.25	42.4	70	34.97	7.08	40.9
Night	45	49	40.58	6.30	47.7	64	35.77	7.17	41.8
US-60	50	87	39.66	7.52	48.0	107	35.88	7.90	44.8
First Avenue	45	152	39.84	6.42	46.5	178	39.04	7.27	46.9
Sixth Street	30	67	35.24	5.50	40.5	97	31.63	6.05	37.7
Broadway Boulevard									
Day	45	156	41.16	5.47	47.0	143	37.41	6.47	43.5
Night	45	96	37.27	5.54	44.0	116	36.32	5.43	42.3
All approaches		693	38.91	6.38	45.8	839	36.13	7.30	43.5

Table 3. Distance from intersection at beginning of yellow interval.

	Last Vehicle Through Intersection		First Vehicle to Stop	
Intersection Approach	Mean Distance (ft)	Standard Deviation	Mean Distance (ft)	Standard Deviation
University Drive	118.9	64.0	268.7	74.2
Southern Avenue				
Day	142.9	60.2	262.6	60.1
Night	152.6	72.8	272.0	52.2
US-60	149.6	62.2	262.5	54.4
First Avenue	136.3	46.9	238.4	48.0
Sixth Street	102.7	33.7	202.5	51.0
Broadway Boulevard				
Day	146.5	49.3	250.5	45.9
Night	114.1	46.4	240.9	49.0
All approaches	135.4	54.5	246.6	56.1

established on each of the study approaches, a grid was developed so that the location of a vehicle could be determined when the film was projected on a screen. The grid indicated the distance from the intersection. In all cases, this distance was measured from the crosswalk.

For each of the vehicles that were the first to stop after the beginning of the yellow interval, the following information was extracted from the film record:

1. Distance from the intersection at the beginning of the yellow interval,
2. Location of the vehicle when the brakes were applied (as indicated by the brake lights),
3. Location of the vehicle when it stopped,
4. Time required for the vehicle to stop,
5. Perception-reaction time (determined as the time between the beginning of the yellow interval and the application of the brakes), and
6. Type of vehicle if other than a passenger car or light truck.

Based on this information, the approach speed and the average deceleration rate was computed for each stopping vehicle.

In addition, the behavior of the last vehicle to pass through the intersection after the beginning of the yellow was determined by making the following observations:

1. Location of the vehicle at the beginning of the yellow interval,
2. Time elapsed from the onset of the yellow interval until the vehicle entered the intersection,
3. Type of vehicle if other than a passenger car or light truck, and
4. If the vehicle entered the intersection on the red signal indication.

For the vehicles that did not stop, the approach speed was also computed. In this case, the determination of speed was accomplished by using the elapsed time and the distance traveled.

RESULTS

Table 2 indicates the observed speed characteristics at the study sites. At all six study locations, the mean approach speeds of the last vehicle through the intersection were higher than the first vehicle to stop. It should be recognized that the measurement of the speed of the first vehicle was made at a point where some deceleration may have occurred. Also, speed measurements for the through vehicles were made before the onset of the yellow interval, and some of those vehicles may have been accelerating. It was not possible, however, to determine if these vehicles were accelerating from the films that were made.

Although the posted speed limits ranged from 30 to 50 mph, there was much less variation in the vehicle approach speeds. The difference between the high and low mean speed was less than 6 mph for the last vehicle through the intersection and less than 8 mph for the first vehicle to stop. This indicates that there was not that much diversity in the vehicle operations when comparing the sites.

With respect to the comparison of the intersection approaches where the day and night studies were conducted, there was no significant difference in the approach speeds for the first vehicle to stop. The approach speeds were significantly higher at night for the through vehicles in the case of the Southern Avenue site and were significantly lower at night at the Broadway Boulevard site. It might be expected that the nighttime approach speeds would consistently be higher because of the lower volumes.

Table 3 summarizes the mean distance from the intersection at the beginning of the yellow interval for the two groups of vehicles at each site. As would be expected, the distance from the intersection was considerably less for the last vehicle through the intersection. In fact, the mean distance differences ranged from approximately 100 to 150 ft for the study approaches. Figure 1 depicts the cumulative frequency distributions of the distance from the intersection for all sites.

A summary of the observed perception-reaction

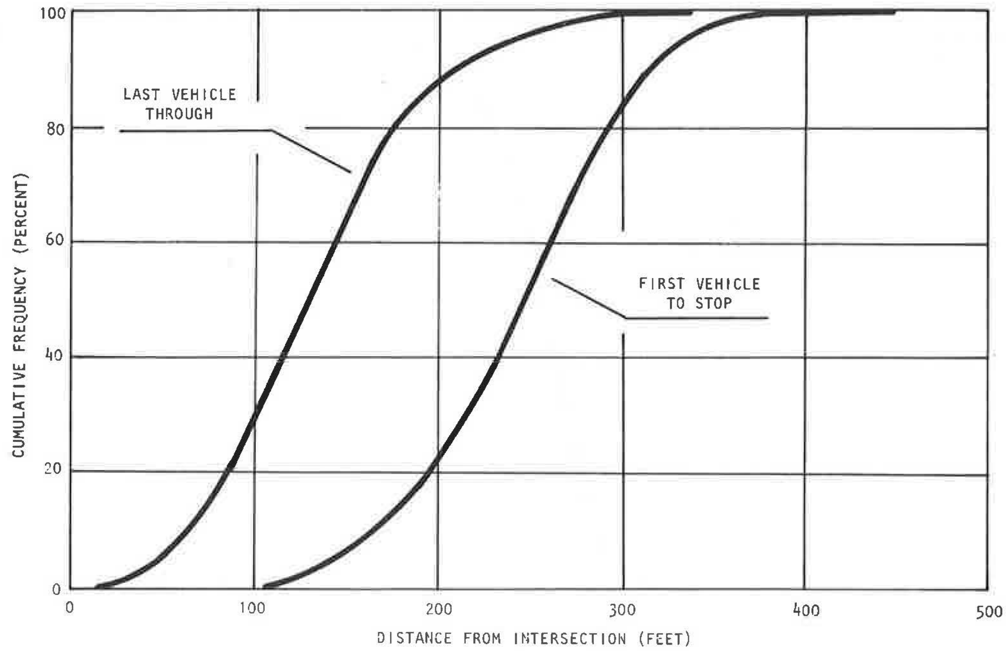


Figure 1. Cumulative frequency distribution of vehicle location at onset of yellow interval.

times is given in Table 4. As indicated previously, the perception-reaction time was determined by measuring the time between the onset of the yellow interval and the application of the brakes. These times are for the stopping vehicles only. Generally, the study team was unable to determine the perception-reaction time for the drivers who chose to proceed through the intersection. There were a few cases during the night studies where it was possible to detect that drivers began to apply the brakes but then proceeded through the intersection. In these cases, a brief flickering of the brakelights was recorded on the film. The cumulative frequency distribution curve for all approaches is shown in Figure 2.

It was theorized that the perception-reaction time is related to such factors as the location of the vehicle at the beginning of the yellow interval, the approach speed, or even possibly the deceleration rate that was used by the driver. These hypotheses were tested by using correlation and regression analyses and computing the coefficient of determination (r^2). When these variables were analyzed in terms of the relation with the length of the perception-reaction time, the following r^2-values were obtained:

Variable	r^2
Distance from intersection	0.08
Approach speed	0.09
Deceleration rate	0.01

These results would serve to indicate that there was little relation between the perception-reaction time and these variables.

Table 5 summarizes the deceleration rates that were observed at the study locations, and the cumulative frequency distribution of the deceleration rates is shown in Figure 3. It should be emphasized that these observed values reflect driver behavior under the set of conditions experienced at the study sites. Further analysis of the variation in the deceleration rates with respect to the study sites is discussed later.

The tabulation of the vehicles entering the intersection on the red signal indication is given in

Table 4. Perception-reaction times.

Intersection Approach	Mean Time (s)	Standard Deviation	85th Percentile Time (s)
University Drive	1.28	0.82	2.0
Southern Avenue			
Day	1.49	0.62	1.9
Night	1.43	0.73	2.0
US-60	1.38	0.60	2.1
First Avenue	1.24	0.51	1.8
Sixth Street	1.55	0.70	2.0
Broadway Boulevard			
Day	1.16	0.48	1.5
Night	1.09	0.44	1.5
All approaches	1.30	0.60	1.8

Table 6. Generally, the intersections with the shorter yellow interval and the all-red phase resulted in a higher percentage of vehicles entering on the red indication. This would be expected due to the shorter yellow duration. The First Avenue site had an extremely high percentage of vehicles entering on the red signal compared with the other locations. Although this intersection is situated in a more outlying area than the other two study locations in Tucson, there is no clear explanation for this percentage being so much higher than the other sites.

When comparing the variation between the day and night conditions, the percentage of entering vehicles was drastically reduced during the night conditions. This may be partly explained by the fact that there were less vehicle queues during the nighttime period. The deceleration rate at the Broadway Boulevard site also decreased during the nighttime period.

COMPARATIVE ANALYSIS OF STUDY SITES

The second major portion of the analysis effort dealt with the comparison of observed behavior at the different study sites. Basically, these comparisons were intended to test the influence of the driving environment and signal timing practices.

The mean and standard deviation for each of the

Figure 2. Cumulative frequency distribution of observed perception-reaction times.

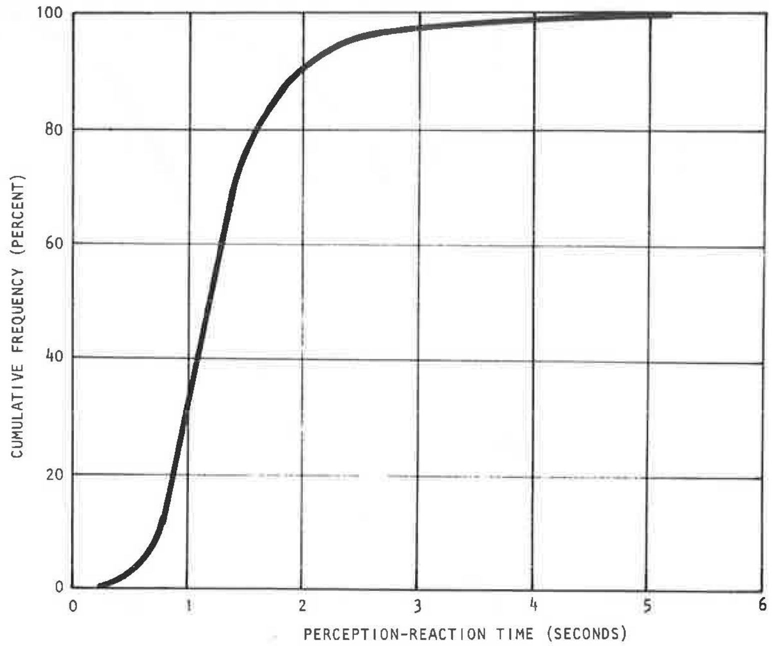


Table 5. Observed deceleration rate.

Intersection Approach	Change Interval[a]	Mean Approach Speed (mph)	Mean Rate (ft/s/s)	Standard Deviation	85th Percentile Rate (ft/s/s)
University Drive	5Y	33.69	7.00	3.80	11.5
Southern Avenue					
Day	5Y	34.97	10.73	3.02	13.9
Night	5Y	35.77	11.60	2.57	14.8
US-60	5Y + 3AR	35.88	11.79	3.43	15.8
First Avenue	3Y + 2AR	39.04	12.45	3.52	16.1
Sixth Street	3Y + 2AR	31.63	13.87	4.54	18.2
Broadway Boulevard					
Day	3.6Y + 2AR	37.41	12.83	4.12	17.2
Night	3.6Y + 2AR	36.32	9.67	3.06	12.5
All approaches		36.13	11.58		

[a]Denotes design and duration of change interval. For example, 5Y + 3AR indicates 5 s of yellow plus 3 s of all-red time.

Figure 3. Cumulative frequency distribution of observed deceleration rates.

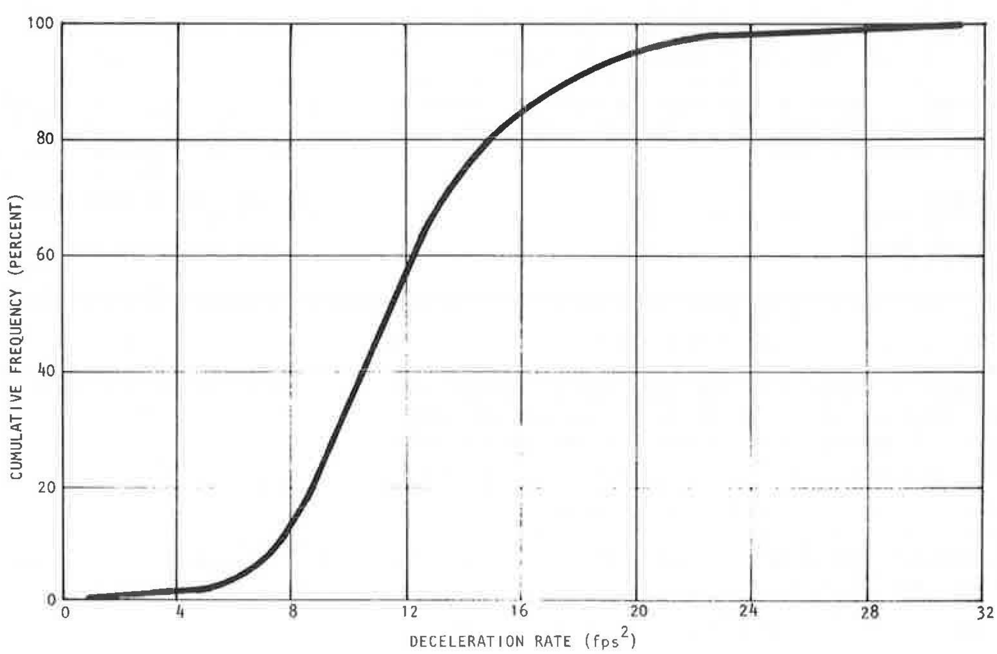


Table 6. Percentage of last vehicles through intersection on red indication.

Intersection Approach	Change Interval[a]	Sample Size of Last Vehicle Through Intersection	Percentage Entering on Red Indication
University Drive	5Y	16	0
Southern Avenue			
Day	5Y	69	2.9
Night	5Y	49	0
US-60	5Y + 3AR	87	2.3
First Avenue	3Y + 2AR	152	29.6
Sixth Street	3Y + 2AR	67	8.9
Broadway Boulevard			
Day	3.6Y + 2AR	156	8.3
Night	3.6Y + 2AR	96	1.0

[a]Denotes design and duration of change interval. For example, 5Y + 3AR indicates 5 s of yellow plus 3 s of all-red time.

Table 7. Comparison of Phoenix area sites (day observations).

Item	Sites Compared: US-60 and University Drive	US-60 and Southern Avenue	Southern Avenue and University Drive
First vehicle to stop			
Approach speed	N	N	N
Perception-reaction time	N	N	N
Distance from intersection	N	N	N
Deceleration rate	S	N	S
Last vehicle through intersection			
Approach speed	S	S	N
Distance from intersection	N	N	N

Note: N = difference is not significant, and S = significant difference.

Table 8. Comparison of Tucson area sites (day observations).

Item	Sites Compared: First Avenue and Broadway Boulevard	First Avenue and Sixth Street	Sixth Street and Broadway Boulevard
First vehicle to stop			
Approach speed	S	S	S
Perception-reaction time	N	S	S
Distance from intersection	S	S	S
Deceleration rate	N	S	N
Last vehicle through intersection			
Approach speed	N	S	S
Distance from intersection	N	S	S

Note: N = difference is not significant, and S = significant difference.

Table 9. Comparison of day and night behavior.

Item	Site: Southern Avenue	Broadway Boulevard
First vehicle to stop		
Approach speed	N	N
Perception-reaction time	N	N
Distance from intersection	N	N
Deceleration rate	N	S
Last vehicle through intersection		
Approach speed	S	N
Distance from intersection	N	N

Note: N = difference is not significant, and S = significant difference.

measured parameters were computed. Where data from intersections were combined for a particular analysis, these values were computed for the combined data. Potential differences in behavior were analyzed by examining differences in the means for specific groups or pairs. In all cases, the 95 percent confidence level was used for the purpose of assessing statistical significance.

Table 7 gives a summary of the results for a comparison of the sites in the Phoenix metropolitan area. For this analysis, a particular site was compared with each of the other sites in that metropolitan area. Note that there was a statistically significant difference in the deceleration rate at the University Drive site even though there was no significant difference in the other parameters for the first vehicle to stop. For the last vehicle through the intersection after the beginning of the yellow interval, there were two cases where the differences in the approach speeds were significant.

A similar comparison of the sites in Tucson yielded a somewhat different set of results (Table 8). For this group of intersections, the comparison revealed significant differences in a number of measures of behavior.

Table 9 gives the results of the comparison of the day and night studies at the two sites selected for that purpose. In this case, the observed day and night behavior at each of the two sites were compared. Again, there were some differences between the two study sites. As was previously indicated, the observed deceleration rates were significantly lower at night for the Broadway Boulevard location. The Southern Avenue site revealed no difference in the day and night comparison, except that the mean approach speed for the last vehicle through the intersection was significantly higher at night.

All three of the study locations in Tucson had change intervals with the all-red phase; thus, an analysis of the influence of the yellow only versus the yellow plus all-red intervals required a comparison of the Phoenix and Tucson area sites. For this analysis, the Southern Avenue and the University Drive study approaches in the Phoenix area were used. The data for these intersections were compared with the information for the three Tucson intersections. The results of this analysis are given in the table below (note, N = difference is not significant, and S = significant difference):

Item	Difference
First vehicle to stop	
Approach speed	S
Perception-reaction time	N
Distance from intersection	S
Deceleration rate	S
Last vehicle through intersection	
Approach speed	S
Distance from intersection	N

The mean approach speeds for both the stopping and through vehicles were significantly higher in Tucson. In contrast, however, the distance from the intersection at the beginning of the yellow interval was significantly less and the deceleration rate was higher in Tucson.

CONCLUSIONS

The results of this study reflect the behavior of drivers given the existing intersections and traffic signal timing. It is not possible to anticipate what the resulting driver behavior would be with a different set of conditions. Based on the observations and analyses that were made a part of this study, the following conclusions can be drawn:

1. The observed mean deceleration rates chosen by drivers at the various study sites ranged from 7.0 to 13.9 ft/s/s. The mean deceleration rate for all observations was 11.6 ft/s/s.
2. The day and night comparison of driver behavior revealed mixed results in that the mean deceleration rate was not significantly different during the nighttime period at one site; however, there was a decrease in the mean rate from 12.9 to 9.7 ft/s/s at the Broadway Boulevard location.
3. With respect to the perception-reaction times of drivers, the observed mean time for each of the sites ranged from 1.16 to 1.55 s. The mean for all approaches was 1.30 s. The 85th percentile value for the time measurement ranged from 1.5 to 2.1 s.

Although the nighttime studies revealed mean times that were slightly less at both sites, these differences were not statistically significant. Factors such as approach speed, distance from the intersection at the beginning of the yellow interval, and the deceleration rate that is used by the driver had little or no influence on the perception-reaction time.

4. The comparison of study sites in each of the metropolitan areas also provided mixed results. In the Phoenix area, the mean deceleration rate at the University Drive location was significantly less than at the other two sites even though most of the other measures showed no significant differences. Similar comparisons for the Tucson sites revealed differences in a number of the measured parameters.
5. In terms of the comparison of the yellow only versus the yellow plus all-red change intervals, there were no significant differences in the perception-reaction times. This was true when the intersection with an all-red interval in the Phoenix area was compared with the other sites that had a yellow only interval. It was also true when the intersections in Tucson were compared with those in Phoenix.

The analysis of the deceleration rates for the two types of change intervals did not yield consistent results. Although the deceleration rates for the Tucson sites were higher than those observed in the Phoenix area, there was a variation in the significance of the differences between intersections with the same type of change interval.

In reviewing the comparisons of the observed behavior, the use of some degree of caution should be exercised. For example, small differences in deceleration rates may be statistically significant but may have no appreciable affect on the evaluation and solution of traffic problems.

In addition, the project staff spent considerable time observing the operations of these intersections during the course of the study. Although the observed deceleration rates and the perception-reaction times varied from the values recommended by current practice, it should be recognized that the intersections appeared to be operating reasonably well with respect to the change interval.

ACKNOWLEDGMENT

This paper is based on research undertaken as part of a Highway Planning and Research Program project entitled, An Evaluation of Driver Behavior at Signalized Intersections. The research was conducted by the Arizona Transportation and Traffic Institute, College of Engineering, University of Arizona, and the College of Engineering, Arizona State University, in cooperation with the Arizona Department of Transportation and the Federal Highway Administration (FHWA), U.S. Department of Transportation.

Discussion

Peter S. Parsonson

The most controversial issue in the preparation of the second edition of the Institute of Transportation Engineers (ITE) Transportation and Traffic Engineering Handbook (9) was the calculation of the yellow and all-red intervals. The final draft of the handbook, published in 1982, called for the use of a perception-reaction time of 1 s. The paper by Wortman and Matthias does not include within its scope an evaluation of this current practice; but it is important that the discussions of their paper include a comparison of their findings with the ITE handbook recommendations.

Wortman and Matthias observed more than 800 stopping drivers and found an average perception-reaction time of 1.3 s. ITE uses 1.0, so we need to consider whether the ITE procedure gives yellow times that are 0.3 s too short.

There are two reasons why this paper does not represent a challenge to the currently accepted value of 1.0 s for perception-reaction time. First, the perception-reaction times reported by Wortman and Matthias are for the first vehicle to stop after the yellow begins. For all we know, many of these vehicles were so far from the intersection when the yellow came on that it was out of the question to continue through the intersection. These drivers could react at their leisure and brake comfortably to a stop. They tell us nothing about how fast they would react when there is a real decision to be made.

Second, it is necessary to consider perception-reaction time and deceleration rate jointly, as these two variables are tied together as determinants of stopping distance and required yellow time. The Wortman and Matthias data for perception-reaction time and deceleration rate, taken together, produce calculated yellow times comparable with those derived from the ITE guidelines.

For example, consider an approach with a speed of 50 mph. The ITE values of 1 s for perception-reaction time and 10 ft/s/s for deceleration rate yield a yellow time of 4.7 s. The calculation, when using the Wortman and Matthias values of 1.3 s and 11.6 ft/s/s, gives 4.5 s, which is just a couple of tenths of a second less than the ITE result. That difference makes sense because ITE assumes a wet road, while the Wortman and Matthias data came from dry roads. The average drivers in the Wortman and Matthias study may be allowing themselves a little extra time to react because they know they can easily make up for it by braking a little more heavily on a dry road. At 35 mph, the two sets of values give an identical 3.5 s.

It is well known (10) that the minimum yellow time can be computed by dividing the stopping distance by the approach speed, as follows:

$$Y = \text{stopping distance/speed} = [\nu t + (\nu^2/2a)]/\nu = t + (\nu/2a) \quad (1)$$

If stopping distance is measured directly by observing the behavior of traffic at the onset of the yellow interval, the required yellow time can be calculated without the need to make assumptions for t and a. From 50 mph, for example, Zegeer (11) found that 90 percent of the drivers will decide to stop if they are 350 ft from the intersection when the yellow begins. A yellow time long enough for 90 percent of the drivers is as follows:

$$Y = \text{stopping distance/speed} = 350/(50 \times 1.47) = 4.76\text{ s} \quad (2)$$

This minimum yellow is entirely empirical and is independent of assumptions for t and a. It can be used to calculate combinations of t and a that will satisfy it, as follows:

$$4.76 = t + [(50 \times 1.47)/2a] = t + (36.75/a) \quad (3)$$

A table of combinations that satisfy this relation is given below:

t	a
0.75	9.2
1.0	9.8
1.1	10.0
1.3	10.6
1.5	11.3
2.5	16.3

This table shows that provision of a yellow of 4.76 s allows the driver stopping from 350 ft a wide range of responses. A quick-reacting driver can decelerate comfortably to a stop, while one slower to react must compensate by braking more forcefully. For example, a driver who reacts in 1-s flat can decelerate comfortably at 9.8 ft/s/s, while the driver who requires 1.3 s must brake more heavily at 10.6 ft/s/s.

The point is that it is the combination of t and a that counts. The combination reported by Wortman and Matthias is in harmony with the combination suggested by ITE. The small difference probably can be accounted for as dry road versus wet road.

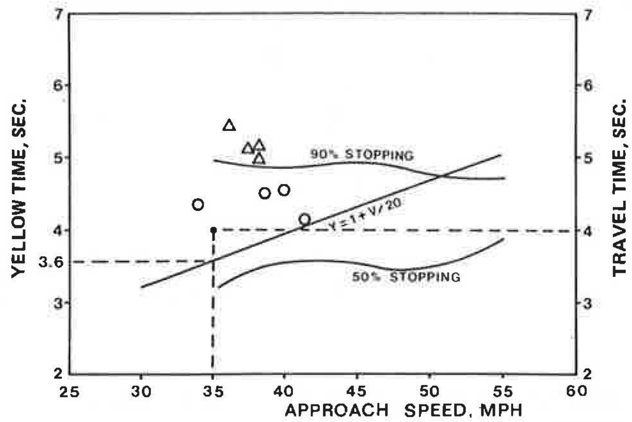
Figure 4 shows Zegeer's observed distances for 50 and 90 percent stopping, converted to travel time to the intersection by dividing by the approach speed. The data are for 2100 drivers responding to the yellow interval in Kentucky. The upper curve means that 90 percent of these drivers stopped if the yellow came on when they were about 4.8 s of travel time from the intersection. The curve is quite flat; values of just under 5 s were observed over the entire range of speeds studied. At 50 mph, 4.8 s is a yellow time long enough for 90 percent of the Kentucky drivers. These motorists seem to be indicating that they want a constant yellow of almost 5 s, regardless of the approach speed.

Figure 4 also shows a curve for yellow time calculated by using the ITE values of t = 1 and a = 10, as follows:

$$Y = t + (\nu/2a) = 1 + [\nu/(2 \times 10)] = 1 + (\nu/20) \quad (4)$$

Consider a driver approaching at 35 mph. Suppose he or she is shown a yellow calculated by using t = 1 and a = 10, which yields 3.6 s. Now suppose the vehicle is 4 s from the intersection when the yellow comes on. There is a 70 percent chance that the driver will decide to stop (obtained by interpolating between Zegeer's 50 and 90 percent curves). There is a 30 percent chance that he or she will

Figure 4. Certain relation involving yellow time, travel time, and approach speed.



decide to clear, in which case the driver will then run the red because of the deficient yellow. The driver seems to want 4.8 s--he or she needs 4.0 s--but the driver gets only 3.6 s, so he or she may well run the red. The Zegeer data suggest that yellows calculated by the ITE guidelines, or by the Wortman and Matthias data, are too short for speeds less than 50 mph if we are trying to meet the needs of the 90th percentile driver.

The eight plotted points on Figure 4 cannot be explained without further discussion of entering on the red because of yellow time deficiency. The First Avenue site was observed to have an extremely high percentage of vehicles entering on the red signal compared with other locations. The reason for this seems clearly to be that this site was much more deficient in yellow time than any of the others. Table 10 shows the deviations of the observed yellow times from those calculated by using t = 1 s and a = 10 ft/s/s for the eight sites. Figure 5 shows the percentage entering on the red plotted against these deviations expressed as a surplus or deficiency in yellow time. The figure shows that the First Avenue site can be understood if the percentage entering on the red increases exponentially (or logarithmically) with increasing deficiency of yellow. The figure also suggests that, at zero deficiency (obtained by using t = 1 and a = 10 for our calculations), we can expect about 3 percent of the vehicles to enter on the red. This might be considered an acceptable level, inasmuch as it is a common rule of thumb in the positive-guidance area that an operational deficiency exists if traffic conflicts or erratic maneuvers are exhibited by more than 3 percent of the approaching vehicles.

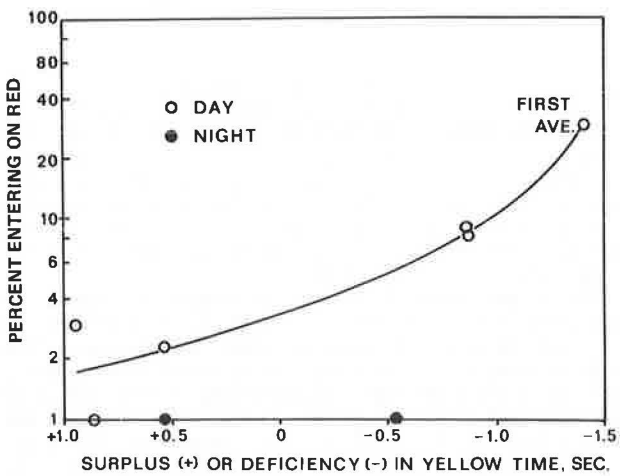
The final comment is an explanation of the eight plotted points on Figure 4. These points represent the first vehicles to stop at the eight sites reported by Wortman and Matthias. Specifically, the ordinate of each point is the mean travel time to the intersection for the first vehicles to stop. The ordinates are not the yellow times at these intersections. The points plotted as triangles are the four sites shown in Table 10 to have a surplus of yellow time, and the circles are four locations deficient in yellow time. All of these drivers decided to stop because they correctly judged that their travel times to the intersection were too great to allow entry on the yellow. The average ordinate of these points, which represents 100 percent stopping, lies close to Zegeer's 90 percent curve and indicates good agreement. It is most interesting to note that the sites deficient in yel-

Table 10. Deviations of yellow time from calculated standard.

Intersection Approach	Percentage Entering on Red	85th Percentile Approach Speed[a] (mph)	Calculated Standard Yellow Time[b] (s)	Actual Yellow Time (s)	Surplus(+) or Deficiency(-)[c] (s)
University Drive	0	44.6	4.2	5	+0.8
Southern Drive					
Day	2.9	41.6	4.1	5	+0.9
Night	0	44.7	4.4	5	+0.6
US-60	2.3	46.4	4.5	5	+0.5
First Avenue	29.6	46.7	4.4	3	-1.4
Sixth Street	8.9	39.1	3.8	3	-0.8
Broadway Boulevard					
Day	8.3	45.3	4.4	3.6	-0.8
Night	1.0	43.1	4.2	3.6	-0.6

[a] Average of last vehicle through the intersection and first vehicle to stop.
[b] Derived by using the equation 1 + (v/2a ± 64.4g), where a = 10 ft/s/s and g = approach gradient.
[c] Derived by subtracting actual yellow time from calculated standard yellow time.

Figure 5. Percent entering on red versus yellow-time surplus or deficiency.



low all plotted lower than those with a surplus. That is, drivers approaching yellow-deficient signals were willing to stop from locations closer to the intersection, which required quicker reaction and/or heavier braking. The data suggest that the drivers are aware of the lengths of the yellow times at these intersections and do factor this into their decision whether to stop. This finding conflicts with the 1979 report by Stimpson (12), which indicated that the length of the yellow interval does not affect driver behavior.

Discussion

Jack A. Butler

The work of Wortman and Matthias has produced data that confirms the findings of other researchers and reinforces current trends in calculating vehicle change intervals. The conclusions of the authors, however, do not fully explain the data and fail to answer questions, stated and implied, in their report. This discussion will expand their analysis and propose further explanations of the data.

The first goal of the subject study was the determination of actual deceleration rates applied in stopping a vehicle at the monitored signal-controlled intersections. By employing field study methods to survey unsuspecting drivers, the possibility of driver behavior modification was removed.

Given that it has been shown (13) that acceptable deceleration rates fall within the range of 8-12 ft/s/s (15th to 85th percentiles), the observed values generally fall within the upper half of the acceptable range. The two deviating means can be explained by the approach geometry.

Although not included in the report, the University Drive site includes an at-grade railroad crossing within the stopping distance. Wortman disclosed in a conversation that they had observed drivers applying a higher deceleration rate on the upstream side of the crossing. The reported mean of 7.0 ft/s/s is an average that may not accurately indicate a rate used at a signalized intersection.

The upper limit on the range of observed rates of 13.9 ft/s/s at Sixth Street includes the effect of gravity on slowing the vehicle. Parsonson and Santiago (14) published a report that details an easy method to evaluate the effect of roadway slope on vehicle deceleration. Removing the deceleration due to gravity on the 2 percent uphill grade produces a rate of 12.6 ft/s/s. It is this value that should be used in discussing driver behavior, as it is the force perceived by the motorist.

All observations were made on dry pavement. Because it might be expected that wet pavement conditions would lower the study values for deceleration, Wortman and Matthias' findings tend to reinforce the use of 10 ft/s/s as a mean deceleration rate in yellow interval calculations.

The second goal of the study was the collection of nighttime data and the evaluation of various intersection environments. The data do not indicate any differences among these factors except as noted above.

The third goal, evaluation of all-red intervals, was not fully accomplished. The measure of effectiveness selected--vehicles entering on red--is indicative of yellow interval performance. It has been shown (5) that the length of the yellow interval does not effect driver behavior. Indeed, it is the yellow interval that must conform to driver behavior. A too-short yellow will naturally produce a large number of red-aspect violations. The subject study provides some confirmation of that principle. Data were not presented that allow the comparison of the all-red timing and intersection geometry. Further work is needed to accomplish this third goal.

It is possible to calculate more reasonable yellow intervals by applying the following formula (14):

$$y = t + [v/(2a + 64.4g)] \qquad (5)$$

where

t = reaction time (s),
v = 85th percentile approach speed (ft/s),
a = deceleration rate (ft/s/s), and

g = grade as percent divided by 100.

When this is done, a clear relation emerges between yellow time deficiency and red-aspect violations.

Table 11 lists the values relevant to this discussion; Tables 12 and 13 illustrate yellow and all-red times over a variety of speeds and intersection widths. The 85th percentile speeds were estimated by adding one standard deviation to the 50th percentile speed. The means for the last car through and the first to stop were combined, as were the standard deviations. This technique attempts to balance the effect of acceleration of vehicles that did not stop with the deceleration that occurs dur-

Table 11. Reported and derived data.

Item	University Drive	Southern Avenue Day	Southern Avenue Night	US-60	First Avenue	Sixth Street	Broadway Boulevard Day	Broadway Boulevard Night
Yellow interval (s)	5	5	5	5	3	3	3.6	3.6
All-red interval (s)	0	0	0	3	2	2	2	2
85th percentile speed (ft/s)	63	61	66	63	65	55	64	60
Calculated yellow (s)	4.2	4.1	4.3	4.2	4.3	3.6	4.2	4.0
Actual minus calculated yellow (s)	0.8	0.9	0.7	0.8	-1.3	-0.6	-0.6	-0.4
Last through vehicles entering on red (%)	0	3	0	2	30	9	8	1
Mean stopping distance (ft)	269	263	272	263	238	203	251	241
Stopping distance implied by calculated yellow (ft)	265	250	271	265	280	198	269	240
Distance traveled at 85th percentile speed during actual yellow (ft)	315	305	330	315	195	165	230	216
Mean reaction time (s)	1.3	1.5	1.4	1.4	1.2	1.6	1.2	1.1
Mean rate of deceleration (ft/s/s)	7.0	10.8	11.6	11.8	12.4	12.6[a]	12.9	9.7
Deceleration rate implied by current yellow for 85th percentile speed (ft/s/s)	7.9	7.6	8.3	7.9	16.3	12.5[a]	12.3	11.5

[a] Value has been adjusted to remove effect of approach grade.

Table 12. Time (in seconds) of yellow interval.

85th Percentile Speed	Uphill (%) +4	+3	+2	+1	Level	Downhill (%) -1	-2	-3	-4
25	2.63	2.68	2.73	2.78	2.84	2.90	2.96	3.03	3.11
30	2.95	3.01	3.07	3.14	3.21	3.28	3.36	3.44	3.53
35	3.28	3.35	3.42	3.49	3.57	3.66	3.75	3.85	3.95
40	3.60	3.68	3.76	3.85	3.94	4.04	4.14	4.25	4.37
45	3.93	4.02	4.11	4.20	4.31	4.42	4.54	4.66	4.80
50	4.26	4.35	4.45	4.56	4.68	4.80	4.93	5.07	5.22
55	4.58	4.69	4.80	4.92	5.04	5.18	5.32	5.47	5.64
60	4.91	5.02	5.14	5.27	5.41	5.56	5.71	5.88	6.06
65	5.23	5.36	5.49	5.63	5.78	5.94	6.11	6.29	6.48

Yellow Intervals by Grade of Approach

Note: Yellow interval table values derived from the formula:

$$y = t + [v/(2a + 64.4g)]$$

where

y = yellow interval (s),
t = reaction time (set at 1.0 s),
v = 85th percentile approach speed (ft/s),
a = deceleration rate (set at 10 feet/s/s), and
g = grade of approach over the braking distance (percent divided by 100).

Table 13. Time (in seconds) of all-red interval.

85th Percentile Speed	All-Red Intervals by Width of Approach (ft) 20	30	40	50	60	70	80	90	100	110	120
25	1.09	1.36	1.63	1.90	2.18	2.45	2.72	2.99	3.27	3.54	3.81
30	0.91	1.13	1.36	1.59	1.81	2.04	2.27	2.49	2.72	2.95	3.17
35	0.78	0.97	1.17	1.36	1.55	1.75	1.94	2.14	2.33	2.53	2.72
40	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04	2.21	2.38
45	0.60	0.76	0.91	1.06	1.21	1.36	1.51	1.66	1.81	1.97	2.12
50	0.54	0.68	0.82	0.95	1.09	1.22	1.36	1.50	1.63	1.77	1.90
55	0.49	0.62	0.74	0.87	0.99	1.11	1.24	1.36	1.48	1.61	1.73
60	0.45	0.57	0.68	0.79	0.91	1.02	1.13	1.25	1.36	1.47	1.59
65	0.42	0.52	0.63	0.73	0.84	0.94	1.05	1.15	1.26	1.36	1.47

Note: All-red interval table values derived from the formula:

$$r = (w + l)/v$$

where

r = all-red interval (s),
w = width of intersection (ft),
l = length of vehicle (set at 20 ft), and
v = speed of vehicle (ft/s).

ing the driver reaction time. The values for t and a are 1.0 s and 10 ft/s/s, respectively.

One also finds a close agreement between the predicted stopping distance and that that was actually observed. The longer reaction times were offset by higher deceleration rates. The only exception is First Avenue, where the observed mean is 15 percent less than expected. This is also the approach with the highest red-aspect violation rate (30 percent) and the greatest yellow time deficiency. It may be concluded that many drivers chose not to apply the high rate of deceleration necessary on this approach to keep from entering on red.

A similar, but less severe, yellow time deficiency and red-aspect violation relation exists at the Sixth Street and Broadway Boulevard sites. The lower violation rate observed at night for Broadway Boulevard is directly related to the lower approach speed at night.

The yellow deficiency can be further refined by determining the deceleration rates implied by the existing signal timing at the 85th percentile approach speed. These values all fall within the acceptable range except for those locations that experience the highest red-aspect violation rates. At the First Avenue site, a deceleration rate of 16.3 ft/s/s would be necessary given a 1.0-s reaction time. The higher-speed drivers have obviously indicated their dislike of such a high rate by entering on red.

Taken together, the data show that drivers reject deceleration rates significantly greater than 12 ft/s/s and that there is an evident relation between the length of the yellow and red-aspect violation. Implied in this is the observed absence of drivers who adjust their behavior significantly to conform to yellow times.

It may be further stated that driver action was not affected by the presence of an all-red interval, which gives support to the findings of Benioff and others (15) that an increase in red-aspect violation does not accompany the use of such intervals. Beyond that, it is not possible to examine the given all-red times, since the critical factor of intersection width was omitted. Absent as well was a discussion of right-angle and other accidents that would provide insight on the effectiveness of all-red intervals.

The final goal of the subject study was the recording of driver reaction times. The authors admit in the paper that they had no way of determining the reaction time of drivers who chose not to stop. It may be further suspected that they had no way of determining the true reaction time; i.e., did the drivers react in a leisurely manner.

In spite of those shortcomings, the data seem to refute the contention of the American Association of State Highway Officials (AASHO) (16) that the 95th percentile alert reaction time is 1.0 s. The results also place an upper limit on the range of driver reaction times.

As Parsonson shows in his discussion, for any given speed there is a range of reaction times and deceleration rates that produce the same stopping distance. Because the various sites had similar approach speeds, it is not possible to select the correct combination. Further research is needed to resolve the issue, including the question of the applicability of observed values to design calculations.

Committee 4A-16 (Use and Timing of Vehicle Change Intervals) of ITE will soon begin field experiments to evaluate the effectiveness of a range of reaction times and deceleration rates in calculating the yellow interval length.

The work of Wortman, Matthias, and their team members comes at a time when a decision must be made on selecting a realistic change interval calculation methodology. In that context, their report provides considerable guidance.

REFERENCES

1. Arizona Traffic Accident Summary--1980. Arizona Department of Transportation, Phoenix, 1980.
2. L. Rach. Traffic Signals. In Transportation and Traffic Engineering Handbook, 2nd ed., Prentice-Hall, Englewood Cliffs, NJ, 1982.
3. Traffic Engineering Policies, Guides, and Procedures. Arizona Department of Transportation, Phoenix, Policy PGP-4-4B-3-0, Oct. 1980.
4. W.L. Williams. Driver Behavior During the Yellow Signal Interval. TRB, Transportation Research Record 644, 1977, pp. 75-78.
5. W.A. Stimpson, P.L. Zador, and P.J. Tarnoff. The Influence of the Time Duration of Yellow Signals on Driver Response. ITE Journal, Nov. 1980, pp. 22-29.
6. P.S. Parsonson and A. Santiago. Traffic-Signal Change Period Must be Improved. Public Works, Sept. 1981, pp. 110-113.
7. D. Gazis, R. Herman, and A. Maradudin. The Problem of the Amber Signal Light in Traffic Flow. Operations Research, Vol. 8, No. 1, Jan.-Feb. 1960.
8. R.S. Jenkins. A Study of Selection of Yellow Clearance Intervals for Traffic Signals. Michigan Department of State Highways and Transportation, Lansing, Rept. TSD-TR-104-69, Feb. 1969.
9. ITE. Transportation and Traffic Engineering Handbook, 2nd ed. Prentice-Hall, Englewood Cliffs, NJ, 1982.
10. H.H. Bissell and D.L. Warren. The Yellow Signal Is Not a Clearance Interval. ITE Journal, Vol. 51, No. 2, Feb. 1981, pp. 14-17.
11. C.V. Zegeer. Effectiveness of Green-Extension Systems at High-Speed Intersections. Division of Research, Bureau of Highways, Kentucky Department of Transportation, Lexington, Res. Rept. 472, May 1977.
12. W.A. Stimpson and others; A.M. Voorhees and Associates, Inc. The Influence of the Time Duration of Yellow Traffic Signals on Driver Response. Insurance Institute for Highway Safety, Washington, DC, Jan. 1979.
13. P. Olson and R. Rothery. Deceleration Levels and Clearance Times Associated with Amber Phase of Traffic Signals. Traffic Engineering, April 1972, pp. 16-19 and 62-63.
14. P. Parsonson and A. Santiago. Design Standards for Timing and Traffic Signal Clearance Period Must Be Improved to Avoid Liability. Compendium of Technical Papers, ITE Annual Meeting, Aug. 1980, pp. 67-71.
15. B. Benioff, F. Dock, and C. Carson. A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals. FHWA, Rept. FHWA-RD-78-77, May 1980.
16. A Policy on Design Standards for Stopping Sight Distance. AASHO, Washington, DC, 1971.

Publication of this paper sponsored by Committee on User Information Systems.